NCHRP
REPORT 568

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Riprap Design Criteria, Recommended Specifications, and Quality Control

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Riprap Design Criteria, Recommended Specifications, and Quality Control

P. F. Lagasse
P. E. Clopper
L. W. Zevenbergen
J. F. Ruff
AYRES ASSOCIATES INC.
Fort Collins, CO

Subject Areas
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To provide additional expertise and experience with riprap design and installation outside the United States, two of the research team members were from Europe. Dr. M.H. Heibaum, a geotechnical engineer from the German Federal Waterways Engineering and Research Institute (BAW), provided access to the current literature and latest advances in riprap design, specifications, and field implementation of testing and inspection procedures to install and maintain riprap revetment along Germany's extensive waterway system. Mr. H.J. Verheij, a hydraulic engineer from Delft Hydraulics provided access to the current literature and latest advances in riprap technology in the Netherlands. Dr. Verheij is a technical contributor to the *Manual on the Use of Rock in Hydraulic Engineering* published by the Netherlands Center for Civil Engineering Research and Codes and provided the research team insight into the development of this reference work and advances since its publication.

The scope of this study required consideration of the production and construction aspects of rock riprap. Mr. R.M. Madden (Pine Bluff Sand and Gravel, Baton Rouge, Louisiana) and Mr. J.K. Egbert (Nordic Industries, Marysville, California) served on our advisory panel and provided the research team a direct linkage to the rock production and contracting industry. Dr. E.V. Richardson (Ayres Associates) and Dr. S. Maynord (Vicksburg, Mississippi) also served on the advisory panel and reviewed initial and final drafts of many sections of this report.

A special acknowledgment is made to the 33 respondents to the survey, including 23 state DOTs, which helped establish the existing state of practice in riprap design, installation, and maintenance as a basis for this study.

The participation, advice, and support of NCHRP panel members throughout this project are gratefully acknowledged.

FOREWORD

By Staff
Transportation Research Board

This report presents the findings of a study to develop design guidelines, material specifications and test methods, construction specifications, and construction, inspection, and quality control guidelines for riprap at streams and riverbanks, bridge piers and abutments, and bridge scour countermeasures. Recommendations are provided on a design equation or design approach for each application. Filter requirements, material and testing specifications, construction and installation guidelines, and inspection and quality control procedures are also recommended for each riprap application. To guide the practitioner in developing appropriate designs for riprap armoring systems for these applications, the findings and recommendations are combined to provide design guideline appendixes for (1) Design and Specification of Rock Riprap Installations and (2) Construction, Inspection, and Maintenance of Rock Riprap Installations. This report will be particularly useful to bridge, hydraulic, and highway engineers, as well as bridge maintenance and inspection personnel responsible for design, construction, inspection, and maintenance of bridges and other highway structures.

Many different techniques are currently used to determine the size and extent of a riprap installation, and existing techniques and procedures for design of riprap protection can be confusing and difficult to apply. Depending on the technique used to size riprap, the required size of stone can vary widely. Most states have specifications for classifying riprap size and gradation, but there is not a consistent classification system or set of specifications that can be used when preparing plans or assembling a specification package for a project. In addition, various construction practices are employed for installing riprap; many of them are not effective and projects requiring the use of riprap historically have suffered from poor construction practices and poor quality control. The intent of this study was to develop a unified set of guidelines, specifications, and procedures that can be accepted by the state DOTs.

Under NCHRP Project 24-23, Ayres Associates reviewed foreign and domestic technical literature and surveyed practitioners including hydraulic engineers from state DOTs, FHWA, other federal agencies, and consulting firms to establish the state of practice in riprap design. Design equations for sizing riprap were evaluated with sensitivity analyses using laboratory and/or field data for the applications of interest. Based on the sensitivity analyses, a design equation or design approach is recommended for each application. However, sizing the stone is only the first step in the comprehensive design, production, installation, inspection, and maintenance process required for a successful riprap armoring system. Filter requirements, material and testing specifications, construction and installation guidelines, and inspection and quality control procedures are also necessary. Specific criteria or approaches for these requirements are developed for each riprap application area.

In this report, riprap failure mechanisms are identified as a basis for developing inspection guidance, and selected case studies of failures are used to emphasize the need for post-flood/post-construction inspection. In addition, concepts (but not design guidance) for a bioengineering or hybrid design approach for bank stabilization using a combination of rock and vegetative treatments are discussed. Design guidelines were developed and are included as appendixes to this report. Typical details for the riprap applications investigated in this study are available on the TRB website in two computer-aided design (CAD) formats. These files can be downloaded from the project description web page (www4.trb.org/trb/crp.nsf/All+Projects/NCHRP+24-23).

CONTENTS

1 Summary

5 **Chapter 1** Introduction and Research Approach
5 1.1 Scope and Research Objectives
5 1.2 Research Approach
6 1.3 Research Tasks
7 1.4 Special Requirements

9 **Chapter 2** Findings
9 2.1 Review of Technical Literature
9 2.2 Survey for Current State of Practice
10 2.3 Riprap – An Integrated System
14 2.4 Synthesis of Current State of Practice

47 **Chapter 3** Interpretation, Appraisal, and Applications
47 3.1 Introduction
47 3.2 Revetment Riprap
95 3.3 Bridge Pier Riprap
100 3.4 Bridge Abutment Riprap
105 3.5 Riprap for Countermeasures
110 3.6 Riprap Design Variables
111 3.7 Riprap Failure Mechanisms
127 3.8 Bioengineering/Hybrid Design
135 3.9 Riprap Design Software
135 3.10 Implementation

139 **Chapter 4** Conclusions and Suggested Research
139 4.1 Applicability of Results to Highway Practice
139 4.2 Conclusions and Recommendations
143 4.3 Suggested Research

144 **Chapter 5** References

A-1 **Appendix A** Bibliography of Current Practice

B-1 **Appendix B** Survey of Current Practice

C-1 **Appendix C** Guidelines for the Design and Specification of Rock Riprap Installations

D-1 **Appendix D** Guidelines for the Construction, Inspection, and Maintenance of Rock Riprap Installations

SUMMARY

Riprap Design Criteria, Recommended Specifications, and Quality Control

Overview

This research accomplished its basic objectives of developing design guidelines; recommended material specifications and test methods; recommended construction specifications; and construction, inspection, and quality control guidelines for riprap for a range of applications, including revetment on streams and riverbanks, bridge piers and abutments, and bridge scour countermeasures such as guide banks and spurs. This study did not involve any original laboratory experiments, but some analysis (specifically, one- and two-dimensional computer modeling) was necessary to address issues related to input hydraulic variables for design. A fundamental premise of this study is that riprap is an integrated system and as such, successful performance of a riprap installation depends on the response of each component of the system to hydraulic and environmental stresses throughout its service life.

A review of the foreign and domestic technical literature and a survey of practitioners including state DOT hydraulic engineers, FHWA Resource Center and Division/District engineers, and hydraulic engineers from other federal agencies and consulting firms were used to establish the state of practice in riprap design for each of the applications at the outset of the study. The summary of current practice in Chapter 2 is the basis for the interpretation, appraisal, and application recommendations in Chapter 3. Design equations for sizing riprap are evaluated with sensitivity analyses using laboratory and/or field data, where available, for the applications of interest to this study. Based on the sensitivity analyses, a design equation or design approach is recommended for each application.

Sizing the stone is only the first step in the comprehensive design, production, installation, inspection, and maintenance process required for a successful riprap armoring system. Filter requirements, material and testing specifications, construction and installation guidelines, and inspection and quality control procedures are also necessary. Specific criteria or approaches for these requirements are developed for each riprap application area.

Guidance on determining design variables and design examples are provided for each application. Design of riprap for overtopping flow conditions on roadway embankments and flow control countermeasures is also considered. An annotated description of riprap design software and reference data sets for testing design software or spreadsheets are included. Riprap failure mechanisms are identified as a basis for developing inspection guidance, and selected case studies of failures are used to emphasize the need for post-flood/post-construction inspection. In addition, concepts (but not design guidance) for a bioengineering or hybrid design approach for bank stabilization using a combination of rock and vegetative treatments are discussed.

Revetment Riprap

Based on a screening of the many revetment riprap design equations found in the literature, seven equations are evaluated with sensitivity analyses using both field and laboratory data.

One, from the U.S. Army Corps of Engineers (USACE) Engineering Manual (EM) 1601 (1991), is recommended for streambank revetment design. Factors considered were the ability of the basic equation to discriminate between stable and failed riprap using field and laboratory data, bank and bend correction factors, and the reasonableness of safety/stability factors. Design requirements and procedures for both geotextile and granular filters are considered in detail and guidance is provided for the full life cycle of a revetment riprap system. Laboratory and field tests for both quality control and inspection and inspection guidance (with reference to the National Bridge Inspection Standards) are provided. A standard riprap gradation specification that considers design, production, and installation requirements is proposed together with a standardized riprap size classification system. Installation and construction guidance for toe down and transitions is developed for the revetment application.

Bridge Pier Riprap

There have been many recent studies for sizing pier riprap using a variety of parametric groupings with significant variation in recommended stone size. After a preliminary screening, the FHWA Hydraulic Engineering Circular (HEC) 18 (Richardson and Davis, 1995)/HEC-23 (Lagasse et al., 2001) equation, which was derived from work by A.C. Parola, J.S. Jones, and A.C. Miller (1989), is compared to several other equations using three laboratory data sets. Based on this sensitivity analysis, the HEC-23 and Parola et al. equations provide the best balance between the objective of rarely (if ever) undersizing bridge pier riprap and the desire to not be overly conservative. As these equations are very similar, the HEC-23 equation is recommended for design practice.

The laboratory results and design recommendations from a concurrent study of countermeasures to protect bridge piers from scour (NCHRP Project 24-07[2]) are evaluated regarding filter requirements, riprap extent, and other construction/installation guidelines for pier riprap. Specifically, guidelines, derived from European practice, for the use of geotextile containers as a means of placing a filter for pier riprap are presented. Construction and installation guidelines and constructability issues are investigated, including dumping versus controlled placement, underwater versus dry installation, and buried versus mounded placement.

Bridge Abutment Riprap

Based on the findings of Chapter 2, only the abutment riprap sizing approach as developed by FHWA and presented in HEC-23 was considered to be a candidate for further investigation. The approach consists of two equations: one for Froude numbers less than 0.8 and the other for higher Froude numbers. There are no field data available to test these equations and the only available laboratory data set was used to develop the equations. The FHWA equations rely on an estimated velocity, known as the characteristic average velocity, at the abutment toe. Rather than evaluating these equations using the same laboratory data set used to develop them, the method for estimating the velocity at the abutment is investigated in detail. Two-dimensional modeling was performed to evaluate the flow field around an abutment and to verify or improve the Set-Back Ratio (SBR) method for estimating velocity for the design equations. Results of the modeling indicate that if the estimated velocity does not exceed the maximum velocity in the channel, the SBR method is well suited for determining velocity at an abutment.

Riprap for Countermeasures

In general, design guidelines and specifications for riprap for countermeasures are similar to those for bankline revetment or abutments. Consequently, recommendations for revetment

riprap are adapted to the countermeasure application. Guidance for sizing and placing riprap at zones of high stress on countermeasures (e.g., the nose of a guide bank or spur) is investigated. The feasibility of using an abutment-related characteristic average velocity for countermeasure riprap sizing is evaluated, and a recommendation on an adjustment to the characteristic average velocity approach for guide bank riprap design is provided. Guidance from USACE (EM 1601) is cited for sizing riprap for spurs. An equation for sizing riprap under overtopping conditions on the embankment portion of a countermeasure is also provided.

Design Guidelines

To guide the practitioner in developing appropriate riprap designs and ensuring successful installation and performance of riprap armoring systems for bankline revetment, at bridge piers, and at bridge abutments and guide banks, the findings of Chapter 2 and the recommendations of Chapter 3 are combined to provide detailed guidelines in a set of appendixes:

- Appendix C, Guidelines for the Design and Specification of Rock Riprap Installations
- Appendix D, Guidelines for the Construction, Inspection, and Maintenance of Rock Riprap Installations

Conclusions, Recommendations, and Suggested Research

The intent of this study was to develop a unified set of guidelines, recommended specifications, and procedures that can be accepted by the state DOTs for the design, installation, and inspection of riprap for a range of applications, including at streams and river banks, at bridge piers and abutments, and on countermeasures such as guide banks. This research effort is comparable in intent to the recent work by the European Union that resulted in adoption of a unified standard for riprap that transcends geographic and institutional boundaries. Specific conclusions and recommendations are contained in Chapter 4 for each of the functional areas investigated for the riprap applications of interest to this study:

- Riprap design equations
- Filter requirements
- Material and testing specifications
- Construction/installation guidelines
- Inspection and quality control
- Other topics considered

In developing the design guidelines, additional information or data would have supported more detailed guidance in several areas. Additional research in the following areas would support extending the recommendations of this study:

- Additional computer modeling, or physical modeling in a hydraulics laboratory, to enhance design guidelines for flow control structures such as spurs and bendway weirs
- Evaluation of the results of ongoing NCHRP projects on abutment scour and abutment scour countermeasures to refine the guidelines for abutment riprap
- Additional laboratory studies and the gathering of field or performance data would support extending the results of this study for the applications investigated in this study

- Post-project monitoring and maintenance reporting by the DOTs and other bridge owners and funding to develop a performance evaluation database
- Laboratory or field studies of improved techniques for riprap transitions or toe downs to enhance guidance on this critical component of riprap design
- Field studies to assess the long-term durability and functionality of geotextile filters in a variety of riprap application environments

CHAPTER 1

Introduction and Research Approach

1.1 Scope and Research Objectives

1.1.1 Background

FHWA, USACE, and state DOTs have developed or used methods of sizing riprap for use in protecting bridge abutments, piers, channels, guide banks, dams, embankments, and other structures vulnerable to erosion. Most of the methods are based on, or have been derived from, methods originally presented by Isbash or Shields in the 1930s. Other methods of sizing riprap have resulted from empirical studies that have been designed to protect specific structures such as piers and abutments.

Existing techniques and procedures for design of riprap protection can be confusing and difficult to apply. A brief review of the literature indicates that many different techniques are used to determine the size and extent of a riprap installation. Depending on the technique used to size riprap, the required size of stone can vary widely. Most states have their own specifications for classifying riprap size and gradation, and there is not a consistent classification system or set of specifications that can be used when preparing plans or assembling a specification package for a project. In addition, various construction practices are employed for installing riprap; many of them are not effective and projects requiring the use of riprap historically have suffered from poor construction practices and poor quality control.

1.1.2 Scope of Research

From the background discussion in the previous section can be seen that many methods and criteria are available for designing riprap for erosion protection of riverbanks, bridge piers and abutments, guide banks, and other highway structures in riverine environments. Different design criteria for riprap can give differing results for protecting the same installation. In addition, the design procedure may be confusing to apply and can result in unsuitable gradations and ambiguous specifications. Many state highway departments have developed their own specifications based on trial, error, and field experience. To provide adequate protection, riprap must be properly designed and specified, and then must be installed in ways to match the specifications. Riprap placement has not always matched industry standards because of poor quality control, often at the quarry, and poor construction practices at the site. Determining the appropriate size, gradation, and/or weight of riprap is often overlooked. Construction practices of dumping and bulldozing often are not satisfactory and careful inspection must be performed.

As a result, there is a need for riprap design approaches that can be applied consistently for erosion and scour protection on river banks, bridges, and channel control structures. A consistent classification system and standard specifications and construction practices are also required to provide more reliable and cost-effective riprap installations.

The objectives of this research were to develop design guidelines; recommended material specifications and test methods; recommended construction specifications; and construction, inspection and quality control guidelines for riprap at streams and riverbanks, bridge piers, bridge abutments, guide banks, and other locations requiring scour countermeasures.

1.2 Research Approach

1.2.1 Overview

This project was, primarily, a synthesis study. No original experimental work was undertaken. The evaluation and recommendations are based on the laboratory and field data available at the time; however, some analytical work (i.e., one- and two-dimensional hydraulic modeling) was necessary to address issues related to input hydraulic variables for

design. The challenge of this research project was to (1) identify viable existing riprap design equations and collect, organize, and evaluate data related to installation and performance of riprap in the field; (2) recommend design equations, criteria, and construction specifications for specific applications; and (3) prepare guidelines for material testing and quality control of riprap at the quarry and construction site and for riprap placement. Riprap is not considered a permanent structure and future inspection and maintenance must be considered, as well. Post-construction inspection of riprap at river and bridge structures must be part of the state DOT bridge programs.

This section outlines the approach for conducting the study. During Phase I, a literature review of riprap design criteria was conducted in conjunction with a survey of various federal agencies and state DOTs through a questionnaire and interviews. The research team then synthesized and evaluated the data and prepared an interim report with recommendations for Phase II. Following a meeting with the NCHRP panel for this project, the Phase II work plan was finalized. During Phase II, riprap design equations were evaluated; material tests and recommended specifications were developed; and construction guidelines and recommended specifications for installation and inspection were prepared.

1.2.2 Integration of European Technology

This project required a review of the technical literature from domestic and foreign sources. To facilitate this review and to provide additional expertise and experience with riprap design and installation, two of the research team members were from Europe. Dr. M.H. Heibaum, a geotechnical engineer from the German Federal Waterways Engineering and Research Institute (*Bundesanstalt für Wasserbau* or BAW), provided access to the current literature and latest advances in riprap design, specifications, and field implementation of testing and inspection procedures to install and maintain riprap revetment along Germany's extensive waterway system. The BAW Code of Practice – Use of Standard Construction Methods for Bank and Bottom Protection on Waterways (MAR) and BAW Code of Practice – Use of Geotextile Filters on Waterways (MAG) are valuable summaries of the state of practice in Germany for riprap design, testing, specifications, and installation. In addition, Dr. Heibaum provided access to the Proposed Draft Harmonized Standard (18 October 2000) for Armourstone prepared by the European Committee for Standardization (*Comité Européen de Normalisation* or CEN). The draft standard (CEN 2000) was finalized by CEN in 2002.

Dr. H.J. Verheij, a hydraulic engineer from Delft Hydraulics provided access to the current literature and latest advances in riprap technology in the Netherlands. Dr. Verheij is a technical contributor to the Netherlands Center for Civil Engineering Research and Codes (CUR) *Manual on the Use of Rock in Hydraulic Engineering* (1995). This massive volume is considered the standard reference work in much of Europe on "the entire life cycle of rock structures" and provides overall guidance on planning and designing of riprap. It includes chapters on material, physical processes and design tools, inland waterway structures (revetment), construction, and maintenance. Dr. Verheij provided the research team insight into the development of this reference work and advances since its publication.

1.3 Research Tasks

Considering the research approach discussed and outlined in the previous sections, the following specific tasks were completed to accomplish project objectives. The task statements parallel, with minor modifications, those suggested in the original research problem statement. Phase I included Tasks 1 through 4 and Phase II consisted of Tasks 5 through 8.

1.3.1 Task 1 – Literature Review

The research team reviewed the technical literature from domestic and foreign sources and assessed the adequacy and extent of existing information used to design, specify, and construct riprap. Specifically, riprap design equations and techniques for determining (1) size of stone for riprap design, (2) gradation requirements, (3) material quality tests, and (4) thickness requirements were reviewed.

1.3.2 Task 2 – Survey for Current State of Practice

The research team surveyed federal agencies, state DOTs, and other agencies to determine practices used to design, specify, and construct riprap. The survey collected data on riprap design equations, classification systems, specifications, standard plans, and construction guidance. Based on the survey results, the research team interviewed selected agencies to acquire detailed information.

1.3.3 Task 3 – Synthesize Current State of Practice

Based on the findings of Tasks 1 and 2, the research team synthesized the current state of practice for designing, specifying, and constructing riprap at stream and river banks, bridge piers, bridge abutments, guide banks, and other locations requiring scour countermeasures. It performed a

critical evaluation of all commonly used design equations, material specifications and test methods, and construction practices. Based on the critical evaluation, the research team determined equations, specifications, test methods, and construction practices to be developed in Phase II.

1.3.4 Task 4 – Interim Report

The research team submitted an interim report documenting the information in Tasks 1, 2, and 3. Specifically, the interim report included the following:

- Summary of the findings of the literature review
- Results of the survey
- Results of the critical evaluation, including a detailed report of the design equations, material specifications and test methods, and construction practices evaluated
- An updated work plan as a separate appendix on how the research team intended to complete Phase II. Specifically, equations, specifications, test methods, and construction practices to be developed in Phase II were identified.

The research team then met with the NCHRP Project 24-23 panel to discuss the interim report and revised work plan.

1.3.5 Task 5 – Develop Design Guidelines

Based on the panel's guidance during the interim meeting, the research team determined or developed the following:

- Design equations and methodologies to use for each different design application
- Guidance on proper determination of design variables (e.g., velocity multiplication factors, design flood frequency and freeboard, flow velocities, characteristics, and flow depths)
- Sensitivity of equations and methods being evaluated to ranges in flow parameters, flow depths, flow velocities, side slopes, and other important variables
- Limitations on using the recommended equations and methods
- Filter requirements, design methods, and types
- Vertical and lateral extent and configuration of filter and riprap
- Durability and susceptibility to ice and debris damage.

1.3.6 Task 6 – Material and Testing Specifications

The research team developed material specifications for riprap and test methods for riprap gradation and material quality and contacted riprap producers (e.g., quarry operators and national associations) to evaluate the feasibility of producing recommended gradations.

1.3.7 Task 7 – Develop Construction Guidelines

The research team developed construction guidelines and specifications with consideration of the practicality of constructing riprap using the recommended procedures (e.g., at piers and abutments, under wet or dry conditions, on side slopes, in deep water, or at sites requiring dewatering). The research team contacted experienced construction contractors, state and federal construction agency personnel, and national and state construction associations to evaluate the practicality and constructability of the guidelines and specifications.

1.3.8 Task 8 – Submit Final Report

The research team submitted a final report documenting the entire research effort. The design guidelines; recommended material specifications and test methods; recommended construction specifications; and construction, inspection, and quality control guidelines are included as stand-alone appendixes.

1.4 Special Requirements

In addition to the eight specific tasks required for project completion, the research problem statement included the following special requirements:

- This research was limited to a review, synthesis, and critique of the current state of knowledge, with the results used in the development of new design methods, material tests, specifications, and guidelines. The research was accomplished without conducting new laboratory or field studies.
- This research did not include use of riprap for shoreline protection, roadside ditches, roadside drainage, or culvert outlets.
- The literature review, synthesis, and critical evaluation in Tasks 1, 2, and 3 included pertinent computer software used for riprap design.
- This final report includes sample problems illustrating application of each of the design methods developed in Task 5. The design and construction specifications are in a format suitable for adoption and use by AASHTO and state DOTs. Chapter 4 includes recommendations for future riprap research.

- The report "California Bank and Shore Rock Slope Protection Design" (Racin et al., 2000), Caltrans Study No. F90TL03, was obtained and reviewed. The annotated bibliography in Chapter 8 of that report was thoroughly reviewed during Task 1 and cited source documents on various riprap design methods were investigated.
- Progress on the following initiatives sponsored by NCHRP was monitored and included where appropriate:
  - NCHRP Project 24-18A, "Countermeasures to Protect Bridge Abutments from Scour"
  - NCHRP Project 24-07(2), "Countermeasures to Protect Bridge Piers from Scour"

CHAPTER 2

Findings

2.1 Review of Technical Literature

The review of the technical literature included domestic and foreign sources and an assessment of the adequacy and extent of existing information used to design, specify, and construct riprap. Specifically, riprap design equations and techniques for determining (1) the size of stone for riprap design, (2) the gradation requirements, (3) the material quality tests, and (4) the thickness requirements were reviewed.

During the literature search and document compilation process, a relational database management system was developed to collate, track, and maintain information relevant to the literature resources. Microsoft® Access was employed as the database software because of its wide distribution and availability and its compatibility with Microsoft® Windows operating system.

The primary function of a database is to organize lists of information and then to provide alternative means of viewing, sorting, and retrieving that information. The database is useful as a paper list, but also has information storage and selective retrieval functions. Viewing features in Access provide an efficient means of organizing data based on any characteristic chosen by the program user, and reports can be generated to display selected information as required.

More than 300 literature sources were identified as having relevant background information for NCHRP Project 24-23. All documents acquired were categorized using the following seven criteria:

- Original data provided
- Design methodology developed or presented
- Quality assurance/quality control (QA/QC) tests identified
- Maintenance guidelines presented
- Material specifications provided
- Construction/installation guidance provided
- Inspection guidance provided

The database thus provides not only a citation list, but an annotation of the information contained in each document. This categorization allows the user to rapidly screen the citation list into subsets that contain only the information of interest to the user. The citations can also be selected and sorted by year of publication, author, or title. The database was screened using the seven criteria previously mentioned. The distribution of documents that provide information in one or more of the categories follows:

Original Data:	21%
Design Methodology:	45%
QA/QC Tests:	5%
Maintenance Guidelines:	1%
Material Specifications:	11%
Installation Guidance:	13%
Inspection Guidance:	4%

References from the database cited in this report are included in Chapter 5 and the complete database bibliography is provided in Appendix A. Findings from the technical literature review are combined with the results of a survey into a synthesis of the current state of practice for riprap design, specifications, and quality control in Section 2.4.

2.2 Survey for Current State of Practice

Federal agencies, state DOTs, and other agencies were surveyed to determine practices used to design, specify, and construct riprap. The survey collected data on riprap design equations, classification systems, specifications, standard plans, and construction guidance. Based on the survey results, selected agencies were interviewed to acquire detailed information.

The survey was sent to a mailing list, which included all state DOT hydraulic engineers and FHWA Resource Center and Division/District engineers. Surveys were also sent to the U.S. Army Corps of Engineers (USACE), Bureau of Reclamation, Natural Resources Conservation Service (NRCS), Office of Surface Mining (OSM), and several consulting firms. When documenting DOT practices, an attempt was made to determine how state DOTs address site conditions that are outside of the intended limits of the design equations (e.g., side slope angle, bed slope angle, velocity, discharge, and shear stress).

Of the 33 completed questionnaires received, 24 states are represented with 11 states located west of the Mississippi River and 13 located east of the Mississippi. Most of the individuals responding were from state DOTs. Several federal agencies were represented and one consulting firm responded. Of the 37 individuals who participated in completing the surveys (some surveys were completed by several people), seven identified themselves as licensed engineers.

A database of the responses was created using the Microsoft® Access relational database management system (see Section 2.1). A general summary of the responses given that pertain to design guidelines, material and testing specifications, construction and installation guidelines, inspection and quality control, and specific applications (revetment, pier, abutment, and countermeasures) is provided in Appendix B. Findings from the survey are combined with results of the literature review into the synthesis of current practice in Section 2.4.

2.3 Riprap – An Integrated System

Before summarizing the state of current practice for specific riprap applications in Section 2.4, it is appropriate to consider a generalized overview of riprap armoring systems. A properly designed, installed, and maintained riprap system, as an integrated whole, has a functionality that is greater than the sum of its parts, i.e., successful performance depends on the system responding to hydraulic and environmental stresses as an integrated whole throughout its service life. This point of view provides context for the findings of this study and the recommendations that follow.

2.3.1 General

Erosion is a natural process resulting from water attacking stream and river banks. The erosion dislodges and removes material from one area; water transports the material and deposits it at some area downstream. Local scour can occur at structures located in a stream. Man-made changes to a river where streambank or bed soils have been disturbed or vegetation has been removed can induce or cause erosion or scour. Properly designed erosion control works, such as riprap, can reduce or prevent natural and induced erosion and scour.

Riprap is defined as "broken stone or boulders placed compactly or irregularly on dams, levees, dikes, etc., for protection of earth surfaces against the action of waves or currents . . ." (ASCE, 1962). Riprap may have been used to protect structures in rivers for several centuries. Leonardo da Vinci probably was the first to refer to rock protection in his description of vortices, particularly vortices behind a pier in a river. In a paper by Fasso (1987), included in the International Association for Hydraulic Research (IAHR) Jubilee Volume entitled *Hydraulics and Hydraulic Research, A Historical Review*, vortices behind a pier in a river are depicted in sketches by Leonardo, and in his notes Leonardo suggested protecting the bed by means of "an apron of stones well linked together with swallow tailed joints."

The annuals of the Institute of Civil Engineers, in London, contain references throughout the 1800s and early 1900s to bridge construction and repairs in India resulting from local scour. The rivers in India caused difficulties because of the "great depth of fine sand found in most Indian rivers" (Stoney, 1898). When a scour hole at a pier was identified, large rock was customarily dropped into the areas surrounding the pier to protect against future scour. Often rock was dumped from the bridge on the upstream side of the pier during the flood to reduce the extent of scour.

Because riprap is a natural material composed of stone or boulders and is readily available in many areas, it has been used extensively in erosion protection works. In some areas, riprap is produced by quarrying hard, durable rock. In other areas, riprap is collected from talus or by excavating large river cobbles from alluvial deposits. 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 riprap can provide long-term protection if it is inspected and maintained on a periodic basis as well as after flood events.

As a natural material, riprap is non-polluting, and, during normal river flow conditions, it can provide habitat, hiding, and resting areas for fish and aquatic invertebrates. When riprap is specified for a project, a source to obtain riprap is sought as near as possible to the construction site. This proximity is mainly an economic factor, but also is an attempt to obtain rock of similar color and texture to local rock to provide a more natural appearance along the river bank lines. However, when riprap is placed above the low waterline to provide flood protection, the ribbon of rock on the banks will be evident. To provide a more vegetated appearance, top soil can be placed over the riprap above the high water line and seeded with grasses and shrubs. This cover material likely will

be eroded during a flood event and replacement of the top soil and vegetation would be necessary to restore a vegetated appearance.

Design of a riprap scour control system requires knowledge of river bed, bank, and foundation material; flow conditions including velocity, depth, and orientation; riprap characteristics of size, density, durability, and availability; location, orientation, and dimensions of piers, abutments, guide banks, and spurs; and the type of interface material between riprap and underlying foundation, which may be geotextile fabric or a filter of sand and/or gravel.

Riprap is also used as an emergency measure to reduce or stop scour. Maintenance personnel often stockpile riprap to use to protect streambanks, bridge piers, and abutments and to prevent incising channels from continuing to degrade or to head cut upstream. The riprap is generally end dumped from a dump-truck. These measures generally are initiated without proper design and without bedding because of emergency conditions and may not provide long-term protection.

2.3.2 Life-Cycle Approach

Designing riprap as an integrated system requires a life-cycle approach to the design, production, transport, installation, inspection, and maintenance of the system. The challenge for this project is to develop a unified set of guidelines, specifications, and procedures that can be accepted by the state DOTs. The recently adopted riprap standards for the European Union provide an example of a unified standard for riprap that transcends geographic and institutional boundaries (CEN, 2002).

The efficacy of rock riprap depends on quality of the rock; weight, shape, or size of individual rocks; slope of the embankment or channel; thickness of the riprap layer; and stability of the bedding or filter on which the riprap is placed. Because of the size and weight of riprap, transport and placement is generally by mechanical means. Failure of riprap often is the result of poor construction techniques and poor quality control relating to weight or size. Quality control begins at the quarry. Inspection must ensure correct weight or size, density, and gradation. Transportation can be by truck, train, or barge where segregation can occur. Stockpiles at the job site should be checked for segregation and adjustments made to ensure that proper gradation is maintained.

Thus, uniform specifications and/or guidelines for riprap must be developed considering production capabilities and control at the quarry as well as at the job site and during transportation, handling, moving, and placement. Guidelines and procedures for onsite inspection and monitoring riprap also should be developed providing reasonable limits and tolerances for materials and workmanship that can be expected as construction industry standards. Constructability issues must be considered so as to accommodate site constraints, permit conditions, and the like. The physical characteristics of the system need to be considered for placement under water versus in the dry and for installations below versus above the (unscoured) bed level. Additionally, the placement of ancillary system components, including filter and/or bedding requirements, must be addressed for various riprap applications. Practical matters of installation often dictate that suitable options be developed for these components, particularly when applications must address placement under water or in flowing water.

A life-cycle concept, as applied to an erosion or scour countermeasure such as riprap, would incorporate a host of factors into a framework for decision making that considers initial design, construction, and long-term maintenance. These factors could include engineering judgment applied to design alternatives, materials availability and cost, installation equipment and practices, and maintenance assumptions.

Estimating life-cycle costs for a riprap project would require 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

Obviously, quantity and unit cost of alternative materials will vary depending on the specific project conditions, as well as local and regional factors. The following issues should be considered when developing a life-cycle cost estimate:

- Availability of materials of the required size and weight
- Haul distance
- Site access
- Equipment requirements
- Construction underwater versus placement in the dry
- Environmental and water quality issues and permitting requirements
- Habitat mitigation for threatened and endangered species
- Traffic control during construction and/or maintenance activities
- Local labor rates
- Construction using DOT resources versus outside contract
- Design life of the installation
- Anticipated frequency and extent of periodic maintenance and repair activities

While the intent of this project is not to develop a framework for estimating the life-cycle cost of a riprap installation, the life-cycle concept emphasizes the need to consider riprap

as an integrated system where the performance of all system components is considered throughout the design life of the project. For a discussion of life-cycle costs related to bridge scour countermeasure selection, see Lagasse et al. (2006).

2.3.3 Risk and Failure

The risk of failure should be considered when evaluating the performance of riprap as an integrated system to prevent erosion or scour. There are a number of methods available for assessing the causes and effects of a wide variety of factors in uncertain, complex systems and for making decisions in the light of uncertainty. One approach, failure modes and effects analysis, is a qualitative procedure to systematically identify potential component failure modes and assess the effects of associated failures on the operational status of the system (Johnson and Niezgoda, 2004).

Formulation of a failure modes and effects analysis begins with identification of the system and all of its components. Next, possible failure modes are defined using documented case studies, laboratory experimentation, field experience, and expert opinion. Once the failure modes are identified for each component of the system, their effects on the system and other system components, consequences, risk of occurrence, methods of detection, and compensating factors (e.g., possible corrective actions) are listed. By using numeric ratings for consequences and occurrences, in addition to a rating for detectability (i.e., the likelihood that the failure mode will be observed), failure modes can be prioritized to focus a greater level of effort on higher priority failures (Johnson and Niezgoda, 2004).

Applying a failure modes and effects analysis to a riprap installation emphasizes the integrated nature of the riprap system and provides a method to identify system failure as a basis for evaluating riprap performance. In developing a risk-based method for selecting bridge scour countermeasures, Johnson and Niezgoda (2004) developed a failure modes and effects analysis for riprap similar to Table 2.1.

The most common method of prioritizing failure modes is through implementation of risk priority numbers. These numbers are the product of the occurrence, consequence, and detectability ratings assigned to a given failure mode and can be used to suggest the appropriate nature and extent of corrective actions (Johnson and Niezgoda, 2004). For purposes of this study, applying the concepts of a failure modes and effects analysis to riprap (Table 2.1) serves to underscore the relationships among system components and provides an initial definition of system failure. Riprap failure mechanisms for the range of applications considered in this study are discussed further in Section 3.7.

2.3.4 Service Life and Safety

In 1949 the State of California Department of Public Works (DPW), Division of Highways, initiated a 10-year critical review of bank protection in California highway practice. This review resulted in the 1960 publication "Bank and Shore Protection in California Highway Practice" (State of California DPW, 1960) that addresses concepts of service life and safety in riprap revetment design and maintenance that provide insight on life-cycle considerations for a riprap installation.

The report notes that an earlier DPW policy of adding bank protection after an erosion hazard was evident as a result of flood damage had largely been supplanted by a policy of "foresighted inclusion of high-standard protective

Table 2.1. Failure modes and effects analysis for riprap revetment.

Failure Modes	Effects on Other Components	Effects on Whole System	Detection Methods	Compensating Provisions
Translational slide or slump (slope failure)	Disruption of armor layer	Catastrophic failure	Mound of rock at bank toe; unprotected upper bank	Reduce bank slope; use more angular or smaller rock; use granular filter rather than geotextile fabric
Particle erosion (rock undersized)	Loss of armor layer, erosion of filter	Progressive failure	Rock moved downstream from original location, exposure of filter	Increase rock size; modify rock gradation
Piping or erosion beneath armor (improper filter)	Displacement of armor layer	Progressive failure	Scalloping of upper bank; bank cutting; voids beneath and between rocks	Use appropriate granular or geotextile filter
Loss of toe or key (under designed)	Displacement or disruption of armor layer	Catastrophic failure	Slumping of rock, unprotected upper bank	Increase size, thickness, depth, or extent of toe or key

works in new projects." This new policy shifted the burden for system performance from maintenance to design and construction. As a maintenance operation, bank protection was guided by flood damage and concentrated on points of maximum exposure. The shift in responsibility for system performance "induced centralization of design, with a trend toward standardization. . . ." However, this trend also resulted in less attention being paid to light-duty, short-lived, but less-expensive facilities as protection against temporary hazards, and optimum incorporation of local materials into durable protective works. The report also notes that "protective works destroyed in floods usually have repaid their cost in preventing or minimizing damage to highways" (State of California DPW, 1960).

Overdesign observed in many locations was ascribed to transplant of a successful design to a less hazardous situation. In other locations, design appeared to ignore the principle of **expendability**, which means that bank protection may be damaged while serving its primary purpose most economically. Protective works were usually overlooked during periodic inspections for maintenance of highway facilities, so that deterioration was evident only when failure occurred during flood (State of California DPW, 1960).

This review of bank protection practice includes the following recommendations:

- Design should be governed by the principle of **expendability**; that is, the primary objective is security of the highway, not security of the protective structure. Cheap, replaceable facilities may be more economical than expensive permanent structures.
- Design should be governed by the importance of **dependability**. An expensive structure is warranted mainly for highways carrying heavy traffic or for which no detour is available.
- Design should conform to the principle of **longevity**. Short-lived structures or materials may be economical for resistance to temporary hazards.
- Limits of protection, both horizontal and vertical, should be designed with prudence and judgment. The bottom limit should be secure against scour. The top limit should not arbitrarily be at high-water mark, but above it if overtopping would cause damage and below it if floods move slowly along the upper bank. The end limits should reach and conform to durable natural features.
- Maintenance should distinguish between temporary or expendable facilities that need not be serviced and permanent or indispensable facilities that must be kept intact.

While implementing a life-cycle approach to riprap design embodies more than the "principles" of expendability, dependability, and longevity, these concepts provide a philosophical basis for considering the service life of a riprap installation. Design considerations for the horizontal and vertical limits of protection also introduce the concept of a hybrid design, which will be considered further in this study (see Section 3.8).

When selecting a service-life criterion for various types of bank protection measures for transportation facilities, safety must be a primary consideration (Racin personal communication, 2004). To assume that bank protection is installed to protect a facility (e.g., bridge, roadway embankment) overlooks the mission and design goals of the highway agency. For DOTs, safety of the traveling public is the first priority when setting service-life standards for riprap protection. Concurrent goals are protection of public and private property, protection of fish and wildlife resources, and enhancement of environmental attributes. A riprap system does not protect a facility, but rather the lives of the public who use that facility.

Thus, service life for a riprap installation should be based on the importance of the facility to the public, that is, the risk of losing the facility and how that loss may directly or indirectly affect the traveling public, as well as the difficulty and cost of future repair or replacement. The conditions that constitute an "end of service life" for a riprap installation are largely dependent on the confidence one has that a degraded condition will be detected and corrected in a timely manner (e.g., during a post-flood inspection). Generally, for facilities that are rarely checked or inspected a very conservative (i.e., shorter) service life would be appropriate, while a less conservative standard could be used for facilities that are inspected regularly (Racin personal communication, 2004).

Service life for a riprap installation can be considered a measure of the durability of the total, integrated bank, pier, abutment or countermeasure protection system. The durability of system components and how they function in the context of the overall design will determine the service life of an installation. The response of a riprap system over time to typical stresses such as flow conditions (floods and droughts) or normal deterioration of system components must also be considered. Response to less typical (but plausible) stresses such as fire, vandalism, seismic activity, or accidents may also affect service life. Finally, there may be opportunities for maintenance during the life cycle of a riprap installation and, where such work does not constitute total reconstruction or replacement, maintenance should not be considered as the end of service life for the riprap system. In fact, a life-cycle approach to maintenance may extend the service life of a riprap installation and reduce the total cost over the life of the project.

2.4 Synthesis of Current State of Practice

2.4.1 Revetment Riprap

Design Guidelines

In the following subsections of Section 2, various riprap equations are presented to show their general form and evolution. Because these equations are not meant to be used for computations without reference to the source documents, not all variables were defined nor was a consistent set of symbols used.

Sizing of Revetment Riprap. The use of rock for river training and for closures of breaches in dikes has been documented since about 2000 BC (Fasso, 1987). Rock along with bamboo and fascine bales was used to close breaches in the dikes along the Yellow River. The rivers of India, such as the Ganges, Jhelum, and the Hooghly, eroded the banks and scoured around bridge piers (Stoney, 1898). If a scour hole was identified at a pier, large rocks were customarily dumped into the area to protect against future scour. Rock protection is currently referred to as riprap and is the most common form of bank protection in use today. It is assumed that rock size and placement for these early riprap designs were based upon the experience of the engineer.

Riprap sizing methods evolved from non-scouring, non-silting velocity concepts used for early canal designs that were based upon experience and observation of many canals in different bed and bank materials. The concept of maximum permissible velocities, that is a velocity less than critical velocity causing shear that will erode the channel, was adapted and widely accepted to design canals after Fortier and Scobey published a table of values for permissible velocities in 1926 (Fortier and Scobey, 1926). The paper includes a table attributed to Etcheverry (1916) that included "maximum mean velocities safe against erosion" for coarse gravels (6 to 8 ft/s), conglomerates (8 to 10 ft/s), hard rock (10 to 15 ft/s), and concrete (15 to 20 ft/s). In a Russian article (translated by USACE [Isbash 1935] and later published [Isbash 1936]) relating to river closures by dumping rocks, Isbash presents an equation for mean velocity against stone, which became popular for most riprap design because of lack of other significant data.

$$V = C\left[2g(S-1)\right]^{1/2}(d_{50})^{1/2} \qquad (2.1)$$

where

V = Mean velocity against stone, ft/s or m/s
C = Isbash constant (0.86 or 1.20 – see discussion)
g = Acceleration of gravity, ft/s^2 or m/s^2
S = Specific gravity of stone
d_{50} = Median diameter of spherical stone, ft or m

The lower value for the Isbash constant represents the flow velocity at which loose surface stones first begin to roll. The higher value represents the flow velocity at which stones protected by adjacent particles begin to move and roll until they find another "seat."

Prior to 1920, there was little need or application of bank protection in California, but floods in 1921–22 pointed to the need for bank protection in many locations and particularly along roads built to higher standards. After the flood on February 16, 1927, caused extensive damage to highways in southern California, an investigation of bank-protection devices was undertaken by E. Withycombe. Withycombe's reports record his observations on devices in use at that time, including riprap, and his conclusions established design guidelines carried forward from that time (State of California DPW, 1960). Floods in 1937 in northern California and 1938 in southern California were severe tests for all installations and a committee from the Highway Division appraised the structures' performances. After 1938, the trend of highway practice favored stone structures. In 1949, California Division of Highways appointed a Joint Bank Protection Committee to study "primarily the special treatment of banks of streams, lakes or tidewater and secondarily the treatment of highway embankments to prevent erosion by surface waters. . . ." The outcome of this study was a compilation of data and reports that became the first edition of the California Bank and Shore Protection Manual published in November 1960. An equation and nomograph were developed for slopes no steeper than 1.5H:1V:

$$W = \frac{0.00002V^6 Sg_r \csc^3(\rho-\alpha)}{(Sg_r-1)^3} \qquad (2.2)$$

where

W = Minimum weight of outside stone for no damage, lb
V = Stream velocity to which bank is exposed, ft/s
= 4/3 the average stream velocity for impinging velocities (on outside of bends in line with the central thread), ft/s
= 2/3 the average velocity for tangent (parallel) velocity, ft/s
Sg_r = Specific gravity of the stones
ρ = 70° for randomly placed rock
α = Embankment face slope

The manual states, "Specifications must allow an acceptable nonuniformity for economic use of local materials, and require enforceable and practical rules for placement." Specifications were outlined to test rocks including stone size, specific gravity, soundness, water absorption, and abrasion. The rock protection was to be placed in accordance with one of two

placement methods (A or B). Both methods required a footing trench excavated along the toe of the slope. In Method A placement, no rocks could be dumped. The larger rocks were placed in the toe trench and rocks on the slope were oriented with the longitudinal axis normal to the alignment of the embankment face and arranged to have a 3-point bearing on the underlying rocks. Method B allowed placement of rock by dumping and spreading by bulldozer or similar equipment.

Design of riprap for revetment was based upon permissible or mean velocity because of ease of calculating the velocity. The development of a riprap design based on tractive force was considered to be better than permissible velocities but was slow in coming because of lack of data to evaluate the tractive force at specific points on the bed and bank. In the 1950s, the Bureau of Reclamation began a concerted effort to design canals using tractive force. A Reclamation progress report (Lane, 1952) states, "One advantage of the use of tractive force analysis rather than limiting velocity approach for the design of large canals is that it indicates why higher velocities are safer in large canals than in smaller ones." Theoretical approaches for determining critical tractive forces on channel side slopes for canal designs were being developed (Carter et al., 1953) and designs for stable channels evolved (Lane, 1955; Terrell and Borland, 1958). Shear stress distributions for curved trapezoidal channels (Ippen and Drinker, 1962) provided a picture of the close link to velocity distribution. Ippen and Drinker's tests show that the regions of high shear are located first at the inside of the curve and then move outward downstream from the bend and up on the outside embankment slope. Channel shape appears to play only a small part in shear stress distribution around a bend. As a result of many experimental runs, including Ippen and Drinker's points as well as others, and some tests in natural channels, the ratio of maximum to mean shear stress can be correlated with the ratio of channel width to centerline radius of curvature and the angle of the bend (Montes, 1988). Maximum experimental shear stress ratios range from 2.4 to 2.8.

Stevens (1968) developed a stability factor approach for riprap at culvert outlets based on shear stress that has been adapted to revetments. He considered the forces acting on a particle in the plane of the side slope. The equations given below are for horizontal or parallel flow on an embankment. The expression for the stability factor, SF, for horizontal flow on a side slope with an angle of θ and using rock with an angle of repose of ϕ is

$$S.F. = 0.5 S_m \left\{ \sqrt{\zeta^2 + 4} - \zeta \right\} \tag{2.3}$$

where

$$\zeta = S_m \eta \sec\theta \tag{2.4}$$

$$S_m = \frac{\tan\phi}{\tan\theta} \tag{2.5}$$

Solving for the stability number, η, in terms of the stability factor

$$\eta = \frac{S_m^2 - S.F.^2}{S.F. S_m^2} \cos\theta \tag{2.6}$$

Given the specific weight of water, γ, and rock information such as the specific gravity, Sg, and angle of repose, and knowing the angle of the embankment slope, a stability factor can be assumed and the stability number, η, can be calculated from Equation 2.6. If the shear stress on the side slope, τ_0, is known, the riprap size, d_m, can be obtained from

$$d_m = \frac{21\tau_0}{(Sg-1)\gamma\eta} \tag{2.7}$$

where d_m is in feet, τ_0 is in lb/ft^2, and γ is in lb/ft^3.

As presented in Hydraulic Design Series No. 6 (HDS 6) (Richardson et al., 2001) and Simons and Senturk (1992), the approach requires an iterative solution because the shear stress, τ_0, is a function of the relative roughness (riprap size divided by flow depth).

The revised HEC-11 (Brown and Clyde, 1989) revetment riprap equation is derived based on the Shields equation for incipient motion, average shear stress ($\tau_0 = \gamma R S_f$), the Manning equation to compute friction slope, and the Strickler equation to compute Manning n as a function of particle size. Additional factors are included for bank angle, riprap specific gravity, and desired stability factor. The equation in English units is

$$d_{50} = 0.001 C_{sg} C_{sf} \frac{V_a^3}{d_{avg}^{0.5} K_1^{1.5}} \tag{2.8}$$

where

d_{50} = Median diameter of stone, ft
V_a = Average channel velocity, ft/s
d_{avg} = Average channel depth, ft
$C_{sg} = 2.12/(Sg-1)^{1.5}$
Sg = Riprap specific gravity
C_{sf} = (Stability factor/1.2)$^{1.5}$
$K_1 = [1 - \sin^2\theta/\sin^2\phi]^{0.5}$
θ = Bank angle (degrees)
ϕ = Riprap angle of repose (degrees)

HEC-11 (revised) incorrectly indicates that the equation is valid for English or metric units but the derivation is clearly for English units. For metric units, the constant in Equation 2.8 needs to be 0.00594 ($0.001/0.3048^{1.5}$).

Blodgett and McConaughy (1986) critique several procedures used to design riprap in the 1980s. They evaluate four design procedures to compare with riprap that failed at five sites where they collected data. The four design procedures are HEC-11 (Searcy, 1967), HEC-15 (Normann, 1975), Bank

and Shore Protection (State of California DPW, 1960), and EM 1601 (USACE, 1970). The HEC-11 and EM 1601 equations have been changed significantly since the evaluation. Blodgett and McConaughy also make a graphical comparison of six procedures based on permissible velocities and find a wide variation in stone size for a given average velocity. They compare six procedures that relate stone size to shear stress and find that critical shear stress gives a smaller variation in stone size, although the results are somewhat deceiving because of the rock sizes considered. Of the six equations, two included only rock sizes less than 0.45 ft (0.14 m), two others included rock sizes less than about 1.0 ft (0.31 m), one projected to 3 ft (0.91m), and one had a constant shear stress of 0.25 psf (12 N/m^2) for all rocks to 4.0 ft (1.23 m).

During the 1970s and 1980s, USACE conducted research to improve riprap design. Tests were conducted at Colorado State University and at Waterways Experiment Station in an effort to develop an improved riprap design procedure based on velocity rather than shear stress, because most designers prefer velocity-based methods. Because shear stress is difficult to measure and little information regarding shear stress on riprap was available, USACE initiated a near–prototype riprap test program conducted by S.T. Maynord. Similar to Ippen and Drinker, he found that downstream from the bend, maximum shear stress occurs not at the toe, but on the slope between 0.25 and 0.50 the distance above the toe and water surface (Maynord, 1990). Data and curves are given showing riprap size and shear stress relative to values of riprap size and stress at the toe of the slope as a function of distance on the slope between toe and water surface. Maynord's velocity profile at the exit of the bend indicates the depth-averaged velocity for about 30% of the distance up the slope was equal to the velocity at the toe.

An initial equation (Maynord et al., 1989)—based on velocity and using dimensional analysis for finding riprap rock size, d_{30}, instead of the commonly used d_{50}—was modified to include coefficients to account for stability, velocity distribution, blanket thickness, and side slope correction. Equation 2.9 is the final equation. Values of coefficients are given graphically in Appendix B of EM 1601 (USACE, 1991):

$$\frac{d_{30}}{y} = S_f C_S C_V C_T \left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w} \right)^{0.5} \frac{V_{ss}}{\sqrt{K_1 g y}} \right]^{2.5} \qquad (2.9)$$

where

d_{30} = Particle size for which 30% is finer by weight, ft or m
y = Depth of flow above particle, ft or m
S_f = Safety factor
γ_w = Specific weight of water, lb/ft^3 or kg/m^3
γ_s = Specific weight of particle, lb/ft^3 or kg/m^3
V_{ss} = Characteristic velocity, depth-averaged velocity at point 20% upslope from toe $V_{ss}=V_{avg}[1.74-0.52\text{Log}(Rc/W)]$ for natural channels (Plate B-33 of EM 1601)
g = Acceleration due to gravity, ft/s^2 or m/s^2
C_S = Stability coefficient
= 0.3 for angular rock; 0.375 for rounded rock [for blanket thickness = $1d_{100}$(max) or $1.5d_{50}$(max) whichever is greater, and d_{85}/d_{15} = 1.7 to 5.2]
C_V = Velocity distribution coefficient (Plate B-40 of EM 1601)
= 1.0 straight channels, inside of bends
= $1.283-0.2\log(Rc/W)$ for outside bends (1 for $Rc/W > 26$)
= 1.25 downstream from concrete channels
= 1.25 at end of dikes
Rc = Centerline radius of bend, ft or m
W = Water-surface width at upstream end of bend, ft or m
K_1 = Side slope correction factor (use Plate B-39 of EM 1601)
C_T = Blanket thickness coefficient, function of d_{85}/d_{15} (Plate B-40 of EM 1601)

Guidance is provided in EM 1601, Chapter 3, to estimate the coefficients based on flow conditions and natural stream and channel conditions. There are two adjustments for bend curvature (Rc/W) in Equation 2.9. The first adjustment is the C_V factor that relates to secondary currents causing a velocity component down the bank. The second adjustment is to the velocity, V_{ss}, for revetments on sloping banks and accounts for higher longitudinal velocities on the outside bank.

River and Channel Revetments (Escarameia, 1998) provides an introduction to hydraulics of revetments, outlines a design process, and discusses a number of revetment protection methods including riprap. Escarameia presents three riprap design equations intended to reflect the current use in the United Kingdom, the Netherlands, and the United States. The U.S. equation is Maynord's equation included in EM 1601. Escarameia recommends applying the vertical velocity factor (C_V from EM 1601) as an adjustment to each of the three equations she presents.

The Escarameia and May (1992) equation for sizing revetment riprap is

$$d_{n50} = C \frac{U_b^2}{2g(S-1)} \qquad (2.10)$$

where

d_{n50} = Characteristic size of stone, size of equivalent cube, ft or m

C = Coefficient that accounts for turbulence intensity, TI; for riprap
C = 12.3TI – 0.20
TI = Ratio of root mean square velocity fluctuation over mean velocity measured at a point 10% of flow depth above bed and varies from 0.12 to 0.60 for different structures
U_b = Mean velocity measured at a point 10% of flow depth above bed, ft/s or m/s
g = Acceleration due to gravity, ft/s^2 or m/s^2
S = Specific gravity of stone

In most cases of design, the TI is not known, is difficult to obtain, and must be assumed. For TI less than 0.5, a relationship between U_b and U_d, depth average velocity, was obtained from field measurements and can be used if values for U_b are not available:

$$U_b = (-1.48TI + 1.04)U_d \qquad (2.11)$$

A provisional equation that has not been verified for TI greater than 0.50 is

$$U_b = (-1.48TI + 1.36)U_d \qquad (2.12)$$

For straight channels, U_d can be substituted for U_b and values of C are 1.0 for continuous revetments and 1.25 for edges.

Pilarczyk's (1990) riprap equation is

$$d_{n50} = \frac{\Phi}{\Delta}\frac{0.035}{\Psi_{cr}}K_T K_h K_S^{-1}\frac{U_d^2}{2g} \qquad (2.13)$$

where
d_{n50} = Median diameter of stone, ft or m
Φ = 0.75 for continuous protection, and 1.0 – 1.5 at edges and transitions, and 3.0 for jet impact or screw race velocity
Δ = S – 1
Ψ_{cr} = 0.035 for rock riprap
K_T = 1.0 for normal river turbulence, 1.5 – 2.0 for high turbulence (e.g., downstream of stilling basins, local disturbances, sharp outer bends)
K_h = $(d_{n50} / y)^{0.2}$ where y is depth of flow above toe of bank
K_s = Product of a side slope term and a longitudinal slope term
U_d = Depth average velocity, ft/s or m/s
g = Acceleration due to gravity, ft/s^2 or m/s^2

The K_h factor presented above is for a non-developed velocity profile and results in a larger riprap size than for uniform flow.

The California Department of Transportation (Caltrans) reevaluated the California bank and shore layered rock slope protection (RSP) procedure that was introduced in the original 1960 Bank and Shore Protection (BSP) manual (State of California DPW). The resulting Caltrans report, "California Bank and Shore Rock Slope Protection Design" (CABS) (Racin et al., 2000), focuses on RSP and emphasizes the California bank and shore layered RSP design method. Sixty-five field sites were evaluated in five states and sixty of the field sites are tabulated giving location, RSP design method, description of site, when built, repair history, and present status. The RSP design equation given in the 2000 manual is the same equation as in the 1960 BSP manual. Equation 2.14 gives the minimum standard rock weight for the outer layer of the layered RSP, in standard English units:

$$W = \frac{0.00002V^6\ Sg_r}{(Sg_r - 1)^3 \sin^3(r - a)} \qquad (2.14)$$

The definition of r equals 70° (for randomly placed rubble, a constant). Neither the 1960 manual nor the 2000 manual discusses why the value of r is 70°. However, Blodgett and McConaughy (1986) refer to notes assembled by R.M. Carmany of Caltrans that discuss laboratory experiments conducted by the University of California to determine the minimum force to dislodge a stone from the bank. The University of California constructed a model streambank with small stones arranged as riprap and underlying stones cemented into a plaster base. The side slope was increased until the first outer stone was displaced. A maximum angle of 65° to 70° was attained before the first stone fell out. It is assumed that the value of r equals 70° is based on these tests.

Sensitivity Analysis. In summary, numerous equations are available to design riprap for embankment protection. The equations discussed in this section appear to be more widely used for design than other equations found in the literature. Permissible velocity and/or critical tractive force are the approaches commonly used for sizing riprap. Many engineers feel that a tractive force approach to determining riprap size is preferable; but, because of the difficulty of determining the shear stress at the bed or on the slope of a channel, some form of velocity, either mean or depth-averaged, is more often used to determine riprap size. A sensitivity analysis of the seven most commonly used revetment riprap design equations is presented in Section 3.2.1.

Riprap Filter Requirements. 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 2.1 provides schematic illustrations of the three most typical types of riprap filter configurations.

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.

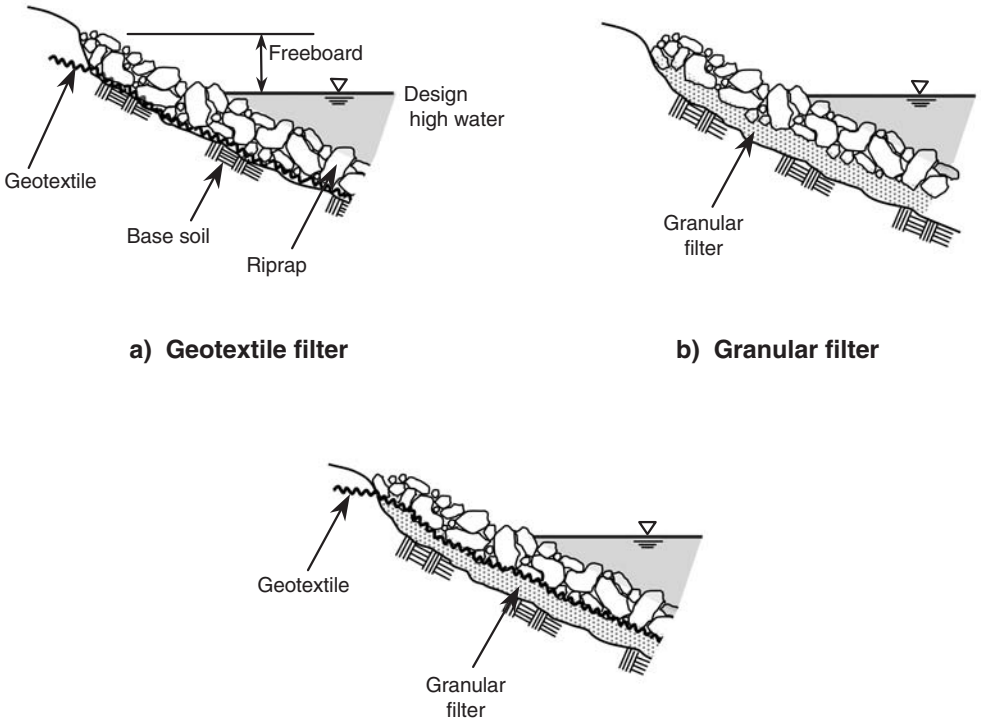
As an example, consider the process of "piping." Piping is basically the washing away of very fine particles, resulting in greater void space in the underlying soil structure. Piping is more likely to occur in non-cohesive/unstable soils that are in contact with a filter that has large openings. The 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 3.2.2.

Material and Testing Specifications

Rock for erosion control should be hard, dense, and durable and should be composed of a suitable range of sizes to ensure stability under the design hydraulic loading. The specification of materials to achieve these characteristics, and the associated testing requirements to ensure compliance with the specifications, are the responsibility of the design engineer.

As applied to riprap (and ancillary components associated with riprap installations), **material specifications** provide a



a) Geotextile filter

b) Granular filter

c) Granular transition layer with geotextile (composite filter)

Figure 2.1. Channel cross sections showing common riprap/filter configurations.

written, verbal description of the quality and characteristics of the armorstone, granular filter material, and/or geotextile that are required for a specific application.

In contrast to material specifications, **testing specifications** establish standardized procedures by which the riprap material can be periodically checked and documented to ensure that the quality of the material placed at the job site meets or exceeds the minimum requirements.

Reference specifications are the preferred method for use in riprap design. Simply put, reference specifications make use of recognized standards, such as "Standard Specifications for Transportation Materials and Methods of Sampling and Testing" (AASHTO, 2003) or "Annual Book of ASTM Standards" (ASTM, 2003a, b), to describe the nature and quality of the material required for a particular installation. Reference specifications are used for both materials and testing methods. Under this system, the engineer maintains control of the material type, size, and quality (thereby achieving the desired life-cycle performance), while at the same time allowing competition during the bidding process.

In contrast, specifications known as "or equal" specifications should be avoided. Under this system, the design engineer specifies a particular material—by brand name, source, or generic name—followed by the phrase "or equal," or "or approved equal." This system of specification is prone to a number of drawbacks, not the least of which is the issue of how "equality" is to be determined, and by whom. For this and many other reasons, organizations such as the Construction Specifications Institute (CSI) and the American Institute of Architects (AIA) discourage the use of "or equal" specifications (Rosen, 1981).

Material Specifications. Material specifications for riprap typically address the following characteristics of the rock used as the armor layer, or of the aggregate underlayer(s):

- Allowable range of sizes (or weights) of the individual particles
- Allowable range of particle shape (length, width, thickness)
- Minimum allowable density (or specific gravity)
- Maximum allowable water absorption
- Minimum allowable durability, which can include
  - Resistance to abrasion
  - Resistance to chemical weathering
  - Resistance to degradation by repeated freezing and thawing
  - Resistance to degradation by repeated wetting and drying

Size. 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 example, the designer may specify a minimum d_{50} or d_{30} for the rock composing the riprap, thus indicating the size for which 50% or 30% (by weight) of the particles are smaller. Stone sizes can also be specified in terms of weight (e.g., W_{50} or W_{30}) through the use of an equivalent spherical or cuboidal particle shape, and the known (or assumed) density of the particle.

Many different systems have been developed to describe the allowable range of particle size distribution (also referred to as gradation). Typically, an agency will designate standardized particle gradations associated with various size categories, referred to as "classes." Once a representative stone size is determined for a particular project, the designer will specify the class of riprap that meets or exceeds this size. Known as the "next larger" method of specification, this technique typically results in some conservatism, in that somewhat larger stone is used compared to the design value.

Standard grading classes for armorstone in Europe are included in a manual prepared under a collaborative project by CEN. The standard grading classes for armor stone are relatively narrow. Rock gradations are divided into three categories (CEN, 2002):

- Heavy grading for larger sizes normally handled as individual particles
- Light grading for armor layers, underlayers (i.e., bedding layers), and filter layers that are produced in bulk
- Fine grading for all rock that can be processed by production screens with square openings less than 360 mm

Table 2.2 provides an example of riprap classes and the allowable range associated with each (after ASTM D 6092, "Standard Practice for Specifying Standard Sizes of Stone for Erosion Control").

Gradation. Most standard classification systems for riprap specification include recommended limits on rock size for each class (e.g., Brown and Clyde, 1989; CEN, 2002). However, very few of the previously discussed studies have specifically examined the effect of riprap gradation on stability.

Most studies suggest that a well-graded riprap layer is better suited to resist the winnowing of bed sediments compared to a layer that exhibits a uniform gradation. HDS 6 (Richardson et al., 2001) states, "A uniformly graded riprap with a median size d_{50} scours to a greater depth than a well-graded mixture with the same median size." HDS 6 attributes this behavior to the ability of the well-graded stone to armor itself after the finer particles have been winnowed away. To mitigate winnowing, the CABS approach (Racin et al., 2000) uses multiple layers, with each layer composed of uniformly graded stone. The outer (armor) layer is composed of the largest stone size, with inner layers composed of successively smaller, uniformly sized stones.

Wittler and Abt (1990) conducted tests on various sizes and gradations of rock riprap having a d_{50} ranging from 2 to 4 in (0.05 to 0.1 m), on slopes ranging from 10% to 20%.

Table 2.2. Standard sizes of riprap.

Particle Mass		Size Designation (Class)					
Pounds	Kilograms	R-1500	R-700	R-300	R-150	R-60	R-20
3000	1400	100					
1500	680	50-100	100				
1000	450						
700	320	15-50	50-100	100			
500	230						
300	140		15-50	50-100	100		
250	110	0-15					
150	68			15-50	50-100	100	
60	27		0-15		15-50	50-100	
45	20			0-15			100
30	14					15-50	
20	9.1				0-15		50-100
10	4.5					0-15	15-50
2	0.9						0-15

Note: Values in table represent percentage by weight of stones lighter than the mass specified.

Source: modified from ASTM D 6092

They observed that greater stability was afforded by the more uniform gradations and concluded that the improved stability of uniform riprap is due to more efficient transfer of stress than that which occurs in well-graded riprap. A more uniform bearing stress between similarly sized particles, and the transfer of loads through the centers of the particles rather than tangentially, are given as possible reasons for the greater stability. The study also concluded, however, that failure of uniformly graded riprap is much more sudden than well-graded riprap. In addition, they observed that the uniform riprap does not tend to "self-heal" as does a well-graded riprap.

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 2.2.

Riprap stones should not be thin and platy, nor should they be long and needle-like. Therefore, specifying an allowable value for the ratio A/C provides a measure of suitable particle shape, because the B axis (width) is intermediate between the two extremes of length A and thickness C. Typically, a maximum allowable value A/C of 3.0 is used:

$$A/C \leq 3.0 \tag{2.15}$$

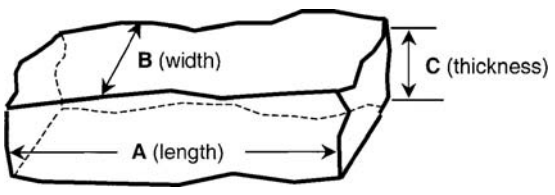


Figure 2.2. Riprap shape described by three axes.

"Angularity" is often used as a qualitative descriptor of shape, inasmuch as it affects the angle of repose. "Angular" particles are defined as having sharply defined edges and corners, whereas "rounded" particles are more potato-shaped, having been worn and abraded by physical contact, typically during fluvial transport. Intermediate between these two extremes are particles that are "subangular" or "subrounded." 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. The density of natural rock, measured in weight per volume, affects the size of stone required to achieve a specified weight. A more useful measure of density 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:

$$S_g = \frac{\gamma_s}{\gamma_w} \tag{2.16}$$

Specific gravities of natural rocks can range from less than 1.0 (e.g., volcanic pumice) to more than 5 (e.g., metallic ores of hematite, magnetite, etc.). Usually, a minimum allowable specific gravity of 2.5 is required for riprap applications (CEN, 2002). Where quarry sources uniformly produce rock with a specific gravity significantly greater (such as dolomite, $S_g = 2.7$ to 2.8), the equivalent stone size can be substantially reduced and still achieve the desired particle weight gradation.

Durability. Degradation of individual riprap particles can occur by physical or chemical processes. As mentioned previously, there are various measures of durability. Ultimately, the durability specification is intended to ensure that the particles will not break down into smaller sizes for at least the intended life of the installation.

Typical durability specifications involve tests whereby samples of the rock are subjected to repeated cycles of stress. The stress involved may be that of abrasion (as in a rotating drum); freezing followed by thawing; wetting followed by drying; or chemical stresses such as immersion in a solution of sodium or magnesium sulfate. A "pass-fail" criterion is specified whereby the sample must retain a certain minimum percentage of its original weight in order to "pass" the test, after the sample has been subjected to a specified number of stress cycles.

Testing Specifications. Standard test methods have been developed to provide qualitative and quantitative measures of the material characteristics described in the previous section. Some tests can be performed in the field, whereas others require controlled laboratory conditions and calibrated, purpose-made instruments. Field tests can be performed at the quarry to pre-qualify potential source areas or at the job site as part of a construction QA/QC program.

CUR and Public Works Department (RWS) (1995) list six reasons for testing rock and aggregate:

- To assess the quality and usefulness of a new source of stone
- To ascertain whether the rock from a given source is changing or constant during the course of production and supply
- To compare the quality of stone from different sources
- To assess sample variability from within one source
- To predict the performance of material in service
- To ascertain that the rock characteristics satisfy a specification

Laboratory and field tests for riprap are evaluated and discussed in detail in Section 3.2.3.

U.S. Standards. Relevant standards relating to material type, characteristics, and testing of riprap and aggregates typically associated with riprap installations (e.g., filter stone and bedding layers) are summarized in Table 2.3, while Table 2.4 provides U.S. standards for geotextiles used in conjunction with riprap installations.

Other Standards. Many other agencies and countries utilize material and testing standards for riprap that are very similar, if not identical, to the AASHTO and ASTM standards described in the previous section.

CUR and RWS (1995) make reference to several additional tests, including the methylene blue absorption (MBA) test, used to quantify the amount of clay minerals present in rock material—considered indicative of its general soundness. The so-called "drop test" whereby a sample is lifted to a certain height (usually 3 m) and dropped onto a bed of other rocks may give an estimate of the percentage of rocks that will break during transport and placement. Non-destructive tests involving sonic methods (e.g., Olson hammer) are described as being in development and calibration; apparently, these methods to date have not yet gained widespread acceptance in the industry.

CEN provides standard gradation classes for aggregates (five classes by size), light riprap (five classes by weight up to 300 kg), and heavy riprap (5 classes by weight up to 15,000 kg) (CEN, 2002). Particles with a length to thickness ratio A/C greater than 3.0 cannot be more than 20% by weight for aggregates and light riprap; for heavy riprap, the limit is 5% based on number of particles. Requirements for documenting the design, production, delivery, and placement of riprap are provided, as is guidance for general record-keeping procedures.

CEN provides recommendations for testing of basaltic rock for resistance to weathering due to solar radiation. This type of degradation is known as Sonnenbrand (literally, "sunburned"). Also, material specifications and testing requirements for steel slag and blast furnace slag are provided.

In Germany, BAW has developed a comprehensive code of practice for the use of geotextiles as a filter medium and/or separation layer beneath riprap (BAW, 1993a). This document provides a very useful matrix of geotextile tests that are required in each of four stages of a project:

1. Design stage (proof of fundamental suitability)
2. Bidding stage (prior to construction)
3. Production at quarry (quality control)
4. Control testing (conducted at random by owner on samples from job site)

When geotextiles are used as a filter, BAW requires them to exhibit a minimum thickness when certain conditions prevail at the installation site. These conditions very often result in the requirement that geotextiles have a thickness of 4.5 mm or more. Therefore, non-woven geotextiles are used much more frequently than woven geotextiles.

Construction/Installation Guidelines

If an integrated approach to riprap design and construction is to be developed, it must include consideration of the multiple factors involved in construction/installation regardless of project size. Information on site requirements, placement of filters, placement and equipment techniques, and termination details for the armor layer are discussed in Section 3.2.4. Photographs of typical riprap construction equipment are included to support the discussion.

Inspection and Quality Control

Based on the survey of current practice (Appendix B), very little guidance is being promulgated by the DOTs for inspection and quality control either during construction or for long-term monitoring. In Section 3.2.5, inspection guidelines and procedures, to include inspection forms, are developed. A field test procedure described by Galay et al.

Table 2.3. Standard specifications and test methods from AASHTO and ASTM for riprap and aggregate.

Desig-nation	Mat'l Spec	Test Spec	Title	Scope	Comments
AASHTO Standards for Rock and Aggregate					
M 43	✓		Specification for Sizes of Aggregate for Road and Bridge Construction	Defines the size designations and ranges for standard classes of coarse aggregate	Typically used for specifying granular filter stone
M 80	✓		Specification for Coarse Aggregate for Portland Cement Concrete	Covers coarse aggregate for use in concrete	Provides references to recommended tests for compatibility with Portland cement under a variety of conditions
TP 61		✓	Method of Test for Determining the Percentage of Fracture in Coarse Aggregate	Determines the percentage by mass that consists of fractured particles meeting certain requirements	Visual determination of fractured particles. Designer must specify a maximum acceptable value resulting from this test method
T 85		✓	Method of Test for Specific Gravity and Absorption of Coarse Aggregate	Determines the specific gravity of the stone and the amount of water absorption after 15 hours of soaking	Designer must specify acceptable values resulting from this test method
T 103		✓	Method of Test for Soundness of Aggregates by Freezing and Thawing	Determines the weight loss due to disintegration by repeated freezing and thawing	Designer must specify the number of cycles and a maximum acceptable value resulting from this test method
T 210		✓	Method of Test for Aggregate Durability Index	Determines the relative resistance of aggregate to degradation by mechanical abrasion	Designer must specify a minimum acceptable value resulting from this test method
TP 58		✓	Method of Test for Resistance of Coarse Aggregate to Degradation by Abrasion in the Micro-Deval Apparatus	Determines the resistance of aggregate to degradation by mechanical abrasion using the Micro-Deval apparatus	Similar to AASHTO T 210; includes steel balls as part of the abrasive charge
T 104		✓	Method of Test for Soundness of Aggregate by Use of Sodium Sulfate or Magnesium Sulfate	Determines the weight loss due to disintegration by repeated immersion in solution and subsequent drying	Simulates freeze-thaw action. Designer must specify the number of cycles and a maximum acceptable value resulting from this test method
ASTM Standards for Rock and Aggregate					
C 535		✓	Test Method for Resistance to Degradation of Large-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine	Determines the resistance of aggregate to degradation by mechanical abrasion, impact, and grinding using the Los Angeles apparatus	Similar to AASHTO T 210; includes steel balls as part of the abrasive charge

(1987) is presented as an example of a simple, practical approach to ensuring (1) that an appropriate riprap size distribution is achieved during construction and (2) that the stone does not deteriorate over the long term. Other field tests suitable for inspection and quality control are discussed in Section 3.2.3.

Post-Construction/Post-Flood Inspection

No specific post-construction/post-flood inspection guidance for riprap was identified in the U.S. literature or as a result of the survey. In Europe (CUR and RWS, 1995), a systematic approach to post-construction inspection has

Table 2.3. Standard specifications and test methods from AASHTO and ASTM for riprap and aggregate (continued).

Desig-nation	Mat'l Spec	Test Spec	Title	Scope	Comments
ASTM Standards for Rock and Aggregate (continued)					
D 3967		✓	Test Method for Splitting Tensile Strength of Intact Rock Core Specimens	Determines the pressure load (force per unit area) required to split a cylindrical rock sample	Designer must specify a minimum acceptable value resulting from this test method
D 4992	✓	✓	Practice for Evaluation of Rock to be Used for Erosion Control	Provides guidance to aid in assessing the suitability of rock for riprap using field and laboratory tests	Includes a good summary of various test procedures. Does not provide suggested values for pass-fail criteria.
D 5240		✓	Test Method for Testing Rock Slabs to Evaluate Soundness of Riprap by Use of Sodium Sulfate or Magnesium Sulfate	Similar to AASHTO T 104, but specifically deals with large rock sizes	Same as AASHTO T 104
D 5312		✓	Test Method for Evaluation of Durability of Rock for Erosion Control Under Freezing and Thawing Conditions	Similar to AASHTO T 103, but specifically deals with large rock sizes	Provides a map of the United States showing isolines of freeze-thaw severity
D 5519		✓	Test Method for Particle Size Analysis of Natural and Man-Made Riprap Materials	Determines the size and mass gradation of rock greater than 3 in in size	Used in conjunction with D 6092 or other gradation classification system
D 5779		✓	Test Method for Field Determination of Apparent Specific Gravity of Rock and Manmade Materials for Erosion Control	Determines specific gravity by weight and water displacement	Field test using simple apparatus
D 5873		✓	Test Method for Determination of Rock Hardness by Rebound Hammer	Determines the "rebound hardness" of a rock specimen, a dimensionless number indicating relative hardness	Can be used in the lab or in the field. Sometimes referred to as the Schmidt Hammer method.
D 6092	✓		Practice for Specifying Standard Sizes of Stone for Erosion Control	Provides recommended gradation ranges for six different classes of riprap	Includes conversion of weight to equivalent size, assuming stone is midway between a sphere and a cube
D 6825			Guide for Placement of Riprap Revetments	Provides guidance on placement of riprap rock as well as ancillary components such as granular filters or geotextiles	Includes equipment requirements and recommendations for earthwork and subgrade preparation

Table 2.4. Standard specifications and test methods from AASHTO and ASTM for geotextiles associated with riprap installations.

Designation	Mat'l Spec	Test Spec	Title	Scope	Comments
AASHTO Standards for Geotextiles					
M 288	✓		Geotextile Specification for Highway Applications	Covers geotextile fabric characteristics for use in various applications, including as a filter under riprap	Includes installation guidelines as well as material requirements
ASTM Standards for Geotextiles					
D 4354		✓	Practice for Sampling of Geosynthetics for Testing	Describes three procedures for the sampling of geosynthetics for testing	Requires that instructions for taking laboratory samples and test specimens be provided for every test method for geosynthetics.
D 4355		✓	Test Method for Deterioration of Geotextiles from Exposure to Ultraviolet Light and Water (Xenon-Arc Type Apparatus)	Determines the deterioration in tensile strength of geotextiles by exposure to ultraviolet light and water	Designer must specify what is a maximum acceptable value resulting from this test method
D 4439			Terminology for Geosynthetics	Provides definitions of terms used in the testing and specification of geosynthetics	
D 4491		✓	Test Methods for Water Permeability of Geotextiles by Permittivity	Determines the hydraulic conductivity (water permeability) of geotextiles in terms of permittivity under standard testing conditions, in the uncompressed state	Includes two procedures: the constant head method and the falling head method. Designer must specify what is a minimum acceptable value resulting from this test method
D 4533		✓	Standard Test Method for Trapezoid Tearing Strength of Geotextiles	Determines the force required to continue or propagate a tear in woven or non-woven geotextiles by the trapezoid method	Designer must specify what is a minimum acceptable value resulting from this test method

been developed to support decisions on maintenance requirements at riprap installations. The approach considers four aspects of inspection/monitoring: (1) location (i.e., settlement or movement of the riprap particles), (2) geometry (i.e., bank slope/sloughing compared to the as-built condition), (3) composition (i.e., loss or movement of rocks or presence of voids), and (4) riprap elements (i.e., deterioration or wear of individual particles). In Chapter 3, riprap failure mechanisms are identified as a basis for developing inspection guidance, and selected case studies of failures are used to illustrate and emphasize the need for post-construction/post-flood inspection (Section 3.7). Sections 3.2.3 and 3.2.5 discuss field tests and other procedures suitable for post-construction/post-flood inspection.

2.4.2 Bridge Pier Riprap

Design Guidelines

Pier Scour. The basic mechanism causing local scour at piers is the formation of vortices (known as the horseshoe vortex) at their base (Figure 2.3). The horseshoe vortex results from the pileup of water on the upstream surface of the obstruction and subsequent acceleration of the flow around

Table 2.4. Standard specifications and test methods from AASHTO and ASTM for geotextiles associated with riprap installations (continued).

Designation	Matíl Spec	Test Spec	Title	Scope	Comments
ASTM Standards for Geotextiles (continued)					
D 4595		✓	Standard Test Method for Tensile Properties of Geotextiles by the Wide-Width Strip Method	Determines the tensile properties of geotextiles using a wide-width strip specimen tensile method. Covers the measurement of tensile strength and elongation	Includes directions for the calculation of initial modulus, offset modulus, secant modulus, and breaking toughness.
D 4632		✓	Test Method for Grab Breaking Load and Elongation of Geotextiles	Determines the breaking load (grab strength) and elongation (grab elongation) of geotextiles using the grab method	Designer must specify what is a minimum acceptable value resulting from this test method
D 4751		✓	Test Method for Determining Apparent Opening Size of a Geotextile	Determines the apparent opening size (AOS) of a geotextile by sieving glass beads through a geotextile	Designer must specify what is a maximum acceptable value resulting from this test method
D 4759	✓		Practice for Determining the Specification Performance of Geosynthetics	Determines the conformance of geosynthetic properties to standard specifications.	
D 4833		✓	Test Method for Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products	Determines the index puncture resistance of geotextiles, geomembranes, and related products	Designer must specify what is a minimum acceptable value resulting from this test method
D 4886		✓	Test Method for Abrasion Resistance of Geotextiles (Sand Paper/Sliding Block Method)	Determines the resistance of geotextiles to abrasion	Designer must specify what is a minimum acceptable value resulting from this test method
D 5321		✓	Standard Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method	Determines the shear resistance of a geosynthetic against soil, another geosynthetic, or a soil and geosynthetic in any combination.	The test method is intended to indicate the performance of the selected specimen by attempting to model certain field conditions.

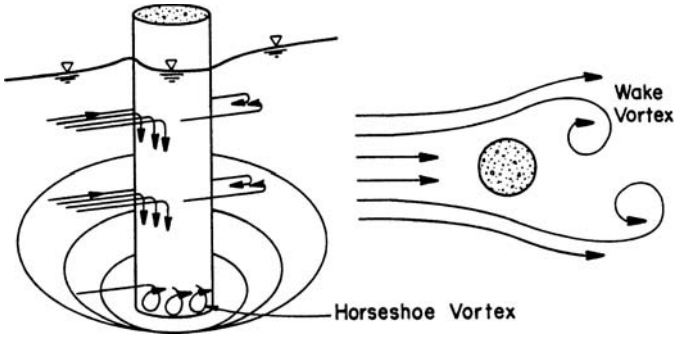


Figure 2.3. Schematic representation of scour at a cylindrical pier.

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 reestablished 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 2.3). 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.

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.

Since 1998, additional studies have been 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 have modeled live-bed conditions, since 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. (See also Section 2.4.6 for a discussion of studies conducted under NCHRP Project 24-07(2)).

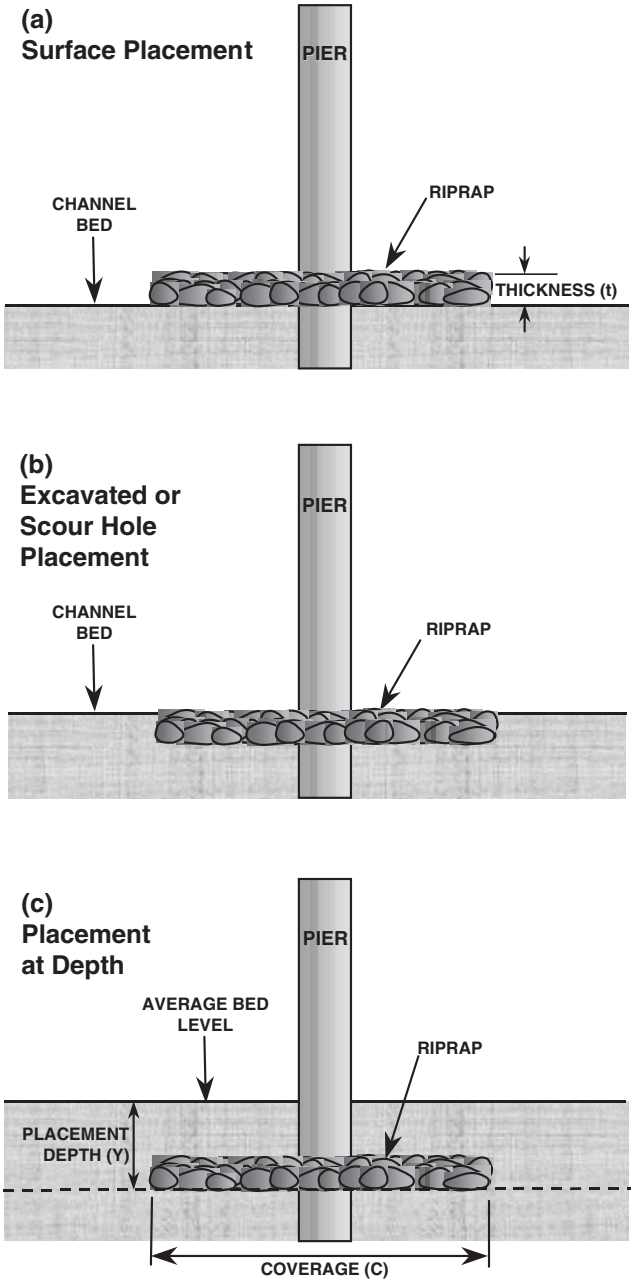


Figure 2.4. Typical pier riprap configurations.

Typically riprap used for pier scour protection is placed on the surface of the channel bed (Figure 2.4a), in a pre-existing scour hole, or in a hole excavated around the pier (Figure 2.4b). However, recent studies as described in the following sections, recommend placing the riprap layer at depth below the average bed level (Figure 2.4c).

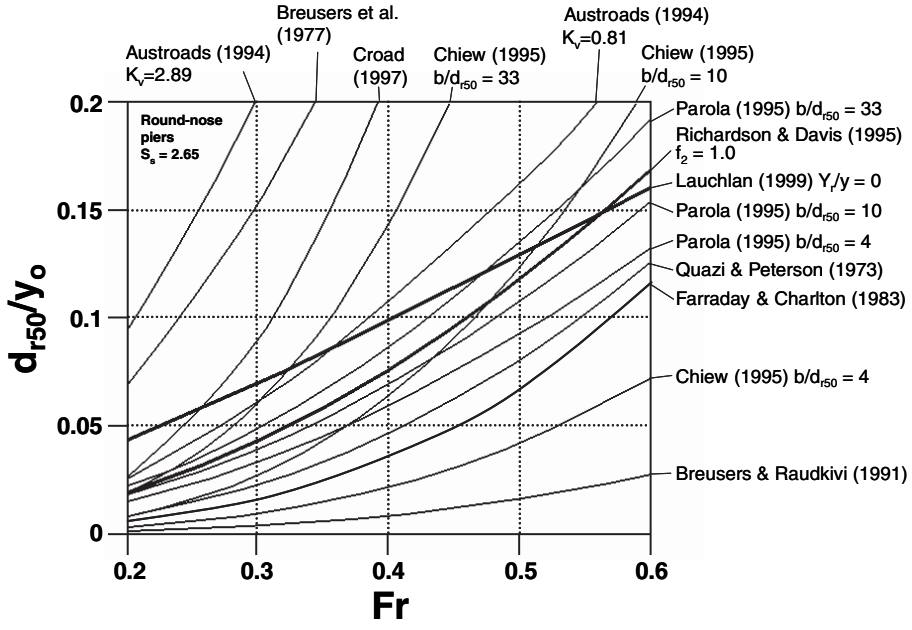
Sizing of Pier Riprap. In addition to the literature review conducted by Parker et al. (1998), comprehensive reviews of the literature on sizing of riprap for bridge piers have been conducted by Fotherby (1995), CUR and RWS (1995), Lauchlan (1999), Melville and Coleman (2000), and Lauchlan and Melville (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 2.5 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-nose 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 2.5). Lauchlan (1999), Melville and Coleman (2000), and Lauchlan et al. (2000a) compare these equations in detail. The lack of consistency among the methods led Melville and Coleman (2000) to recommend the use of the HEC-18 (Richardson and Davis, 1995; because countermeasure design topics are now covered in HEC-23, see also 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) used these methods to assess riprap size requirements for the Hutt Estuary Bridge in New Zealand. They were found to provide good agreement with model study results (Lauchlan et al., 2000b).

To determine the d_{50} size of pier riprap FHWA HEC-18 (Richardson and Davis, 1995) and HEC-23 (Lagasse et al.,



Source: modified from Lauchlan (1999)

Figure 2.5. Comparison of equations for sizing riprap at round-nose bridge piers.

Table 2.5. Equations for sizing riprap at bridge piers.

Reference	Equation	Standard Format (for comparison)	Comments
Bonasoundas (1973)	d_{r50} (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)	$N_{sc} = 1.14\left(\frac{d_{r50}}{y}\right)^{-0.2}$	$\frac{d_{r50}}{y} = \frac{0.85}{(S-1)^{1.25}}Fr^{2.5}$	N_{sc} = critical stability number = $V^2/[g(S_s\text{-}1)d_{r50}]$ 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)	$V = 4.8(S_s-1)^{0.5}d_{r50}{}^{1/3}y^{1/6}$	$\frac{d_{r50}}{y} = \frac{0.278}{(S_s-1)^{1.5}}Fr^3$	
Austroads (1994)	$\frac{d_{r50}}{y} = \frac{0.58K_pK_v}{(S_s-1)}Fr^2$	$\frac{d_{r50}}{y} = \frac{0.58K_pK_v}{(S_s-1)}Fr^2$	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)	$d_{r50} = \frac{0.692(f_1f_2V)^2}{(S_s-1)2g}$	$\frac{d_{r50}}{y} = \frac{0.346f_1^2f_2^2}{(S_s-1)}Fr^2$	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)	$d_{r50} = \frac{0.168}{\sqrt{y}}\left(\frac{V}{U_*\sqrt{(S_s-1)g}}\right)^3$	$\frac{d_{r50}}{y} = \frac{0.168}{(S_s-1)^{1.5}U_*^3}Fr^3$ $U_* = \frac{0.3}{K_dK_y}$	$K_y = 0.783\left(\frac{y}{b}\right)^{0.322} - 0.106$ $0 \le (y/b) < 3$ $K_y = 1$ $(y/b) \ge 3$ $K_d = 0.398\ln\left(\frac{b}{d_{r50}}\right) - 0.034\left[\ln\left(\frac{b}{d_{r50}}\right)\right]^2$ $1 \le (b/d_{r50}) < 50$ $K_d = 1$ $(b/d_{r50}) \ge 50$ K_y = flow depth factor K_d = sediment size factor

2001) recommend using the rearranged Isbash equation to solve for stone diameter for fresh water:

$$d_{50} = \frac{0.692(KV)^2}{(S_s-1)2g} \qquad (2.17)$$

where

d_{50} = Median stone diameter, ft or m
K = Coefficient for pier shape (1.5 for round-nose pier, 1.7 for rectangular pier)
V = Velocity on pier, ft/s or m/s
S_s = Specific gravity of riprap (normally 2.65)
g = Acceleration due to gravity, ft/s^2 or m/s^2

To determine the velocity on the pier, V, 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.

Only recently have studies been conducted to address riprap size with regard to stability at bridge piers under live-bed

Table 2.5. Equations for sizing riprap at bridge piers (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 = $f(b_p/d_{r50})$: $f_3 = 0.83$ $4<(b_p/d_{r50})<7$ $f_3 = 1.0$ $7<(b_p/d_{r50})<14$ $f_3 = 1.25$ $20<(b_p/d_{r50})<33$
Croad (1997)	$\frac{V}{A\sqrt{(S_s-1)gd_{r50}}} = 1.16\left(\frac{y}{d_{r50}} - 2\right)^{1/6}$ $d_{r50} = 17 d_{b50}$	$\frac{d_{r50}}{y}\left(1 - \frac{2d_{r50}}{y}\right)^{0.5} = \frac{0.641}{A^3(S_s-1)^{1.5}} Fr^3$ $d_{r50} = 17 d_{b50}$ 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 Croad (1997)
Lauchlan (1999)	$\frac{d_{r50}}{y} = 0.3 S_f \left(1 - \frac{Y_r}{y}\right)^{2.75} Fr^{1.2}$	$\frac{d_{r50}}{y} = 0.3 S_f \left(1 - \frac{Y_r}{y}\right)^{2.75} Fr^{1.2}$	S_f = safety factor, with a minimum recommended value of 1.1 Y_r = placement depth below bed level

Source: Melville and Coleman (2000)

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 eroded, 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.

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., $d_r/d_{smax} > 20\%$) 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) have provided 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

$$\frac{d_{50}}{y_o} = K_D K_S K_\alpha K_Y 0.3 F^{1.2} \qquad (2.18)$$

where

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

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

However, Fotherby and Ruff (1999) 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 2.18 to be rewritten as

$$\frac{d_{50}}{y_o} = 0.3\left(1 - \frac{Y}{y_o}\right)^{2.75} F^{1.2} \qquad (2.19)$$

For high Froude numbers (Figure 2.5), 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 indicates that riprap size for a given Froude number decreases with increasing placement depth (Y) below the ambient bed level.

In a comprehensive parametric study, Lim and Chiew (2001) noted 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, especially at the upper end of the dune regime where large stones offer no additional protection against pier scour. In contrast, clear-water experiments conducted by Parola (1995) led him to suggest that large riprap may act to dissipate pier-induced vortices, especially when riprap size approaches the size of the vortices. He reasoned 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 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.

Riprap Filter Requirements. There are two kinds of filters used in conjunction with bridge pier riprap; stone filters and geotextile filters. Stone filters are composed of rock that may or may not be graded and has 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.

In Europe, fascine mats are commonly used as a means of placing a geotextile filter in deep water. Fascine mats 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 (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 and RWS (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 placing a geotextile under a riprap with the same areal coverage as the riprap layer resulted in relatively poor performance of the riprap. 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. 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. 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.

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 (3) 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 installation 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 and RWS, 1995). According to Lagasse et al. (2001), 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 (see Section 3.2.2).

Material and Testing Specifications

No material testing specifications specific to pier riprap were found. In general, specifications for revetment riprap will also apply to pier riprap (see Section 2.4.1).

Construction/Installation Guidelines

Specifications and guidance on the placement level, areal coverage, thickness, and grading of a riprap layer placed around a bridge pier vary widely. Table 2.6 summarizes many of the methods used to estimate the extent of coverage, thickness, level of placement, and grading requirements for pier riprap. In this table, the "b" dimension is the pier width perpendicular to the flow direction.

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 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) notes 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 2.4a) because of the ease 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. FHWA (Lagasse et al., 2001; Richardson and Davis, 1995) recommends placing the top of the riprap layer flush with the channel bed for inspection purposes (Figure 2.4b). 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 2.4a), 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 before the 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, 1998, 1999; Ruff and Nickelson, 1999). 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) recommended that riprap near bridge piers would perform most successfully when placed at the trough elevation of the largest bed forms.

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 (see Section 3.7.2). 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 before the transition to a plane bed. 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.

Table 2.6. Methods to estimate riprap extent, gradation, and filter requirements.

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 (2004)	Project around the nose of the pier by a distance = 1.5b	$>2d_{r50}$		
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	$3d_{r50}$	Some distance below bed level to prevent excessive exposure	
Lagasse et al. (2001)	Width > 5b	$> 3d_{r50}$	Top of riprap at bed level	$d_{r50} \geq 0.5d_{rmax}$
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	$2d_{r50}$		$d_{rmax} \leq 2d_{r50}$ $d_{r50} \leq 2d_{r15}$
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	$2d_{r50}$ to $3d_{r50}$	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	$\geq 3d_{r50}$	Place mat below streambed a depth equivalent to the expected scour	
Fotherby (1995) Fotherby and Ruff (1999)	$1.5b_a$ 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 and RWS (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 $3d_{r50}$		
Lim and Chiew (2001)	FHWA coverage of 2b from pier face (extent of coverage has no effect at upper dune regime)	$>1.5d_{r50}$ or d_{r100}		

Source: modified from Melville and Coleman (2000)

The comprehensive study conducted by Lauchlan (1999) indicates that placing the riprap layer at depth (Figure 2.4c) 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 2.18 and 2.19 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 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) found 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 2.6, the recommended coverage varies with pier shape and can extend as little as one pier width from the pier face to as much as 7 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 2 to 3 times the median stone size of the riprap (Table 2.6). 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 similar 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 2.6 provide some guidance on riprap gradation. Brown and Clyde (1989) provide gradation limits and classes and CEN (2002) provides gradation class requirements and grading curves for general use in riprap revetments (see the discussion in Section 2.4.1 for more information).

Summary. Based on much of the information in Table 2.6, Melville and Coleman (2000) provide the following recommendations for riprap protection at bridge piers:

- Riprap size: based on Lauchlan (1999) equation (Equation 2.18) 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

Inspection and Quality Control

No inspection or quality control guidelines specific to pier riprap were found. In general, inspection and quality control guidelines for revetment riprap will also apply to pier riprap (see Section 2.4.1). Since pier riprap may be placed in deep, fast moving water, inspection requirements may influence riprap placement guidelines. As discussed under "Construction/Installation Guidelines," placement on the bed or at the bed surface may be required so that pier riprap condition can be evaluated (Figure 2.4). In addition, inspection techniques such as the use of probes or portable sonic sounders may be required for pier riprap (see Lagasse et al., 2001). In some cases, underwater inspection by divers may be required.

2.4.3 Bridge Abutment Riprap

Design Guidelines

Abutment Scour. Scour occurs at abutments when the abutment and embankment obstruct the flow. Several causes of abutment failures during post-flood field inspections of bridge sites have been documented (Parola et al., 1998):

- Overtopping of abutments or approach embankments
- Lateral channel migration or stream widening processes
- Contraction scour
- Local scour at one or both abutments

Abutment damage is often caused by a combination of these factors. Where abutments are set back from the channel banks, especially on wide floodplains, large local scour holes have been observed with scour depths of as much as four times the approach flow depth on the floodplain. As a general rule, the abutments most vulnerable to damage are those located at or near the channel banks.

The flow obstructed by the abutment and highway approach embankment forms (1) a horizontal vortex starting at the upstream end of the abutment and running along the toe of the abutment and (2) a vertical wake vortex at the

downstream end of the abutment. The vortex at the toe of the abutment is very similar to the horseshoe vortex that forms at piers, and the vortex that forms at the downstream end is similar to the wake vortex that forms downstream of a pier. Research has been conducted to determine the depth and location of the scour hole that develops for the horizontal (so called "horseshoe") vortex that occurs at the upstream end of the abutment, and numerous abutment scour equations have been developed to predict this scour depth.

Abutment failures and erosion of the fill also occur from the action of the downstream wake vortex. However, research and the development of methods to determine the erosion from the wake vortex have not been conducted. An example of abutment and approach embankment erosion of a bridge due to the action of the horizontal and wake vortex is shown in Figure 2.6. The types of failures described above are initiated as a result of the obstruction to the flow caused by the abutment and highway embankment and subsequent contraction and turbulence of the flow at the abutments.

Design Approach. The preferred design approach is to place the abutment foundation on scour resistant rock or on deep foundations. Available technology has not developed sufficiently to provide reliable abutment scour estimates for all hydraulic flow conditions that might be reasonably expected to occur at an abutment. Therefore, engineering judgment is required in designing foundations for abutments. In many cases, foundations can be designed with shallower depths than predicted by the equations when the foundations are protected with riprap and/or a guide bank placed upstream of the abutment designed in accordance with guidelines in HEC-23 (Lagasse et al., 2001). Cost will be the deciding factor (Richardson and Davis, 2001).

Figure 2.6. Scour of bridge abutment and approach embankment.

The potential for lateral channel migration, long-term degradation, and contraction scour should be considered in setting abutment foundation depths near the main channel. The abutment scour equations presented in HEC-18 (Richardson and Davis, 2001) are recommended for use to develop insight as to the scour potential at an abutment.

Where spread footings are placed on erodible soil, the preferred approach is to place the footings below the elevation of total scour. If this is not practicable, a second approach is to place the top of footings below the depth of the sum of contraction scour and long-term degradation and to provide scour countermeasures. For spread footings on erodible soil, protection of adjacent embankment slopes with riprap or other appropriate scour countermeasures becomes especially important. The toe or apron of the riprap serves as the base for the slope protection and must be carefully designed to resist scour while maintaining the support for the slope protection.

In summary, as a minimum, abutment foundations should be designed assuming no ground support (lateral or vertical) as a result of soil loss from long-term degradation, stream instability, and contraction scour. The abutment should be protected from local scour using riprap and/or guide banks. To protect the abutment and approach roadway from scour by the wake vortex, several DOTs use a 50-foot (15-meter) guide bank extending from the downstream corner of the abutment. Otherwise, the downstream abutment and approach should be protected with riprap or other countermeasures (Richardson and Davis, 2001).

Sizing of Abutment Riprap. FHWA conducted two research studies in a hydraulic flume to determine equations for sizing rock riprap for protecting abutments from scour (Pagán-Ortiz, 1991; Atayee, 1993). The first study investigated vertical wall and spill-through abutments that encroached 28% and 56% on the floodplain, respectively. The second study investigated spill-through abutments that encroached on a floodplain with an adjacent main channel (Figure 2.7). Encroachment varied from the largest encroachment used in the first study to a full encroachment to the edge of main channel bank. For spill-through abutments in both studies, the rock riprap consistently failed at the toe downstream of the abutment centerline. For vertical wall abutments, the first study consistently indicated failure of the rock riprap at the toe upstream of the centerline of the abutment.

Cotton (1999) adapted the riprap factor of safety design approach (Stevens et al., 1976), as presented in HDS 6 (Richardson et al., 2001), to riprap design at bridges. This

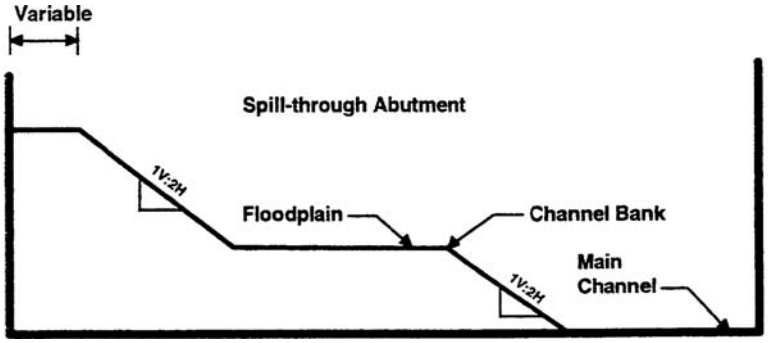


Figure 2.7. Section view of a typical setup of spill-through abutment on a floodplain with adjacent main channel.

conceptual framework allows the method to be applied to a wide variety of river conditions provided that the boundary shear stress can be estimated. Boundary shear stress adjustment factors for a range of conditions are presented, including channel bed in a contraction (bridge section), channel bed at a bridge pier, and abutments and guide banks (formerly known as spur dikes).

Lewis (1972) developed a technique for determining stable rock riprap sizes for flood protection of the channel bed and constricting embankments (abutments) at bridges. The method was tested with data from small-scale, riprap-protected embankments that were tested to destruction. The riprap factor of safety approach (Richardson et al., 2001) is adapted to the prediction of the stability of riprap particles to be placed on the side slopes and spill slopes of spill-through abutments.

In HEC-23 (Lagasse et al., 2001), FHWA recommends an abutment riprap design approach based on the FHWA studies referenced previously. For Froude numbers ($V/(gy)^{1/2}$) less than or equal to 0.80, the recommended design equation for sizing rock riprap for spill-through and vertical wall abutments is in the form of the Isbash relationship:

$$\frac{d_{50}}{y} = \frac{K}{(S_S - 1)}\left[\frac{V^2}{gy}\right] \qquad (2.20)$$

where

d_{50} = Median stone diameter, ft or m
y = Depth of flow in the contracted bridge opening, ft or m
K = 0.89 for a spill-through abutment
= 1.02 for a vertical wall abutment
S_s = Specific gravity of rock riprap
V = Characteristic average velocity in the contracted section, ft/s or m/s (explained below)
g = Gravitational acceleration, ft/s^2 or m/s^2

For Froude numbers greater than 0.80, Equation 2.21 is recommended:

$$\frac{d_{50}}{y} = \frac{K}{(S_s - 1)}\left[\frac{V^2}{gy}\right]^{0.14} \qquad (2.21)$$

where

K = 0.61 for spill-through abutments
= 0.69 for vertical wall abutments

In both equations, the coefficient K is a velocity multiplier to account for the apparent local acceleration of flow at the point of rock riprap failure. Both of these equations are envelope relationships that were forced to overpredict 90% of the laboratory data.

The recommended procedure for selecting the characteristic average velocity is as follows:

- Determine the set-back ratio (SBR) of each abutment. SBR is the ratio of the set-back length (the distance from the near edge of the main channel to the toe of abutment) to channel flow depth (SBR = Set-back length/average channel flow depth). If SBR
  - Is less than 5 for both abutments (Figure 2.8), compute a characteristic average velocity, Q/A, based on the entire contracted area through the bridge opening. This area includes the total upstream flow, exclusive of that which overtops the roadway.
  - Is greater than 5 for an abutment (Figure 2.9), compute a characteristic average velocity, Q/A, for the respective overbank flow only. Assume that the entire respective overbank flow stays in the overbank section through the bridge opening.
  - Is less than 5 for one abutment and more than 5 for the other abutment at the same site (Figure 2.10), a characteristic average velocity using the average bridge velocity may be unrealistically low. Whether it is unrealistically

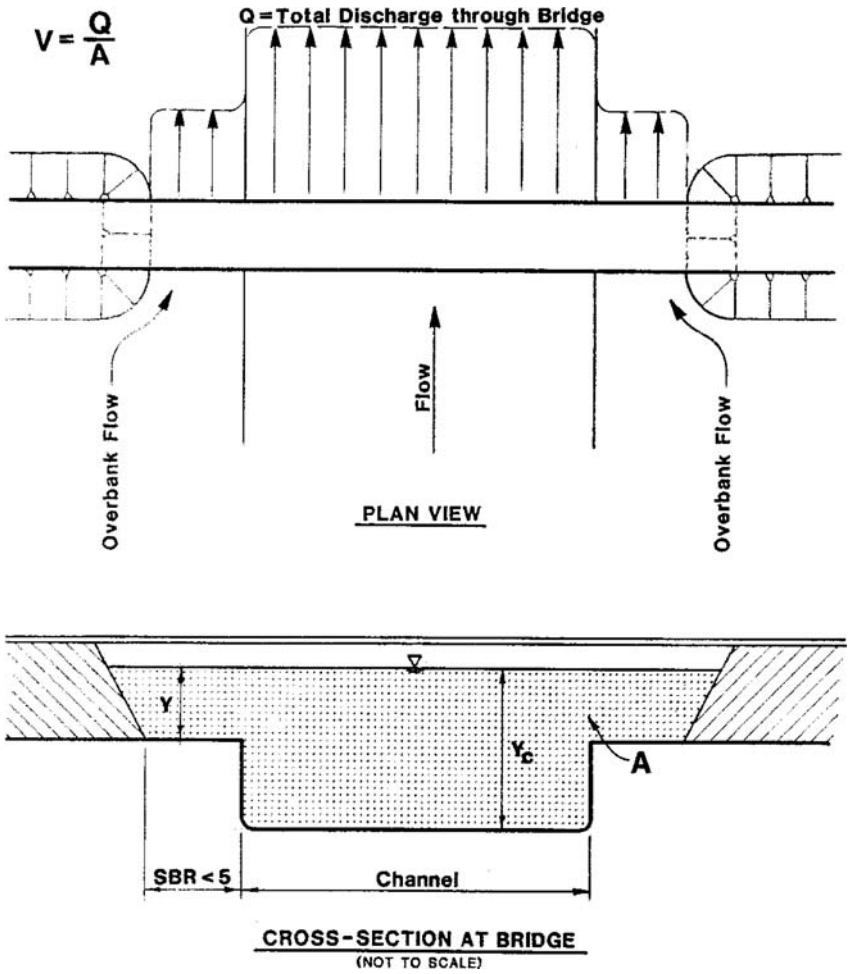


Figure 2.8. Characteristic average velocity for SBR < 5.

low would, of course, depend upon the opposite overbank discharge as well as how far the other abutment is set back. For this case, the characteristic average velocity for the abutment with SBR less than 5 should be based on the flow area limited by the boundary of that abutment and an imaginary wall located on the opposite channel bank. The appropriate discharge is bounded by this imaginary wall and the outer edge of the floodplain associated with that abutment.

- Compute the rock riprap size from Equation 2.20 or 2.21, based on the Froude number limitation for these equations.

Material and Testing Specifications

No material or testing specifications specific to abutment riprap were found. In general, specifications for revetment riprap will also apply to abutment riprap (see Section 2.4.1).

Construction/Installation Guidelines

FHWA HEC-23 (Lagasse et al., 2001) gives the extent of rock riprap and construction/installation guidelines at abutments as follows:

- The apron at the toe of the abutment should extend along the entire length of the abutment toe, around the curved portions of the abutment to the point of tangency with the plane of the embankment slopes.
- The apron should extend from the toe of the abutment into the bridge waterway a distance equal to twice the flow depth in the overbank area near the embankment, but need not exceed 25 ft (7.5 m) (Figure 2.11).
- Spill-through abutment slopes should be protected with the rock riprap size computed from Equations 2.20 or 2.21 to an elevation 2 ft (0.6 m) above expected high water elevation for the design flood. The downstream coverage should extend back from the abutment 2 flow depths

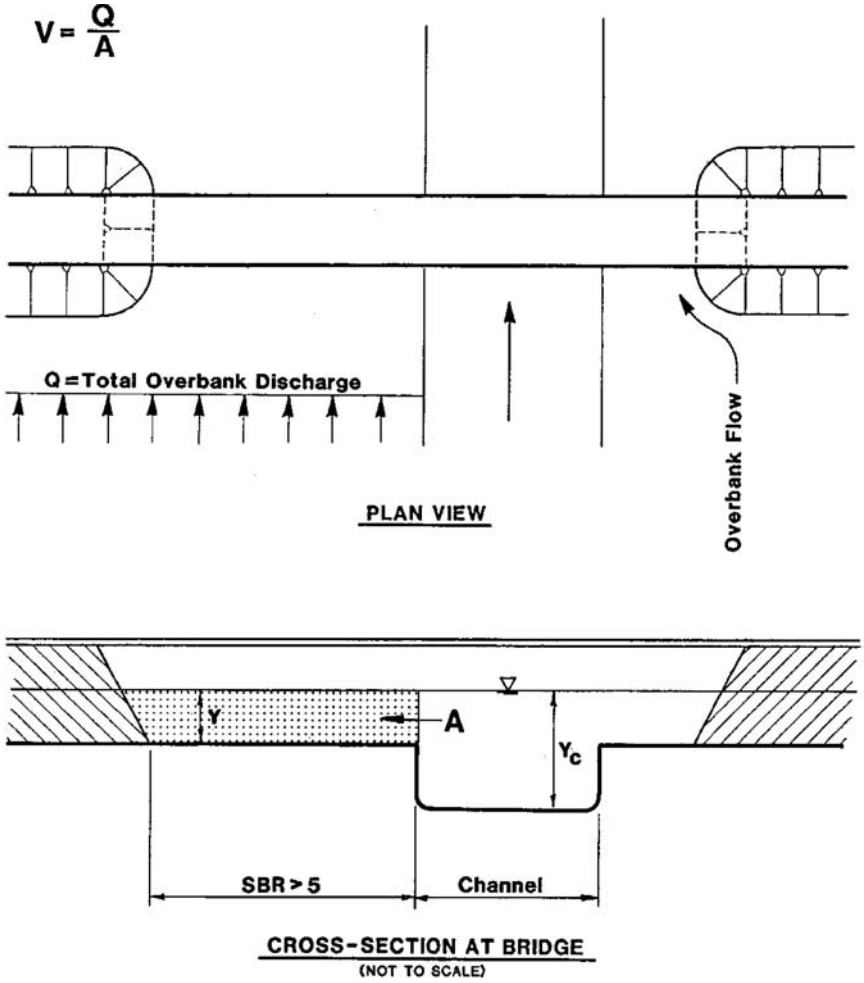


Figure 2.9. Characteristic average velocity for SBR > 5.

or 25 ft (7.5 m), whichever is larger, to protect the approach embankment. Several states in the southeast use a guide bank 50 ft (15 m) long at the downstream end of the abutment to protect the downstream side of the abutment.

- Rock riprap thickness should not be less than the larger of either 1.5 times d_{50} or d_{100}. The rock riprap thickness should be increased by 50% when it is placed under water to provide for the uncertainties associated with this type of placement.
- As with revetments, the rock riprap gradation and potential need for underlying filter material at an abutment must be considered (see Section 2.4.1).

Inspection and Quality Control

No inspection or quality control guidelines specific to abutment riprap were found. In general, inspection and quality control guidelines for revetment riprap will also apply to abutment riprap (see Section 2.4.1). Inspection techniques such as the use of probes or portable sonic sounders may be required for abutment riprap (Lagasse et al., 2001). In some cases, underwater inspection by divers may be required.

2.4.4 Guide Banks and Other Countermeasures

Design Guidelines

Guide Banks. When approach embankments encroach on wide flood plains, the flows from these areas must flow parallel to the embankment to the bridge opening. These flows can erode the approach embankment. A severe flow contraction at the abutment can reduce the effective bridge opening, which could possibly increase the severity of abutment and pier scour. Guide banks can be used in these cases to prevent erosion of the approach embankments by cut-

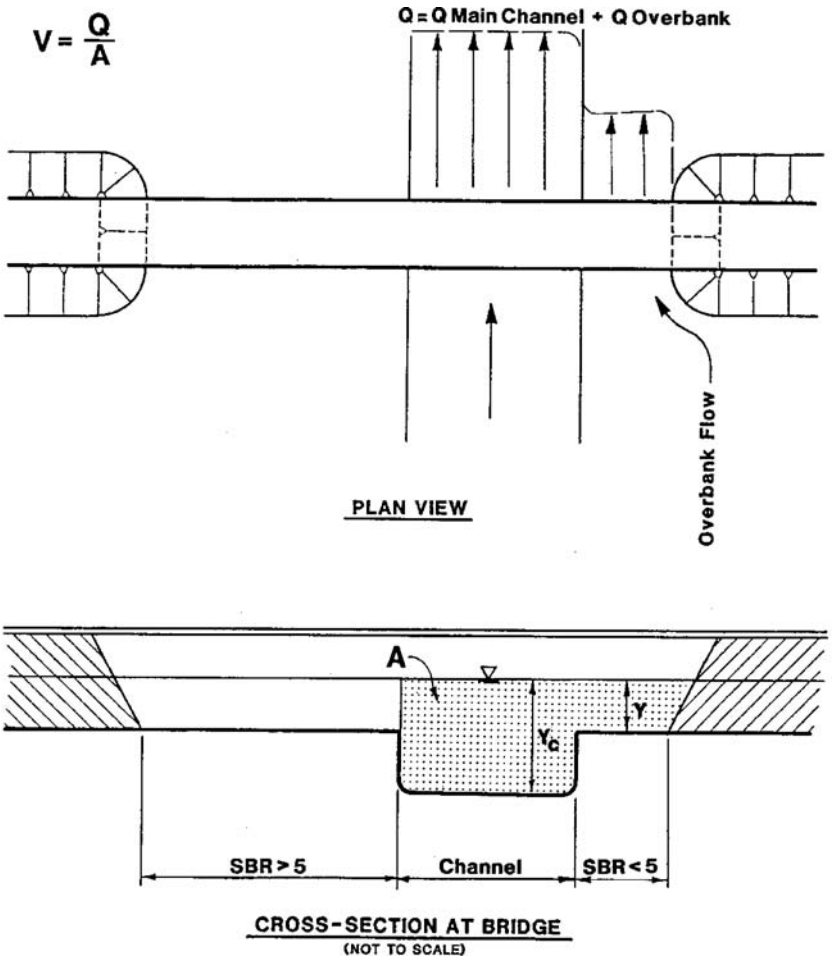


Figure 2.10. Characteristic average velocity for SBR > 5 and SBR < 5.

ting off the flow adjacent to the embankment, guiding stream flow through a bridge opening, and transferring scour away from abutments to prevent damage caused by abutment scour.

Figure 2.12 presents a typical guide bank plan view. It is apparent from the figure that, without this guide bank, overbank flows would return to the channel at the bridge opening, which can increase the severity of contraction and scour at the abutment. Note that, with installation of guide banks, the scour holes that normally would occur at the abutments of the bridge are moved upstream away from the abutments. Guide banks may be designed at each abutment, as shown, or singly, depending on the amount of overbank or flood plain flow directed to the bridge by each approach embankment. The goal in the design of guide banks is to provide a smooth transition and contraction of the stream flow through the bridge opening.

Spurs. A spur can be a pervious or impervious structure projecting from the streambank into the channel. Spurs are used to deflect flowing water away from, or to reduce flow velocities in, critical zones near the streambank; to prevent erosion of the bank; and to establish a more desirable channel alignment or width. The main function of spurs is to reduce flow velocities near the bank, which in turn, encourages sediment deposition due to these reduced velocities. Increased protection of banks can be achieved over time, as more sediment is deposited behind the spurs. Therefore, spurs may protect a streambank more effectively and at less cost than revetments. Furthermore, by moving the location of any scour away from the bank, partial failure of the spur can often be repaired before damage is done to structures along and across the stream (Brown, 1985a; Lagasse et al., 2001).

Spurs are generally used to halt meander migration at a bend. They are also used to focus wide, poorly defined streams into well-defined channels or to improve navigation. The use of spurs to establish and maintain a well-defined channel location, cross section, and alignment in braided streams can decrease the required bridge

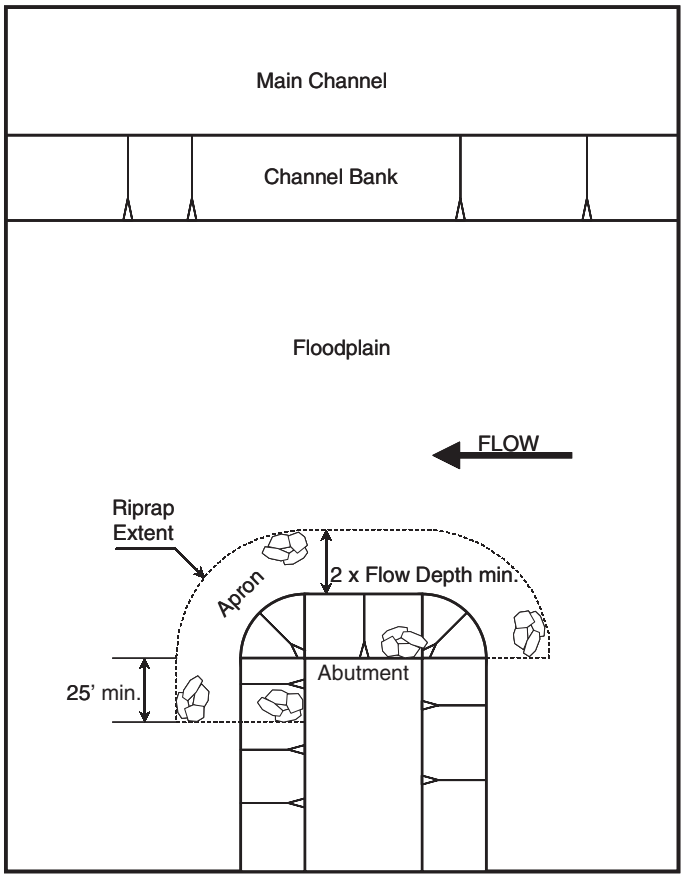
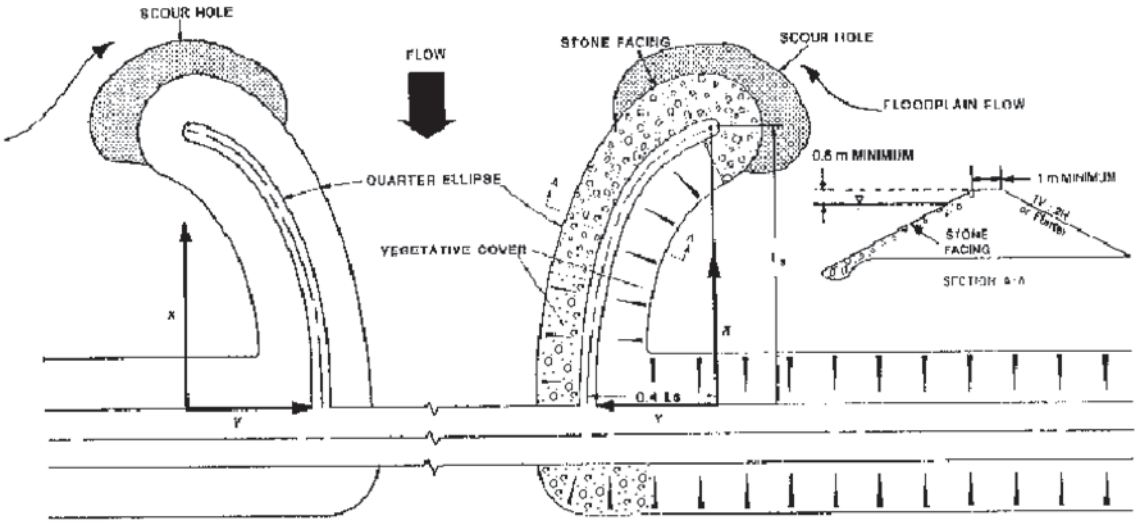


Figure 2.11. Plan view of the extension of rock riprap apron.



Source: modified from Bradley (1978)

Figure 2.12. Typical guide bank.

lengths, thus decreasing the cost of bridge construction and maintenance.

In general, straight spurs should be used for most bank protection. Straight spurs are more easily installed and maintained and require less material. For permeable spurs, the width depends on the type of permeable spur being used. Less permeable retarder/deflector spurs, which consist of a soil or sand embankment, should be straight with a round nose as shown in Figure 2.13.

The top width of embankment spurs should be a minimum of 3 ft (1 m). However, in many cases, the top width will be dictated by the width of any earth-moving equipment used to construct the spur. In general, a top width equal to the width of a dump truck can be used. The side slopes of the spur should be 1V:2H or flatter.

Riprap Sizing and Filter Requirements. Guide banks, spurs, and other river-training countermeasures constructed of soil embankment material must be protected by riprap or other erosion resistant material. In general, revetment riprap design procedures are recommended (see Section 2.4.1) for sizing and filter requirements. However, zones of high shear stress such as the end of a guide bank or spur may require larger rock or additional volume to provide a launching apron. The possibility of overtopping flows must also be considered for many countermeasures.

Riprap Design for Embankment Overtopping. A number of material types and manufactured systems have been identified for use in minimizing or preventing erosion of embankments subjected to overtopping flow. These systems, including riprap, are described in detail in a summary report issued by the ASCE Task Committee on Overtopping Protection (Oswalt et al., 1994).

Traditionally, riprap has been placed on the downstream slope of embankment dams for erosion protection during heavy rainfall and has commonly been assumed inadequate for protection from overtopping flows. Although prototype verification is limited, several investigators have studied riprap stability on steep slopes when subjected to flow. Flow hydraulics on steep embankment slopes cannot be analyzed with standard flow and sediment transport equations. Uniform flow and tractive shear equations do not apply to shallow flow over large roughness elements, highly aerated flow, or chute and pool flow–all of which can occur during overtopping. Riprap design criteria for overtopping protection of embankment dams should prevent stone movement and ensure the riprap layer does not fail. Empirically derived design criteria currently offer the best approach for design (Frizell et al., 1990).

Riprap design to resist overtopping flow is dependent upon the material properties (median size, shape, gradation, porosity, and unit weight), the hydraulic gradient or embankment slope, and the unit discharge. Flume studies were performed to investigate flow through and over rockfill dams, using crushed granite, pebbles, gravel, and cobbles on a range of slopes (Abt et al., 1988, 1991). Threshold flow where incipient stone movement occurs and collapse flow where stone failure occurs were defined. The maximum unit discharge that resists stone movement on steep slopes is a function of the mean water depth, the critical velocity at which the stone begins to move, and an aeration factor defined as the ratio of the specific weight of the air-water mixture to the specific weight of water. A comparison of the various expressions for overtopping flow conditions shows them to be valid for crushed stone with angular shapes (Abt and Johnson, 1991). Knauss developed a rock stability function based on unit discharge, slope, rock packing, and air concentration for sizing riprap, and determined that aeration of flow increases the critical velocity for which riprap on a steep slope remains stable (Oswalt et al., 1994).

Studies were performed in a near-prototype–size embankment overtopping facility to establish new criteria between the design of the riprap layer and the interstitial velocity of water flowing through the riprap layer (Mishra, 1998). An equation was developed to predict the interstitial velocity of water through the rock layer. A universal formula for designing the riprap was derived (see Section 3.5.1). This equation was tested for the data obtained in the 1998 study and previous research studies. The universal riprap design equation was found to satisfactorily predict the size of the riprap to be used for a specified unit discharge and a given embankment slope.

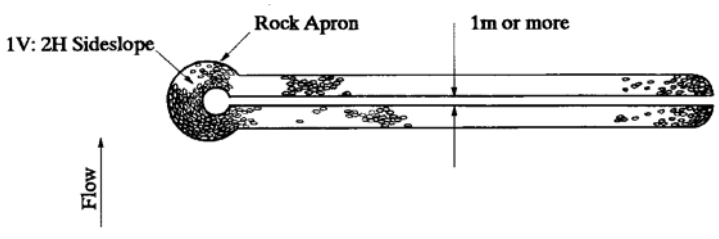


Figure 2.13. Typical straight, round-nose spur.

Material and Testing Specifications

No material or testing specifications specific to countermeasure riprap were found. In general, specifications for revetment riprap will also apply to countermeasure riprap (see Section 2.4.1).

Construction/Installation Guidelines

Riprap on Guide Banks. Riprap should be placed on the stream side face as well as around the end of the guide bank. It is not necessary to riprap the side of the guide bank adjacent to the highway approach embankment (Figure 2.12). A granular or geotextile filter is usually required to protect the underlying embankment material (Lagasse et al., 2001). Riprap should be extended below the bed elevation to a depth equal to or greater than the combined long-term degradation and contraction scour depth, and extend up the face of the guide bank to above the design flow. Additional riprap should be placed around the upstream end of the guide bank.

Riprap on Spurs. Riprap should be placed on the upstream and downstream faces as well as on the nose of the spur to inhibit erosion of the spur (Figure 2.13). Depending on the embankment material being used, a granular or geotextile filter may be required. As with guide banks, it is recommended that riprap be extended below the bed elevation to a depth equal to the combined long-term degradation and contraction scour depth. Riprap should also extend to the crest of the spur, in cases where the spur would be submerged at design flow, or to 2 ft (0.6 m) above the design flow, if the spur crest is higher than the design flow depth. Additional riprap should be placed around the nose of the spur, so that spur will be protected from scour.

Inspection and Quality Control

No inspection or quality control guidelines specific to countermeasure riprap were found. In general, inspection and quality control guidelines for revetment riprap will also apply to countermeasure riprap (see Section 2.4.1).

2.4.5 Riprap Design Software

There is a limited availability of riprap design software, probably because most riprap equations are easy to apply with hand calculations or spreadsheets. The most comprehensive revetment riprap software is Riprap Design System Version 2.0 (West Consultants, 2002). This software computes revetment riprap sizes using seven different methods. The Riprap Design System does not include design of riprap for pier, abutment, or other applications.

Other readily available software packages for revetment riprap include a more limited range of equations. These packages include SAM (Thomas et al., 2002), CHANLPRO (Maynord et al., 1998), and HYCHL (in HYDRAIN Version 6.1; Young et al., 1999). SAM and CHANLPRO perform the calculations based on the EM 1601 procedure. HYCHL performs riprap sizing calculations for channel lining (roadside ditches and natural channels) based on the HEC-11 and HEC-15 procedures. A New York State DOT (NYSDOT) program, STONE3, computes the stability factor based on the HDS 6 procedure. The availability of the NYSDOT software is not known.

One software program (PB_Riprap; Froehlich, 1997) can be used to calculate riprap sizes for revetment (three equations), pier (three equations), abutment (one equation), and propeller applications (one equation). The information provided with this program indicates that it may be freely used and distributed.

Table 2.7. Summary of riprap design software.

Program	Developer	Application		
		Revetment	Pier	Abutment
Riprap Design System V. 2.0	West Consultants	EM 1601 HEC-11 USBR EM-25 USGS WRI 86-4127 CABS Isbash ASCE Manual 54		
HYDRAIN/ HYCHL	FHWA/GKY	HEC-15 HEC-11		
CHANLPRO	USACE	EM 1601		
SAM	USACE	EM 1601		
STONE3	NY State DOT	HDS 6 (SF method)		
PB_Riprap	Froehlich / Parsons Brinckerhoff	Froehlich & Benson HEC-11 Froehlich	Froehlich HEC-18* Breusers & Raudkivi	HEC-18*

*Now in HEC-23

Table 2.7 summarizes the riprap software available. The software programs provide riprap size calculations but do not provide information on other riprap design requirements, such as filter, gradation, or layer thickness. Although not included in this evaluation, several erosion control product manufacturers have produced software that include riprap size estimates for comparison with their products.

The Froehlich (1997) software includes riprap design equations for revetment and piers. The revetment equation appears to provide inconsistent results and the documentation for the pier equation is unavailable. Section 3.9 provides an annotated description of the software listed in Table 2.7 as well as reference data sets for testing software or spreadsheets.

2.4.6 Related NCHRP Studies

NCHRP Project 24-07(2), "Countermeasures to Protect Bridge Piers from Scour"

Research Approach. The study contractor for NCHRP Project 24-07(2) was Ayres Associates Inc, Fort Collins, Colorado. The following summary of the results of testing riprap as a pier scour countermeasure is extracted from the April 2006 Preliminary Draft Final Report for NCHRP Project 24-07(2) (Lagasse et al., 2006).

The objectives of NCHRP Project 24-07(2) were to develop and recommend (1) practical selection criteria for bridge pier scour countermeasures; (2) guidelines and specifications for design and construction; and (3) guidelines for inspection, maintenance, and performance evaluation. The countermeasures considered included riprap, partially grouted riprap, articulating concrete blocks, gabion mattresses, grout-filled mattresses, riprap at skewed piers, mounded riprap, and geotextile sand containers. Project 24-07(2) was an extension of the work conducted by the University of Minnesota on NCHRP Project 24-07(1) (Parker et al., 1998). In addition to providing additional testing for selected pier scour countermeasures, the goal of this project was to develop practical design guidance and specifications for implementation of a variety of pier scour countermeasures in field applications.

The laboratory research was conducted at the Hydraulics Laboratory of Colorado State University (CSU), located at the Engineering Research Center (ERC). The testing was conducted in an indoor laboratory recirculating flume with a large flow capacity. The flume is 8 ft (2.4 m) wide by 4 ft (1.22 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 cubic feet per second (cfs) (2.83 m^3/s) with a series of sediment pumps capable of delivering particle sizes up to 0.5 in (12.7 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 collection. The flume is also equipped with a Plexiglas wall for flow and scour visualization. Figure 2.14 shows a schematic of the flume, data cart, and ancillary components.

To maximize the amount of testing within the available budget, the researchers decided to place three piers along the centerline of the testing flume. Square piers 8 in (0.2 m) long by 8 in (0.2 m) wide were used. Spacing between the piers was approximately 40 ft (12.2 m) to ensure the formation of identical flow lines upstream of each pier. Sand with a d_{50} of approximately 0.6 mm was placed in the flume to a depth of approximately 18 in (0.46 m). The flume layout is indicated in Figure 2.14.

A matrix of flume tests was developed for the research program. Each clear-water test consists 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 $2.5V_{crit}$ and $3.0\ V_{crit}$ with sediment recirculation; therefore, both live-bed and sediment-deficient conditions were examined.

The laboratory tests were not designed to replicate any particular prototype-scale conditions. For example, the $2V_{crit}$ run (using an 8-in [0.2-m] square pier) was not intended to represent a specific scale ratio of a prototype pier or flow condition. However, in each case, the test countermeasure was "designed" to withstand the $2V_{crit}$ hydraulic condition using HEC-18 or HEC-23 guidelines. For example, 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 because of other factors, such as settling, edge undermining, or winnowing of substrate material. Runs utilizing an approach velocity greater than $2V_{crit}$ were intended to take each system to failure by particle dislodgement.

Test Results and Findings. Test results from NCHRP Project 24-07(2) on standard riprap indicated that the stone sizing equation of HEC-23 is appropriate for installations around bridge piers. At piers, best performance was achieved when the riprap was placed with a layer thickness at least three times the median diameter of the stone and was extended at least two times the width of the pier (as measured perpendicular to the approach flow) on all sides.

Both clear-water and live-bed conditions were examined in an 8-ft (2.4 m) wide indoor flume. Under live-bed conditions,

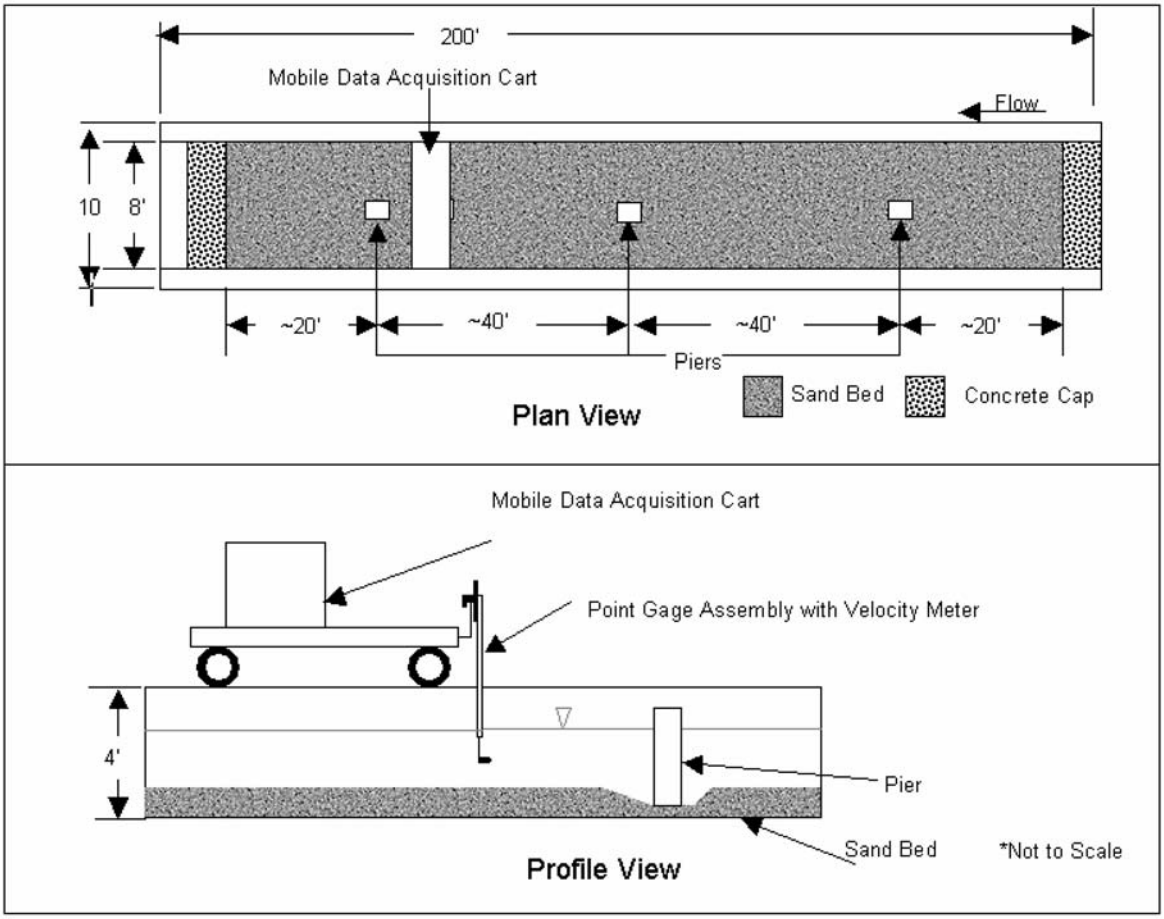


Figure 2.14. Schematic of flume and configuration.

the passage of bed forms increased the amount of scour around the periphery of the riprap installation, which caused rock along the edges to launch into the scoured area. Also under live-bed conditions, poor results were obtained when a granular filter was used, because the filter stone, once exposed by scour at the periphery of the riprap, was rapidly washed away leaving the installation vulnerable to progressive undermining. Good performance under the same conditions was obtained by using a properly selected geotextile as a filter (see Section 3.3.2). The NCHRP Project 24-07(2) study confirmed that the best performance was achieved when the geotextile was not extended out from the pier all the way to the edge of the riprap, but instead extended only about two-thirds that of the riprap, as previously recommended in NCHRP Project 24-07(1) (Parker et al., 1998).

NCHRP Project 24-07(2) also examined the placement of geotextile filter and rock riprap in flowing water under essentially prototype-scale conditions. Sand-filled geocontainers weighing 200 lbs (91 kg) each were dumped into a scour hole around the pier and overlapped to achieve uniform coverage that was snug up against the pier. Rock riprap was then dumped on top of the geocontainers to bring the installation up to or slightly above the ambient bed elevation. Tests were conducted with both loose and partially grouted riprap.

Both installations performed very well under the design conditions. However, when approach velocities exceeded the design conditions by more than about 20%, some particle displacement of the loose riprap installation began to occur. The partially grouted riprap remained stable up to the maximum capacity of the test facility. For further discussion, see Section 3.2.2 of this report. In addition, findings from NCHRP Project 24-07(2) have been incorporated into the design guidelines for bridge pier riprap (see Appendix C).

NCHRP Project 24-18A, "Countermeasures to Protect Bridge Abutments from Scour"

Research Approach. The study contractor for NCHRP Project 24-18A was Michigan Technological University.

Under this project, abutment riprap laboratory testing was done at the University of Auckland, New Zealand.

The objectives of the experiments were to determine the requirements for using riprap, cable-tied blocks, or a combination thereof for protection of bridge abutment structures against scour damage. The expected outcome from the research was to evaluate, develop, and implement guidelines for armoring countermeasures to protect bridge abutments against scour damage. Specifically, the research was to provide appropriate selection, design, and construction guidelines for the use of riprap and cable-tied blocks as abutment scour countermeasures, which practicing engineers in the field could easily understand and use. Issues that needed to be addressed were the applicability, design, construction, maintenance, performance evaluation, environmental effects, reliability, aesthetics, and costs.

The experimental work was conducted in two of the laboratory flumes of the Fluid Mechanics Laboratory of the School of Engineering, University of Auckland. The first flume is 5 ft (1.5 m) wide, 4 ft (1.22 m) deep, and 148 ft (45 m) long. The discharge through the flume is controlled by two pumps, which are capable of recirculating the water in the flume at a combined flow rate in excess of 35 cfs (1 m^3/s). Sediment can be recirculated as a sediment/water slurry. The second flume is 8 ft (2.4 m) wide, 12 in (0.3 m) deep, and about 54 ft (16.5 m) long. Two 6-in (150 mm) diameter pipes and one 8-in (200 mm) diameter pipe supply water to the inlet tank from the laboratory constant head tank. At the end of the flume, a tailgate is used to regulate the flow depth. The discharged water is returned to the laboratory reservoir system, from where it is pumped back to the constant head tank.

The following relevant background literature was reviewed and summarized:

- Sediment transport theory
- Mechanics of scour
- Local flow structure and scour at bridge abutments
- Prediction of local scour depth at bridge abutments

Guidance on the use of riprap and cable-tied blocks as scour countermeasures was examined, along with the recommendations for their use. The relevant experimental studies of abutment scour protection using riprap or cable-tied blocks of Pagán-Ortiz (1991), Atayee (1993), Macky (1986), Croad (1989), Eve (1999), and Hoe (2001) were reviewed.

The two types of bridge abutments investigated were a spill-through abutment situated on the floodplain and a vertical wing-wall abutment situated on the main channel bank. The setup of the spill-through abutment that was constructed in the 8-ft (2.4-m) wide flume is shown in Figures 2.15 and 2.16.

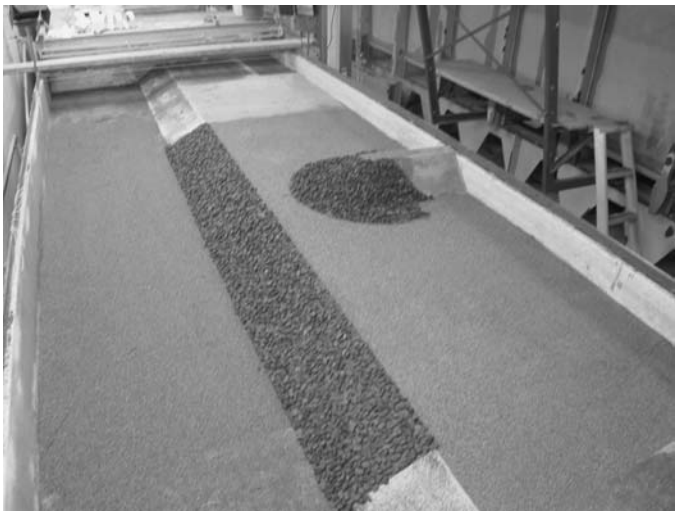


Figure 2.15. Spill-through abutment configuration, 2.6-ft (0.8-m) long abutment on a 4-ft (1.2-m) wide floodplain, NCHRP Project 24-18A.

The setup of the wing-wall abutment that was constructed in the 5-ft (1.5-m) wide flume is shown in Figure 2.17.

A series of riprap countermeasure experiments was conducted in the 8-ft (2.4-m) wide flume to determine the minimum areal riprap placement requirements around the spill-through bridge abutment model situated on the floodplain of a compound channel. The floodplain width and

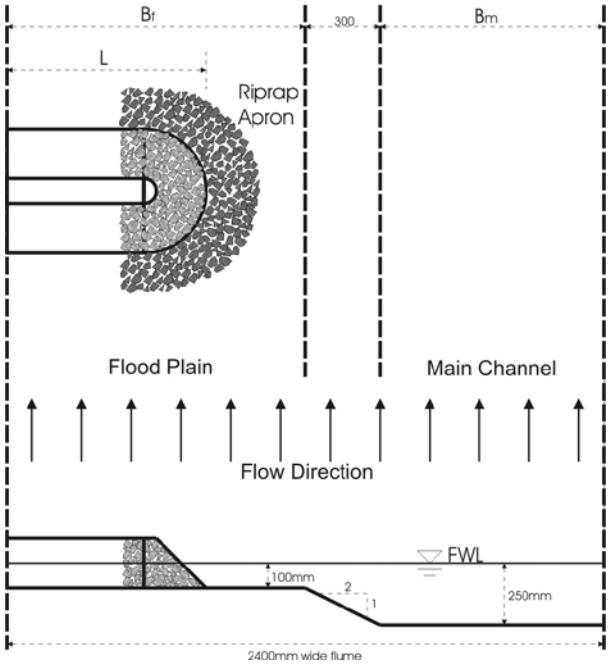


Figure 2.16. Spill-through abutment testing configuration, NCHRP Project 24-18A.

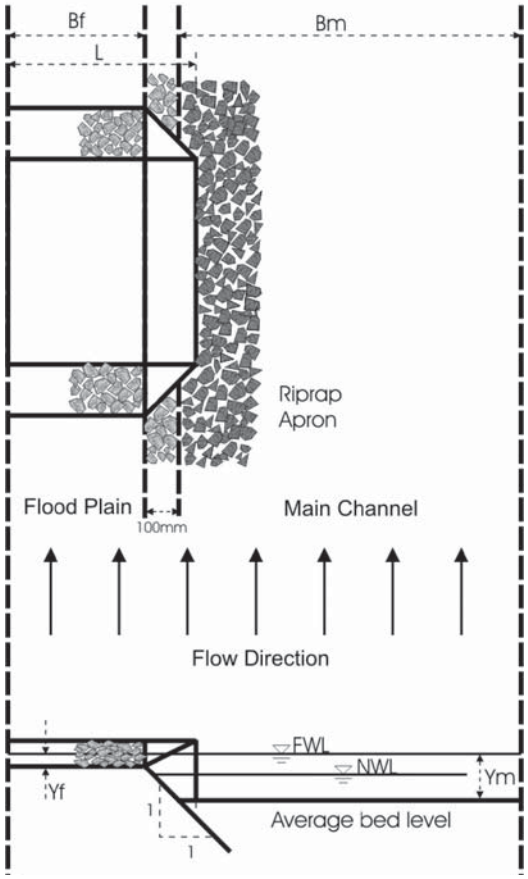


Figure 2.17. Wing-wall abutment testing configuration, NCHRP Project 24-18A.

abutment length were systematically varied for the series of experiments.

Initially, experiments were run with an exposed sediment main channel bank. However, the forced compound channel flow eroded the main channel bank, causing a regression of the floodplain. Therefore, a second set of experiments was run with riprap also placed on the main channel bank to protect the bank from erosion. This method worked well for abutment configurations where the scour hole formed on the floodplain only. An example of this configuration is shown in Figure 2.18.

However, this method did not work well for abutment configurations where the scour hole that formed on the floodplain encroached onto the main channel bank, because the riprap on the main channel bank would fall into the scour hole, inhibiting further scour. An example of this problem is shown in Figure 2.19. For abutment configurations where the abutment spanned the width of the floodplain, the scour hole that would normally form on the main channel bank was completely suppressed by the riprap protection on the main channel bank.

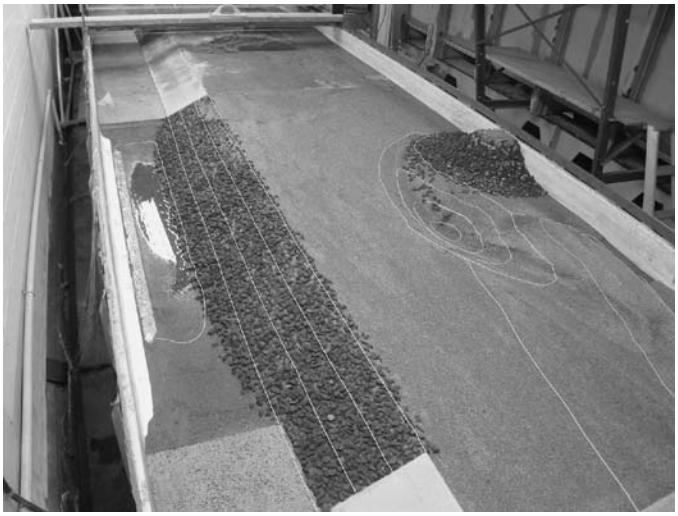


Figure 2.18. Preliminary riprap countermeasure experiment, with the scour taking place on the floodplain only (1.3-ft [0.4-m] long abutment on a 5-ft [1.5-m] wide floodplain), NCHRP Project 24-18A.

The results from the preliminary riprap countermeasure experiments indicate that two scour-failure processes seem significant for abutments.

- One process is attributable primarily to the flow field around the abutment and affects the floodplain bed locally around the abutment. It prevails when the abutment is set back some distance from the bank of the main channel.

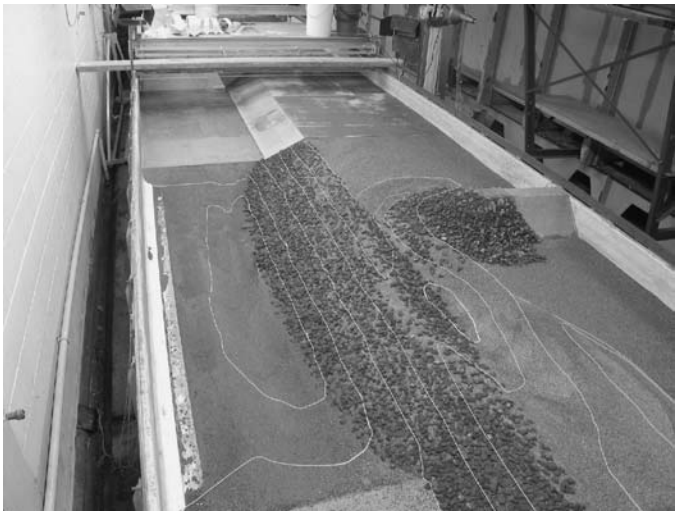


Figure 2.19. Preliminary riprap countermeasure experiment, with the scour hole forming partly over the main channel bank (2.6-ft [0.8-m] long abutment on a 4-ft [1.2-m] wide floodplain), NCHRP Project 24-18A.

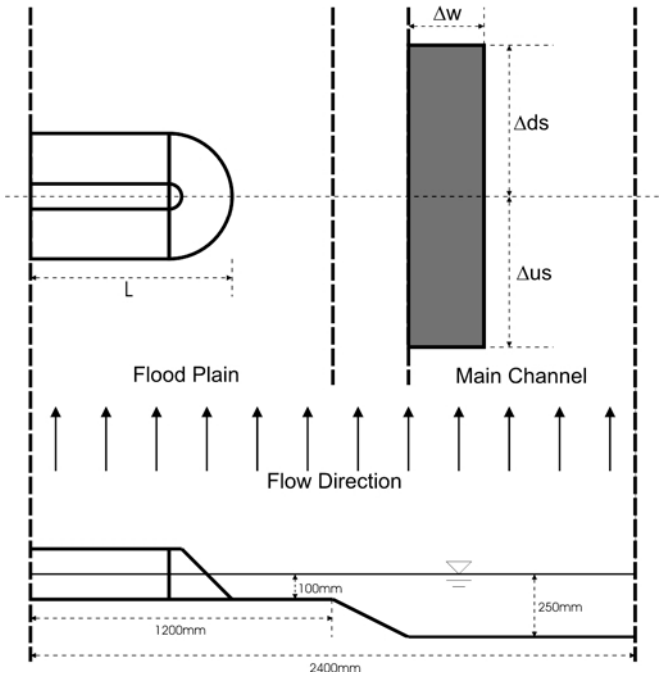


Figure 2.20. Main channel bed riprap experimental schematic, NCHRP Project 24-18A.

- The second process is failure of the bank of the main channel and may occur for various reasons. The abutment becomes endangered when bank failure occurs close to the abutment. Of particular importance for NCHRP Project 24-18A was bank failure caused, or aggravated, by flow around an abutment in proximity to the bank.

A series of experiments was run to determine the upstream and downstream extents of riprap required to stabilize the main channel bed, preventing scour from undermining the channel bank in the vicinity of the abutment (Figure 2.20). The floodplain width was fixed to a width of 4 ft (1.2 m) and the abutment length, apron width, upstream extent, and downstream extent were systematically varied for the experimental series. From this set of experiments, the minimum width and upstream and downstream extents of riprap to be placed on the main channel bed to prevent scour from undermining the channel bank in the vicinity of the abutment were determined.

Testing included erodible floodplain experiments. For cases where the abutment is set back sufficiently on the floodplain (such that the scour hole does not affect the main channel bank), NCHRP Project 24-18A investigated the effect of apron width on the position and size of the scour hole at the abutment, systematically altering apron width, abutment length, and floodplain width. Also, part of the experimental series was repeated with abutments skewed to the flow, to determine the effect of the skew angle on the position and size of the scour hole at the abutment for different apron widths. For cases where the abutment comes right out to the edge of the main channel bank, work was conducted to determine the minimum areal extent of the main channel bed that needs to be protected from local scour to prevent the main channel bank from failing locally at the abutment.

In addition, preliminary scour countermeasure experiments were conducted on a wing-wall abutment (Figure 2.17). A wing-wall abutment model was tested under live-bed conditions at a range of velocities and flow depths to determine the effect of apron width on the position and size of the scour hole at the abutment. The same experimental series was then repeated with different floodplain widths, again to determine the effect of apron width on the position and size of the scour hole at the abutment. The tests also included a "narrow" wing wall under clear-water and live-bed conditions.

Test Results and Findings. Test results for NCHRP Project 24-18A are expected in mid-2006 and were not available to be reported here. Reference to the NCHRP 24-18A final report, when available, is suggested (Barkdoll et al., 2006).

CHAPTER 3

Interpretation, Appraisal, and Applications

3.1 Introduction

This chapter presents an interpretation, appraisal, and applications of the summary of the current state of practice for riprap design, specifications, and quality control in Chapter 2. Design equations for sizing riprap are evaluated with sensitivity analyses using laboratory, and/or field data, where available, for the applications of interest to this study: revetment riprap and riprap for scour protection at bridge piers and bridge abutments, and on flow control countermeasures. Based on the sensitivity analyses, a design equation or design approach is recommended for each application.

Sizing the stone is only the first step in the comprehensive design, production, installation, inspection, and maintenance process required for a successful riprap armoring system. Filter requirements, material and testing specifications, construction and installation guidelines, and inspection and quality control procedures are also considered in this chapter and specific criteria or approaches recommended for each riprap application. Because revetment armoring on streambanks and embankments is by far the most common use of riprap protection, guidelines and specifications for the revetment application are fully developed. For riprap at bridge piers, abutments, and countermeasures, the appraisal emphasizes the differences in the requirements, compared to the revetment application, for a successful armoring system considering the different flow field and hydraulic stresses imposed by each application. For example, a specific design (sizing) equation is recommended for each application and filter requirements are different as well, but most material and testing specifications are common to all applications.

Guidance on determining design variables and design examples are provided for each application. Design of riprap for overtopping flow conditions on roadway embankments and flow control countermeasures such as guide banks and spurs is also considered. An annotated description of riprap design software and reference data sets for testing design software or spreadsheets are included. Riprap failure mechanisms are identified as a basis for developing inspection guidance, and selected case studies of failures are used to emphasize the need for post-flood/post-construction inspection. Finally, concepts (but not design guidance) for a bioengineering or hybrid design approach for bank stabilization using a combination of rock and vegetative treatments are discussed.

To guide the practitioner in developing appropriate riprap designs and ensuring successful installation of riprap armoring systems for bankline revetment, at bridge piers, and at abutments and guide banks, the findings of Chapter 2 and the recommendations of Chapter 3 are combined to provide detailed guidelines in a set of appendixes:

- Appendix C, Guidelines for the Design and Specification of Rock Riprap Installations
- Appendix D, Guidelines for the Construction, Inspection, and Maintenance of Rock Riprap Installations

As appropriate, these guidelines could be considered by AASHTO or state DOTs for adoption and incorporation into manuals, specifications, or other design guidance documents.

3.2 Revetment Riprap

Based on a screening of the many revetment riprap design equations found in the literature (Section 2.4.1), seven are carried forward in this section for a more detailed sensitivity analysis using a field data set. Four of these are selected for additional analyses using a laboratory data set and one, the U.S. Army Corps of Engineers EM 1601 equation, is recommended for streambank revetment design. The recommendation is based on the ability of the basic equation to discriminate between stable and failed riprap, an evaluation of bank and bend correction factors, and the reasonableness of safety/stability factors.

Design requirements and procedures for both geotextile and granular filters are considered in detail and guidance is provided for the full life cycle of a revetment riprap system. Laboratory and field tests for both quality control and inspection and inspection guidance with reference to the requirements of the National Bridge Inspection Standards (NBIS) are provided (U.S.DOT, 2004). A standard riprap gradation specification that considers design, production, and installation requirements is proposed together with a standardized riprap size classification system.

3.2.1 Sensitivity Analysis for Design Equations

As summarized in Section 2.4.1, numerous equations are available to design riprap for embankment protection. The seven equations discussed in this section appear to be more widely used for design than other equations found in the literature. The seven equations are from HEC-11 (Brown and Clyde, 1989), Escarameia and May (1992), Pilarczyk (1990), EM 1601 (USACE, 1991) supplemented by Maynord et al. (1989) and Maynord (1990), Isbash (1935, 1936), CABS (Racin et al., 2000), and HDS 6 (Richardson et al., 2001). Permissible velocity and/or critical tractive force are the approaches commonly used for sizing riprap. Many engineers feel that a tractive force approach to determining riprap size is preferable; but, because of the difficulty of determining the velocity at the bed or on the slope of a channel, some form of velocity—either mean or depth-averaged—is more often used to determine riprap size.

Two of the equations (Pilarczyk and HDS 6) require iterative solutions because a specific flow velocity can produce a range of shear stresses (tractive force) depending on the size of the riprap (roughness of the surface). While iterative solutions may be theoretically sound, the application of an iterative approach in practice can be more difficult. An iterative solution of the Pilarczyk equation can be avoided with some rearrangement of the K_h term.

Of the seven equations considered, four include flow depth as a variable (HEC-11, Pilarczyk, HDS 6, and EM 1601 [Maynord]). Although flow depth should be a factor for bank revetment, it should be a relatively small factor. In both the Pilarczyk and EM 1601 (Maynord) equations, riprap size is proportional to flow depth to the -0.25 power. In the HDS 6 equation, depth is a minor factor except for large riprap sizes relative to the flow depth. Although not immediately evident in the standard presentation of Pilarczyk's equation, riprap size is proportional to velocity to the 2.5 power (like Maynord's [EM 1601] equation).

Therefore, the seven equations can be divided into three groups. The first group includes Isbash, Escarameia-May, and CABS equations, and is of the Isbash form where d_{50} is proportional to velocity squared and not a function of depth ($d_{50} \propto y^0V^2$). The second group includes the EM 1601 (Maynord) and Pilarczyk equations where riprap size is proportional to velocity to the 2.5 power and inversely proportional to depth to the 0.25 power ($d_{50} \propto y^{-0.25}V^{2.5}$). The third group includes HEC-11 and the stability factor approach presented in HDS 6 where riprap size is proportional to velocity cubed or greater. The HEC-11 equation includes velocity cubed and d_{50} is inversely proportional to the square root of depth. The HDS 6 equation is more complex in that the size computation is iterative. The result is that riprap size is proportional to velocity cubed or even a greater power and inversely proportional to the square root of depth or greater power, depending on the relative roughness (d_{50}/y) of the revetment.

The similarity of the HEC-11 and HDS 6 equations is expected because Manning's n is similar when computed by the Strickler equation or by using a relative roughness equation when relative roughness is small. However, when relative roughness is large, the Strickler equation significantly underestimates Manning's n. For low relative roughness conditions, the HEC-11 and HDS 6 equations produce riprap sizes proportional to velocity cubed and inversely proportional to the square root of depth ($d_{50} \propto y^{-0.5}V^3$). In addition to hydraulic variables (velocity and depth), the other differences between riprap sizing equations are factors added to account for turbulence, bank angle, bend radius of curvature, stability factors, rock density, and rock angularity.

In this section, a sensitivity analysis of these seven equations is presented as a two-step process. First, all seven equations are compared to a field data set compiled by Blodgett and McConaughy (1986). Second, based on this screening, four equations are carried forward for a more detailed sensitivity analysis using a laboratory data set compiled by Maynord (1987).

Revetment Riprap Sensitivity Analysis – Field Data

Using data for three sites reported by Blodgett and McConaughy (1986), Table 3.1 shows a comparison of riprap size equations. These three sites were selected for a sensitivity analysis because particle erosion was identified as the failure mechanism. The data for the three sites are based on field measurements taken after the event; so, there are some inconsistencies within the data. Using the equations, for Site 1 and 2, the Escarameia-May, Pilarczyk, and EM 1601 (Maynord) equations resulted in riprap size significantly larger than the failed size, while the other equations resulted in sizes ranging from smaller to slightly larger than the failed size. For Site 3, all of the equations resulted in sizes smaller than the failed size.

According to Blodgett and McConaughy (1986), velocity may have been underestimated at Site 3. Table 3.1 shows the average velocity from the study and a computed average velocity based on the Manning equation and using the water surface slope as the energy slope. For each of the sites, the computed velocity is much higher than the reported velocity,

Table 3.1. Comparison of riprap equations using site data.

Parameter	Site 1 Pinole Creek	Site 2 Sacramento River	Site 3 Truckee River	Site 3* Truckee River
Velocity avg (ft/s)	7.7	6.7	5.2	8.0*
Computed Velocity avg (ft/s)	10.5	7.9	11.2	11.2
Depth avg (ft)	4.9	20.2	10.5	10.5
Depth toe (ft)	7.7	13.0	17.5	17.5
W (ft)	60	723	135	135
Bend Rc (ft)	150	4280	646	646
Rc/W	2.5	5.9	4.8	4.8
Side Slope (xH:1V)	2.0	2.0	1.8	1.8
Riprap S_g	2.65	2.60	2.68	2.68
Water Surface Slope	0.0054	0.00056	0.0030	0.0030
Manning's n	0.030	0.033	0.035	0.035
Failed d_{50} (ft)	0.60	0.51	0.71	0.71
HEC-11 d_{50} (ft)	0.66	0.23	0.15	0.55
Escarameia-May d_{50} (ft)	1.16	0.80	0.47	1.12
Pilarczyk d_{50} (ft)	0.91	0.81	0.41	1.21
EM 1601 (Maynord) d_{50} (ft)	1.55	0.67	0.37	1.09
Isbash d_{50} (ft)	0.39	0.30	0.17	0.41
CABS d_{50} (ft)	0.73	0.57	0.34	0.81
HDS 6 (SF) d_{50} (ft)	0.41	0.23	0.16	0.55

*Site 3 adjusted velocity

and, for Site 3, the computed velocity is more than twice the reported velocity. Because the water surface slope may be steeper than the energy slope (although the report uses the water surface slope to estimate shear stress), Site 3 data were used with an increased velocity (8.0 ft/s [2.4 m/s] as compared with 5.2 ft/s [1.6 m/s]) in the equations.

As with Sites 1 and 2 and the adjusted velocity, the same three equations produced sizes significantly larger than the failed size and the other equations produced riprap sizes ranging from smaller to slightly larger than the failed size. The CABS equation produces riprap sizes slightly larger than the failed size (10% to 20% larger). Figure 3.1 shows the size comparison graphically. It is assumed that sites that plot below or slightly above the failed riprap size would also have failed and that sizes that plot more than 25% above the failed riprap size may not have failed.

Each of these riprap failure sites is on a tight bend as shown by the bend radius divided by width (Rc/W). Site 1 has the tightest bend with Rc/W of 2.5 and the other sites have Rc/W approximately twice this value, although still quite severe. From the velocity, depth, and bend curvature, Site 1 appears to need the largest riprap. This presumption is based on Site 1 having a high average velocity, the lowest average depth, and the lowest Rc/W. The only equation that gives a significantly larger riprap size for this site is the EM 1601 (Maynord) equation, because the EM 1601 (Maynord) equation is quite sensitive to bend radius. Most of the other equations have factors related to bend curvature, but the EM 1601 (Maynord) equation has vertical and longitudinal velocity adjustment factors that are functions based on Rc/W.

Three different sensitivity plots were developed to compare the seven equations. Each of the plots shows an analysis for bank revetment on a 2H:1V side slope using angular riprap with a specific gravity of 2.65. Selection of stability factors, safety factors, and turbulence intensity was based on the individual equation guidance. The first plot (Figure 3.2) holds depth constant at 10 ft (3 m) and varies average velocity from 5 to 15 ft/s (1.5 to 4.5 m/s) for a mild-curvature bend (Rc/W equal to 20). This plot shows that for low velocities, the riprap sizes vary by less than 0.5 ft (0.15 m), but that the variation in predicted size increases to 1 ft (0.3 m) or more for velocities greater than 11 ft/s (3.4 m/s). The HDS 6 equation is the most sensitive to change in velocity. The second plot (Figure 3.3) is also for the same mild-curvature bend but holds velocity constant at 10 ft/s (3 m/s) and varies depth from 5 to 25 ft (1.5 to 7.6 m). Several equations are not related to depth. The HDS 6 factor of safety equation is the most sensitive to depth because of relative roughness effects.

Because the three riprap failure sites identified by Blodgett and McConaughy (1986) were on tight bends, the riprap size versus velocity plot (Figure 3.4) was also developed for a severe-curvature bend (Rc/W = 5). For this condition, there is significantly greater variation in the results of the equations. In Figure 3.4, the EM 1601 (Maynord) and Pilarczyk equations produced similar results. However, had Rc/W been different, slightly higher or lower, the EM 1601 (Maynord) equation would have changed while the Pilarczyk equation would not. This difference is because the EM 1601 (Maynord) equation computes a design velocity based on the average velocity and a function of Rc/W, whereas the Pilarczyk

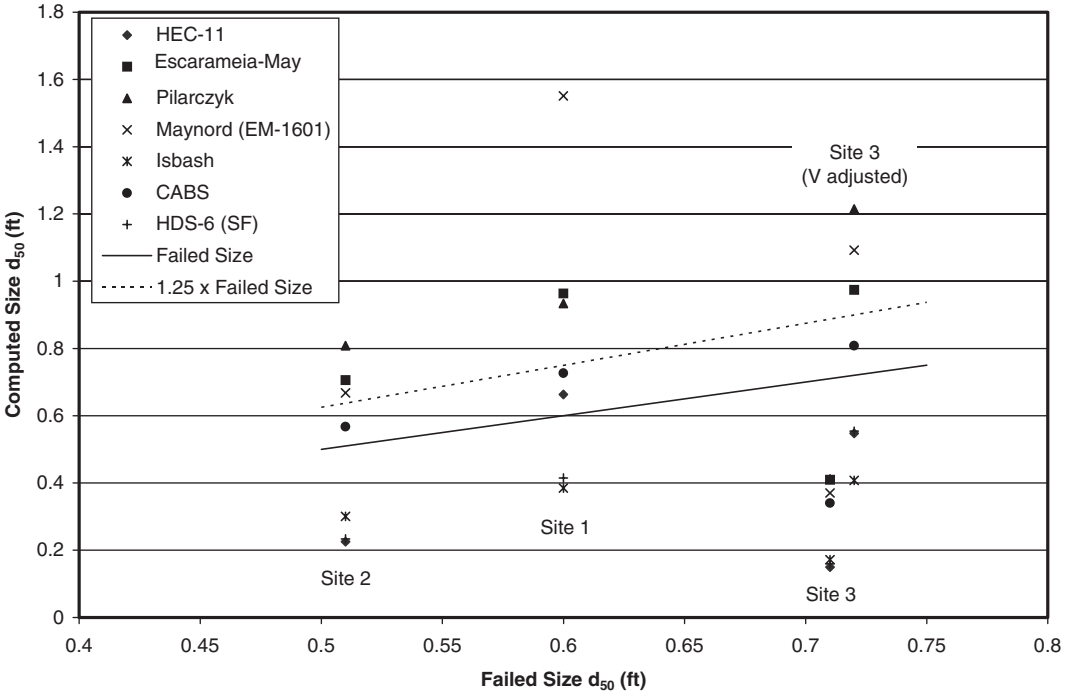


Figure 3.1. Comparison of riprap sizes.

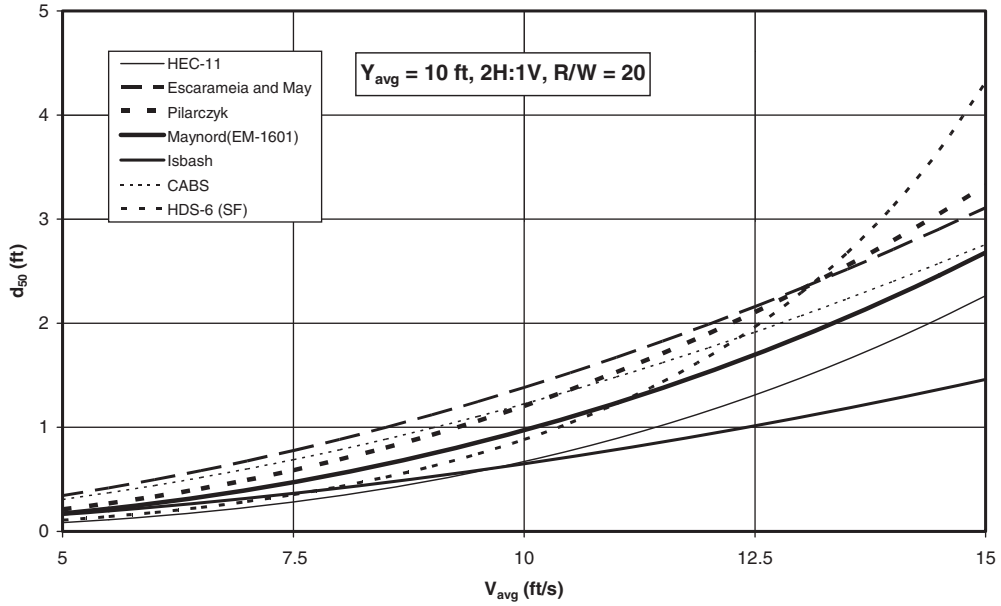


Figure 3.2. Riprap size versus velocity for mild-curvature bend.

equation has a factor (K_T) that takes on a value of 1.5 for sharp outer bends.

Based on this review, the sensitivity analysis, and comparisons with the Blodgett and McConaughy (1986) data, the EM 1601 (Maynord) approach for riprap sizing appears to be the most comprehensive. The EM 1601 (Maynord) equation includes depth as a variable, which, for revetment riprap, should be a factor. The equation is not overly sensitive to velocity and was developed using near-prototype–scale data. Another advantage to the EM 1601 (Maynord) equation is that bend curvature is included as a direct adjustment on velocity rather than a judgment factor selected based on minimal guidance. For the three Blodgett and McConaughy data sites, the EM 1601 (Maynord) equation produced sizes significantly larger than the failed size and produced the largest size for the site with the most severe combination of velocity, depth, and bend curvature.

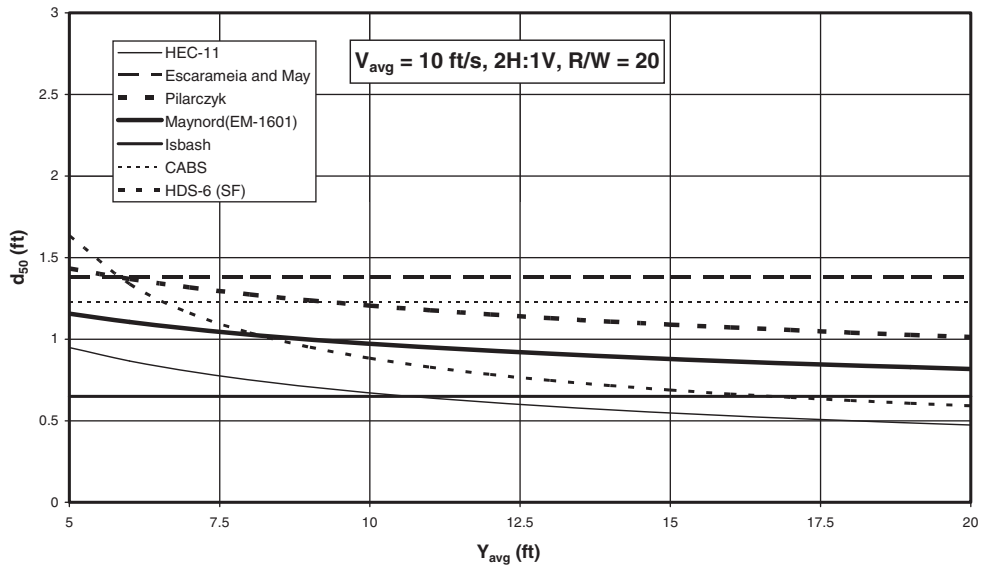


Figure 3.3. Riprap size versus depth for mild-curvature bend.

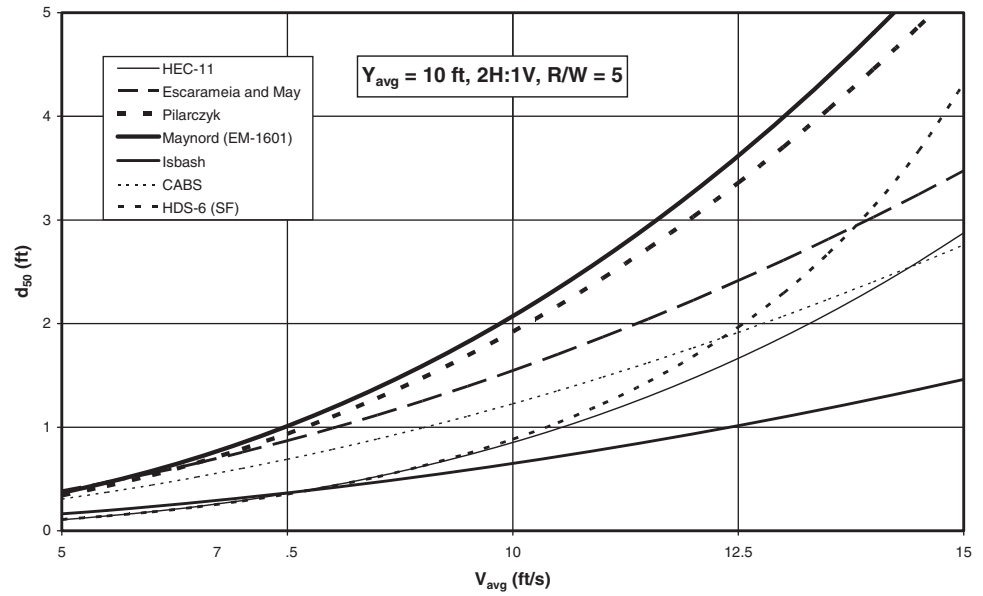


Figure 3.4. Riprap size versus velocity for severe-curvature bend.

In addition to riprap size, gradation, and layer thickness, there are other important factors in revetment design. Standard practice tends to favor a graded riprap with a layer thickness at least 1.5 times the d_{50} stone size and a granular or geotextile filter. The minimum layer thickness is required to accommodate the maximum stone size, provides sufficient depth for riprap interlocking, and limits the penetration of high flow velocities into the riprap voids.

The CABS layered RSP design provides an alternative approach using a more uniform rock placed in multiple layers of decreasing size from the large outer riprap layer to a smaller inner riprap layer(s), backing layer, and RSP fabric. **In the layered approach, the particles of the outer layer are sized for the hydraulic loading produced by either wave or current attack**. Depending on the outer layer size, inner riprap layers and backing layer are prescribed based on filtration requirements. The RSP fabric, therefore, is not a filter fabric, but is selected with sufficient strength and permeability to provide separation between the backing layer and the substrate. Both the 1960 and 2000 versions of the Bank and Shore Protection manual (State of California DPW, 1960; Racin et al., 2000) indicate that the layers provide filtration and require that relatively uniform material be used in each layer. One difference between the original and recent versions of the manual is that

the original manual indicates that voids in the outer layer should be filled with smaller rocks. The recent version indicates that this practice is no longer recommended.

The two approaches (graded versus layered) should achieve the same results with riprap sized to withstand the hydraulic loading. In the standard approach, the filter fabric provides filtration and separation and a bedding layer is often provided to support the riprap and protect the fabric. In the CABS approach, filtration is provided by the multiple inner layers and backing layer, and separation is provided by the RSP fabric. An advantage of the CABS approach is that smaller rocks are not present on the surface and, therefore, are not subject to removal by the hydraulic loading. However, none of the inner layers can be excluded or the revetment will not have adequate filtration. One disadvantage of the CABS approach is that it results in a thicker overall installation and higher cost for materials and construction. Another disadvantage is that if permeability and strength are the only RSP fabric requirements and substrate materials are not considered, the fabric could still clog, or fines could migrate out of the substrate even though the fabric meets CABS guidance.

Based on this screening and comparison with a field data set, four equations were selected for further analysis. The CABS equation is representative of the Isbash-type equations and is supported by detailed design and installation guidance (Racin et al., 2000). The EM 1601 (Maynord) equation appears to be the most comprehensive of all the equations analyzed. The HEC-11 approach is the most frequently used equation for revetment riprap design as reported by the DOTs (see Appendix B), and HDS 6 is representative of a safety factor approach for riprap design.

Revetment Riprap Sensitivity Analysis – Laboratory Data

Basic Form of Riprap Equations. Each of the equations (CABS, EM 1601 [Maynord], HEC-11, and HDS 6) was reduced to its basic form by removing correction factors related to bank side slope, bend radius, and safety/stability and by converting each equation into a consistent dimensionless form. Each of the equations includes the dimensionless parameter of particle size divided by flow depth as the dependent variable and the independent variable is the dimensionless parameter $V/[(S_g-1)gy]^{0.5}$, which is the Froude number divided by the square root of the submerged particle specific gravity. The resulting equations are valid for computing riprap size on a flat channel bed in a straight channel for incipient motion conditions. The riprap equations differ only in the coefficient and exponent applied to the independent variable, except that the HDS 6 equation includes a log term representing the effect of relative roughness of the riprap surface. Because this term is related to the ratio of riprap size to flow depth, it is part of the dependent variable.

The EM 1601 (Maynord) equation was converted to its basic form by removing all correction factors including safety factor (SF = 1), layer thickness factor ($C_T = 1$), vertical velocity distribution coefficient ($C_V = 1$), bank angle factor ($K_1 = 1$), and channel bend correction factor ($V_{ss}/V = 1$). Both the vertical velocity distribution coefficient and the channel bend correction factor are functions of the ratio of bend radius to channel width. For angular riprap, the basic EM 1601 (Maynord) equation is

$$\frac{d_{30}}{y} = 0.30\left[\frac{V}{\sqrt{(S_g-1)gy}}\right]^{2.5} \tag{3.1}$$

The HEC-11 equation was converted to its basic form by removing all correction factors including stability factor (SF = 1) and bank angle factor ($K_1 = 1$). The HEC-11 equation incorporates the bend correction factor into the stability factor as a function of the ratio of bend radius to channel width. The basic HEC-11 equation is

$$\frac{d_{50}}{y} = 0.295\left[\frac{V}{\sqrt{(S_g-1)gy}}\right]^{3} \tag{3.2}$$

Because the CABS equation is used to compute minimum particle weight, the equation was assumed to be for a safety factor of 1.0. To convert the equation from a weight to a nominal size, the shape midway between a sphere and a cube was used. Although the equation includes factors of 2/3 and 4/3 for aligned flow and impinging flow, this factor was not included. If the 2/3 factor were included, the coefficient would be 0.117 rather than 0.263. Because the overall CABS approach uses a relatively uniform gradation, the percent finer (or coarser) of the computed size is not specified. The equation was also solved for a bank angle of zero degrees. The basic CABS equation is

$$\frac{d_n}{y} = 0.263\left[\frac{V}{\sqrt{(S_g-1)gy}}\right]^{2} \tag{3.3}$$

The HDS 6 equation was converted into its basic form by setting the bank angle at zero and the stability factor equal to 1.0. As presented in HDS 6, there is no explicit bend correction. From the specified hydraulic condition, the particle size is determined iteratively since d_{50}/y is included directly and within a log term. The basic HDS 6 equation is

$$\frac{d_{50}}{y}\left[\ln\left(\frac{12.3y}{d_{50}}\right)\right]^{2} = 3.48\left[\frac{V}{\sqrt{(S_g-1)gy}}\right]^{2} \tag{3.4}$$

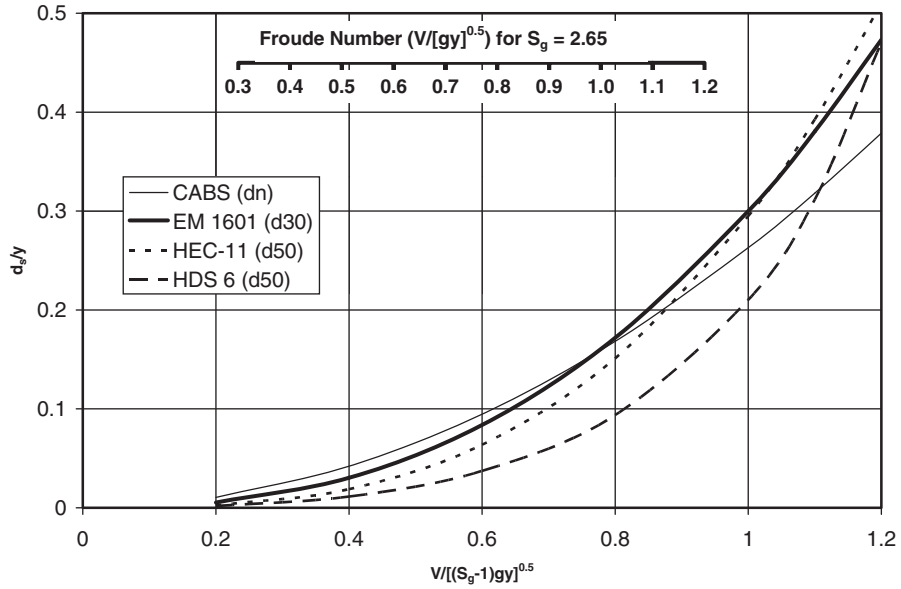


Figure 3.5. Comparison of four basic riprap size equations.

The four equations include the same dependent and independent variables. The differences are the coefficient and exponent applied to the independent variable. Figure 3.5 shows these four equations plotted for comparison. Also shown in Figure 3.5 is the Froude number for a specific gravity of 2.65. The HEC-11 and HDS 6 equations yield the smallest size, especially considering that they compute a d_{50} size as compared with EM 1601 (Maynord), which computes a d_{30} size. For typical design conditions, where a natural channel Froude number ranges from 0.5 to 0.9, the CABS and EM 1601 (Maynord) equations are the most conservative.

Comparison with Laboratory Data. Maynord (1987) ran a series of flume tests with riprap on the bed of straight flumes with vertical side walls. His results included the flow depth and velocity and the riprap specific gravity and size distribution. These data were used to test the basic equations. An equation would have to perform well with this simple case before application of correction factors for bank slope, bend curvature, or safety factor is reasonable. Figures 3.6 through 3.9 show these data plotted with the basic equations. In each of these figures, failed riprap tests are plotted with filled symbols and stable riprap tests are plotted with open symbols. If the equation

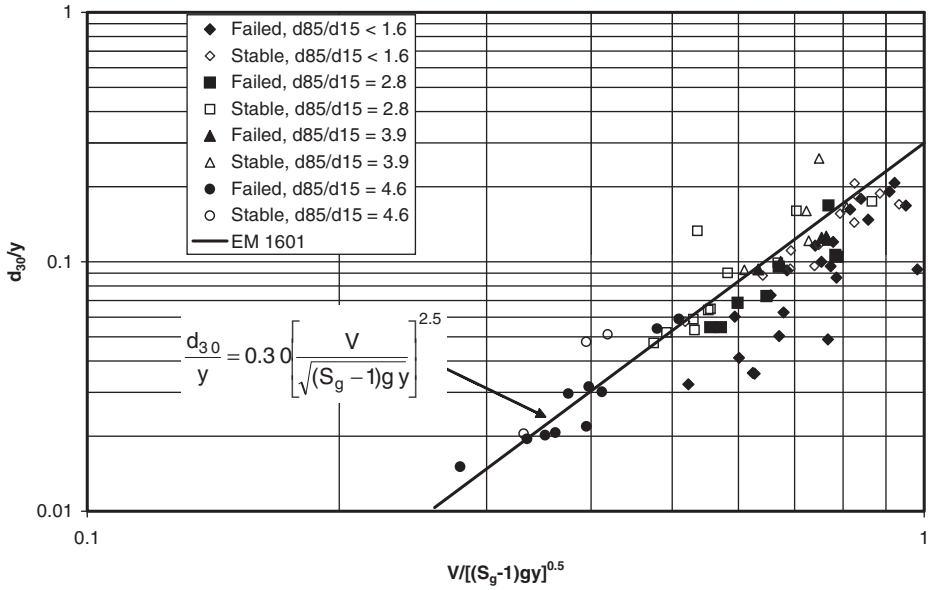


Figure 3.6. Maynord data plotted with EM 1601 equation.

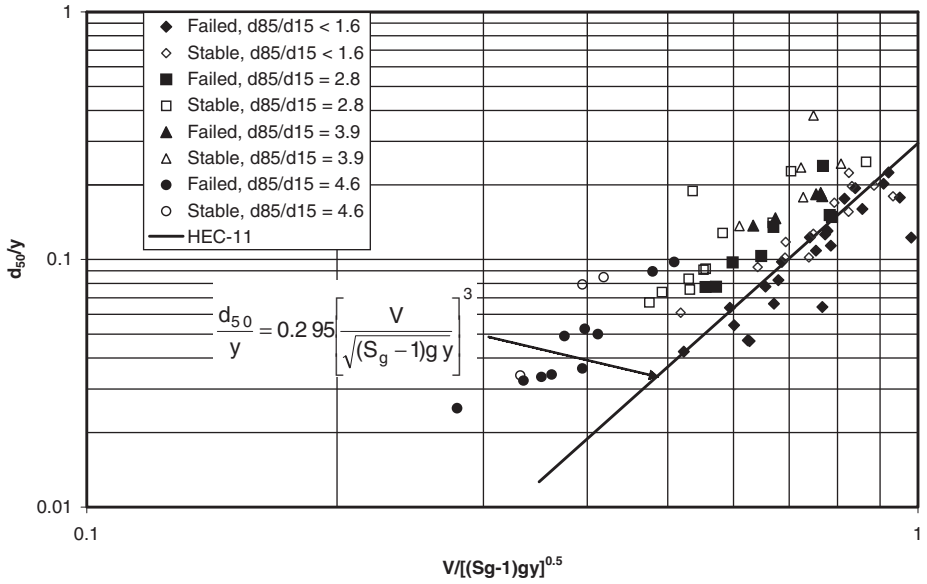


Figure 3.7. Maynord data plotted with HEC-11 equation.

were a perfect predictor of riprap stability, all the failed data would plot below the equation and all the stable data would plot above it. The gradation of the riprap ranged by a factor of 3.7 as represented by d_{85}/d_{15} from less than 1.6 (1.24 to 1.56), which is very uniform, up to 4.6, which is quite well graded.

For EM 1601 (Figure 3.6), the equation envelopes the majority of the failed riprap data and appears to have an appropriate slope and coefficient for these data—as expected, because it includes the data Maynord (1987) used to develop the equation. Because a number of stable riprap points plot below the curve, this equation probably includes some degree of a factor of safety in the basic formulation. It is important to note that the runs where riprap failed did not necessarily represent an incipient motion condition for the riprap, but may have greatly exceeded the threshold hydraulic condition for *initiation* of riprap movement.

The HEC-11 equation (Figure 3.7) does not compare well with the Maynord laboratory data. The only portion of the data that the equation works well for is the uniform gradation (d_{85}/d_{15} <1.6), and the data that deviate the most from the equation are for the most well-graded rock. This result supports Maynord's observation that riprap stability is dictated primarily by the smaller sizes. However, in the derivation of the HEC-11 equation, d_{50} is incorporated in two ways, as the

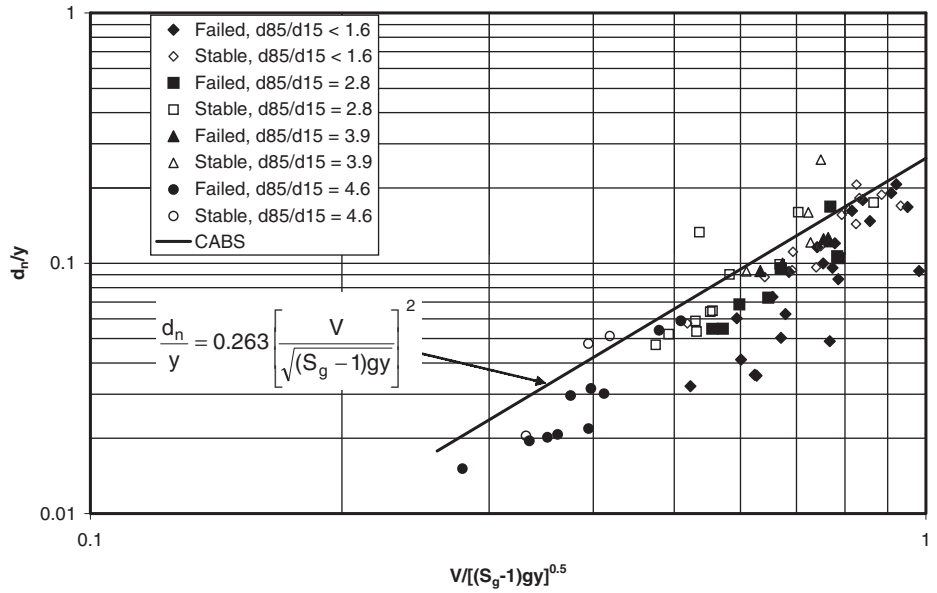


Figure 3.8. Maynord data plotted with the CABS equation.

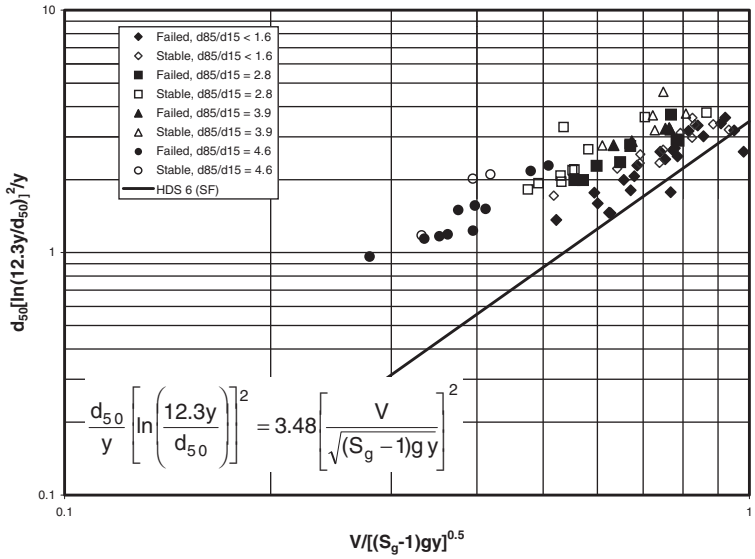


Figure 3.9. Maynord data plotted with the HDS 6 equation.

dependent variable and in the estimation of Manning roughness coefficient. If riprap stability is based on the smaller particles in the distribution (such as d_{30}) and hydraulic roughness is based on the larger sizes in the distribution (such as d_{50} or larger), then the equation should include two sizes to appropriately compute the riprap size. In any case, the HEC-11 equation does not appear to work well for the simple case of riprap stability on a flat channel bed in a straight channel.

The CABS equation was plotted (Figure 3.8) for the Maynord data using d_{30}/y. Although this equation is intended for use in predicting a uniform stone size, the data reflects stone gradations ranging from uniform to well graded. The d_{30} size was selected because, according to the CABS manual, the equation produces the "theoretical minimum size or weight" to resist the hydraulic forces and because the CABS approach uses a relatively uniform gradation. Therefore, the d_{30} size was considered as the most appropriate size for comparison with the equation. Had the d_{50} size been used for this comparison, approximately half of the failed data points would have plotted above the equation line.

Although the equation envelopes the failed riprap data, most of the stable riprap data points also plot below the curve. This result indicates that the equation is conservative in predicting the d_{30} size. It appears that the exponent of 2 is not high enough for these data.

The HDS 6 equation (Figure 3.9) was the least reliable when compared with the Maynord data. Nearly all the data plot above the equation indicating that the equation would predict that all the tested conditions would be stable. The difference between the HDS 6 equation and the other three equations is that it includes a relative roughness term. Although apparently this term is theoretically sound, the equation does not compare well with the laboratory data.

In summary, based on the Maynord laboratory data, the two equations that include the flow resistance of the riprap (HDS 6 and HEC-11) were the least reliable for predicting stable riprap size. Although none of the equations were able to completely discriminate between the stable and failed riprap data, the EM 1601 equation performed the best and the CABS equation performed well when compared with the d_{30} size.

Correction Factors

The basic riprap size equations (Equations 3.1 through 3.4) are valid for determining the incipient riprap size for lining the bed of a straight channel. The other factors that affect riprap size are adjustments for bank angle, bend hydraulics, and stability/safety factors. The sensitivity of riprap size based on changing each of these factors was determined for each of the equations.

Safety/Stability Factor. Figure 3.10 shows the change in riprap size based on varying the safety/stability factor for the equation. For EM 1601, the safety factor is directly applied to the riprap size, therefore an SF of 1.5 results in an increase in size of 50%. CABS (Racin et al., 2000) indicates that making a design more conservative should occur as a final step by selecting the rock weight greater than the computed weight; so, the safety factor was assumed to be applied to weight. Therefore, for CABS, particle size is proportional to the 1/3 power of the safety factor. In HEC-11, the rock size is proportional to the stability factor to the 1.5 power. In the HDS 6 equation, the stability factor and bank angle correction factor appear in the same term. Therefore, the sensitivity of riprap size to the Stability Factor is contingent on first selecting a bank angle.

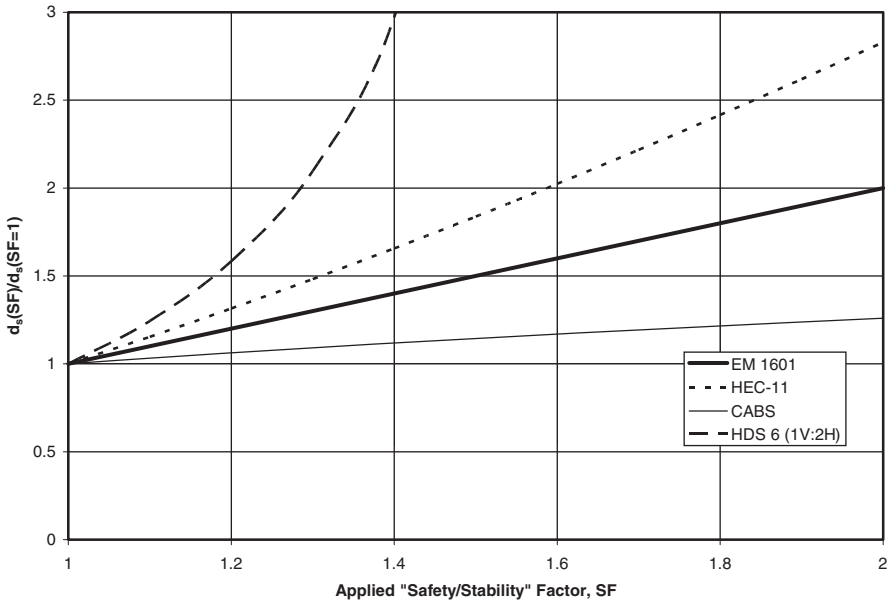


Figure 3.10. Sensitivity of riprap size to safety/stability factor.

Figure 3.10 shows the HDS 6 results for a typical bank slope of 1V:2H. The HDS 6 stability factor is the ratio of the resisting moment to the overturning moment for a particle on a specific slope. Although the definition is very specific, there is no guidance on the selection of appropriate levels of the stability factor for HDS 6. Clearly, selecting a factor of 1.5 has a very different meaning for each of the equations and results in 14%, 50%, 84%, and 380% increases in particle size for CABS, EM 1601, HEC-11 and HDS 6, respectively. A 1.5 factor would increase particle weight by 1.5, 3.4, 6.2, and 111 times the base particle weight for these equations.

Although each of these approaches to incorporating a safety factor is valid, the effect of applying this factor of safety can be related to uncertainty in the dominant variable, which is velocity. Applying a factor of safety of 1.5 by increasing particle weight by 50% in the CABS approach provides for uncertainty in velocity of only 7%. Applying a 1.5 factor of safety by increasing particle size by 50% in the EM 1601 approach provides for uncertainty in velocity of 18%. Applying a stability factor of 1.5 to the HEC-11 equation provides for uncertainty in velocity of 22%. If a stability factor (ratio of resisting to overturning moments) of 1.5 is applied to the HDS 6 equation, it would provide uncertainty in velocity of 119%. Conversely, if a 10% uncertainty in velocity were included, then the required stability/safety factor would be 1.77 for CABS (increase in weight by 77% and/or increase in size by 21%), 1.27 for EM 1601 (increase in size by 27%), 1.21 for HEC-11 (increase in size by 33%) and 1.088 for HDS 6 (increase in size by 21%). Rather than applying any of these safety factors, it may well be a better approach to assign a level of uncertainty to the velocity to compute the required riprap size.

Bend Curvature Correction Factor. Another correction factor is used for bend flow hydraulic conditions due to the increase in velocity and shear stress on the outer bank. Figure 3.11 shows the effect on riprap size for various ratios of bend radius to channel width (R_c/W). HEC-11 uses the stability factor as a correction factor for bend flow conditions as well as debris and ice impact, wave action, and uncertainty in the design parameters. HEC-11 recommends a stability factor of 1.7 for bends tighter than R_c/W of 10, a factor of 1.2 for bends more gradual than R_c/W of 30, and factors ranging from 1.6 to 1.3 between these limits. The stability factor was assumed to vary linearly between these limits. The CABS manual recommends adjusting the average channel velocity by a factor of 2/3 for parallel flow and 4/3 for impinging flow, although there is no guidance on distinguishing between these conditions. For Figure 3.11, bends tighter than Rc/W of 10 were assumed to impinge flow. EM 1601 provides equations for adjusting riprap size for bend flow conditions. The two terms that are included in EM 1601 for bend flow are a vertical velocity correction factor and an adjustment to the velocity. The vertical velocity factor accounts for the downward component of velocity along the outer bank and the velocity adjustment is different for natural channel and trapezoidal channels. An unexpected result of the EM 1601 approach is that for gradual bends, the riprap size may actually be reduced since the correction factor is less than 1.0.

The HDS 6 equation does not specify a bend correction factor; so, any adjustment to velocity or shear stress is up to the engineer. Other FWHA guidance found in HEC-15 (Chen and Cotton, 1988) includes a correction for shear stress as a function of R_c/W. This factor is shown in Figure 3.11 for

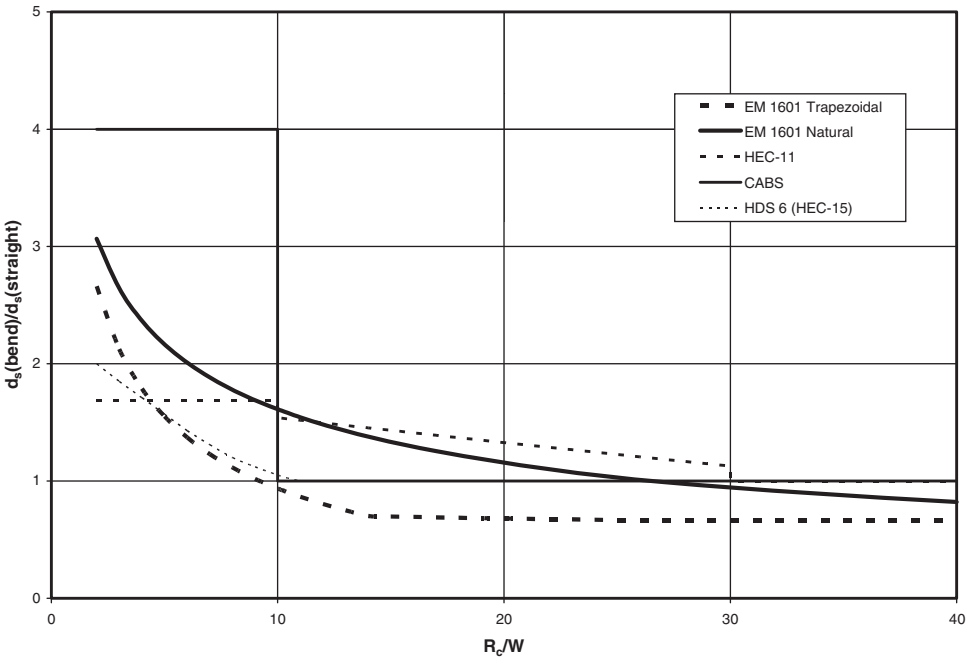


Figure 3.11. Sensitivity of riprap size to bend curvature.

HDS 6. It is similar to the EM 1601 adjustment for trapezoidal channels. Of these relationships, the EM 1601 approach is the most appealing because it is a continuous function and because separate curves are available for trapezoidal and natural channels.

Bank Slope Correction Factor. There is a surprising degree of difference between the side slope correction factors for these four equations (Figure 3.12). The EM 1601 equation has no correction for side slopes flatter than 1V:4H. The other equations all recommend increasing the stone size by 13% for this flat slope. For banks steeper than 1V:2.75H, the HEC-11 and HDS 6 equations require significant increases to riprap size. The EM 1601 relationship is based on laboratory data in which the incipient velocity was measured with all other conditions kept equal. Measurements were obtained for the flume bed and varying bank angles up to 1V:1.25H and the correction factor was obtained as $(V_{bank}/V_{bed})^2$. There was no

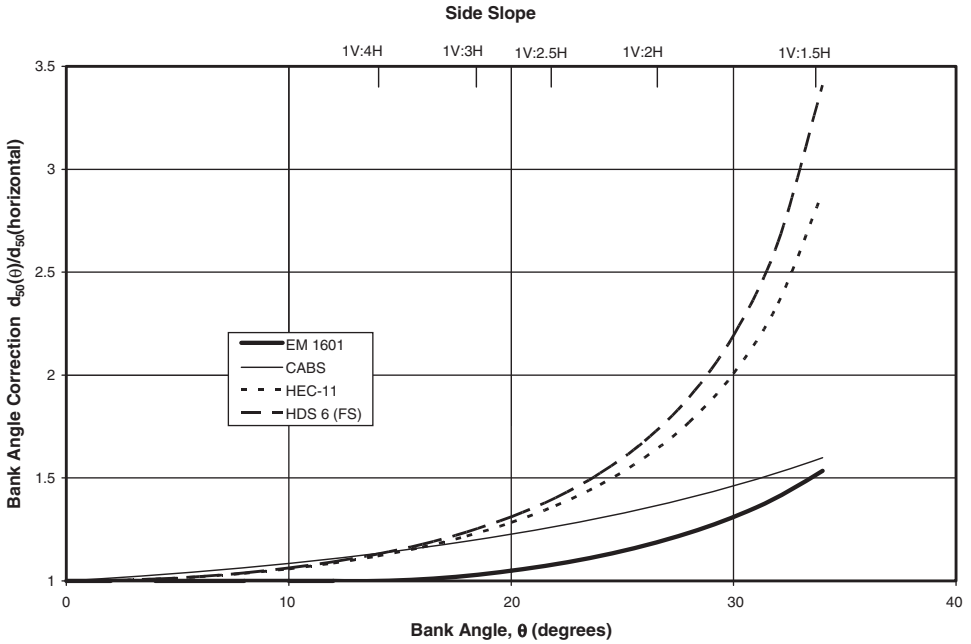


Figure 3.12. Sensitivity of riprap size to bank angle.

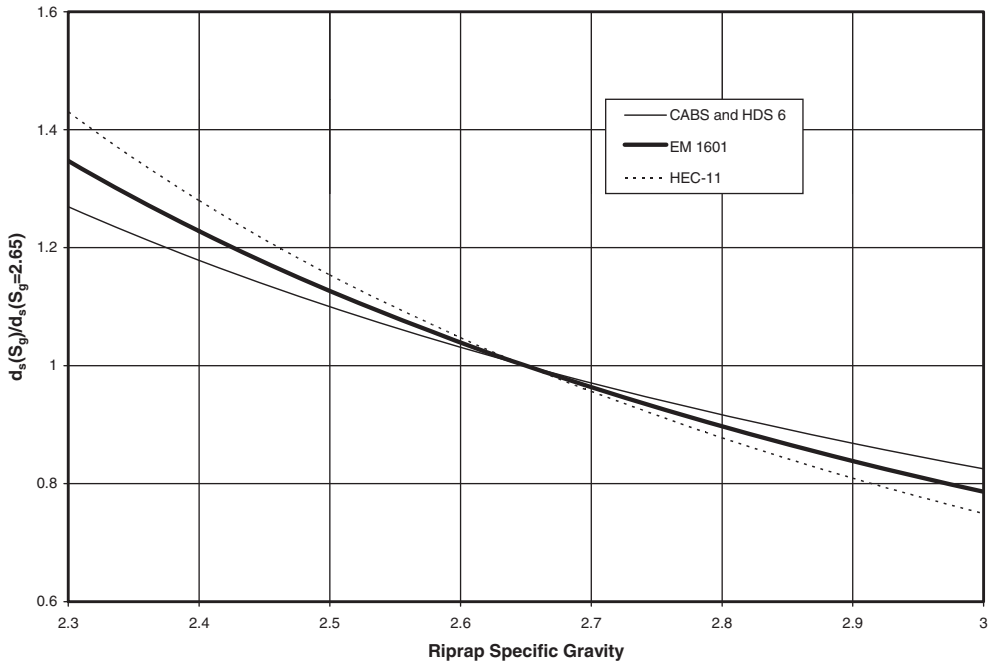


Figure 3.13. Sensitivity of riprap size to riprap specific gravity.

difference in the incipient velocity for the 1V:4H slope and a minor difference for the 1V:3H slope. However, if the required riprap size is proportional to velocity to the 2.5 power, as is used in the EM 1601 equation, then the correction factor developed from the laboratory data should have been computed based on $(V_{bank}/V_{bed})^{2.5}$. This adjustment would shift the EM 1601 line up slightly. The small influence of bank angle on riprap size is discussed by Maynord (1987) and he refers to a personal communication by M.A. Stevens, who states that bank angle should not be a major factor. Although the CABS equation is very different in form from the other equations, it very nearly matches the HEC-11 and HDS 6 equations for slopes up to 1V:3.5H and approaches the EM 1601 results for 1V:1.5H, which is the recommended upper limit for EM 1601 and CABS and is a practical upper limit for HEC-11 and HDS 6.

Specific Gravity Correction Factor. Each of the basic equations includes the rock specific gravity as part of the independent variable for riprap stability on a channel bed. Because the exponent in these equations ranges from 2.0 to 3.0, the required riprap size from these equations differs

Table 3.2. Revetment riprap equation ratings.

Equation	Basic Eq. Test Data	Bend Correction	Bank Angle Correction	Safety Factor	Recommended	Comments
CABS	G	P	G	P		Lacks guidance on bend and safety factor selection.
EM 1601	V	E	G	G	✓	Bank angle correction very minor.
HEC-11	F	F	F	G		Very sensitive to bank angle correction.
HDS 6	P	P	F	P		Lacks guidance on bend and stability factor selection and very sensitive to bank angle and stability factor.

Ratings

E Excellent
V Very Good
G Good
F Fair
P Poor

slightly as specific gravity is varied. Figure 3.13 shows that the CABS and HDS 6 equations are the least sensitive to riprap specific gravity and the HEC-11 equation is the most sensitive. In Figure 3.13, a specific gravity of 2.65 was selected as the basis for comparison. Even though the HDS 6 and HEC-11 equations are derived analytically based on hydraulic processes and the assumptions within the derivations are defendable, this basic factor is not consistent between these approaches.

Summary

As a summary of the results of this review, Table 3.2 provides subjective ratings from "poor" to "excellent" on each equation related to the laboratory data and each of the correction factors. In comparison with the laboratory data, no equation was rated "excellent" because even the EM 1601 equation did not completely separate the failed and stable data. The only excellent rating given in the correction factors was the EM 1601 approach to the bend flow factor. This approach was clearly the most comprehensive. It appears that the bank angle correction incorporated into the HEC-11 and HDS 6 equations are overly conservative, but there may be deficiencies with the approaches in CABS and EM 1601 as well. Finally, there is no clear guidance on selecting safety/stability factors for CABS and HDS 6. Overall, this factor is not well described for any of the equations. **Given the results of the sensitivity analysis, the EM 1601 equation is recommended for bank revetment design as it ranked as high as or higher than the other equations in all of the test categories**.

3.2.2 Filter Requirements

Filter Design

Considering current FHWA and AASHTO guidance, filter design criteria are the most overlooked aspect of revetment riprap design. More 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. Figure 3.14(a) and (b) illustrate the basic difference between stable and unstable soil structures.

Figure 3.14(c) through (f) illustrate several common filtering processes that can occur in stable and unstable base soils (modified from Geosyntec Consultants, 1991). The large arrows indicate the direction of water flow in the base soil. In Figure 3.14(c), 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 3.14(d), 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 fines from being moved by the forces of seepage flow and turbulence at the interface.

In Figure 3.14(e), 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 condition contrasts with the condition shown in Figure 3.14(f), 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 3.14, 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 3.15(a) 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 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.

Base Soil Properties

Base soil is defined here as the subgrade material upon which the riprap and filter will be placed. Base soil can be native in-place material or imported and recompacted 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).

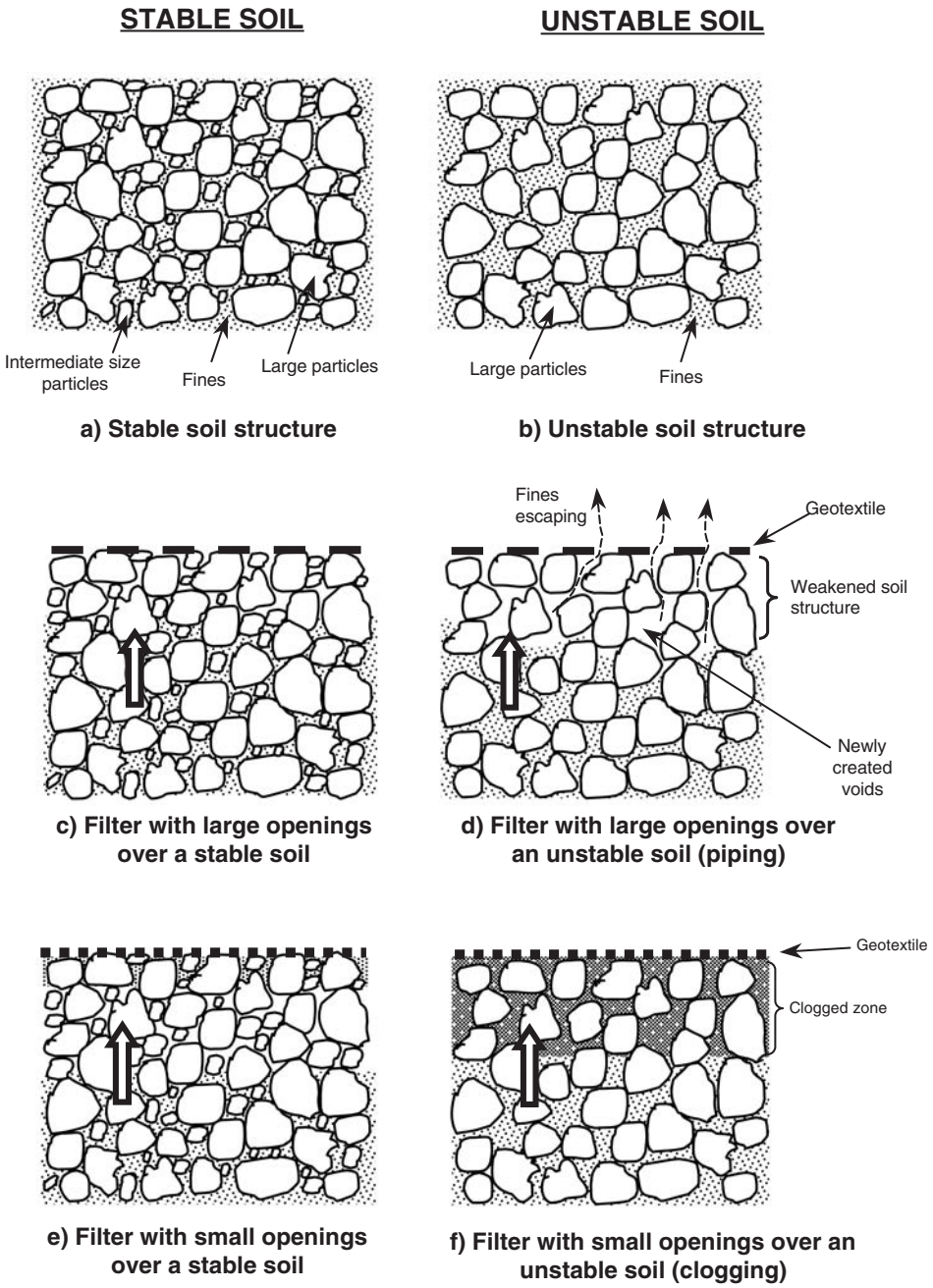


Figure 3.14. Examples of soil and filter compatibility processes.

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

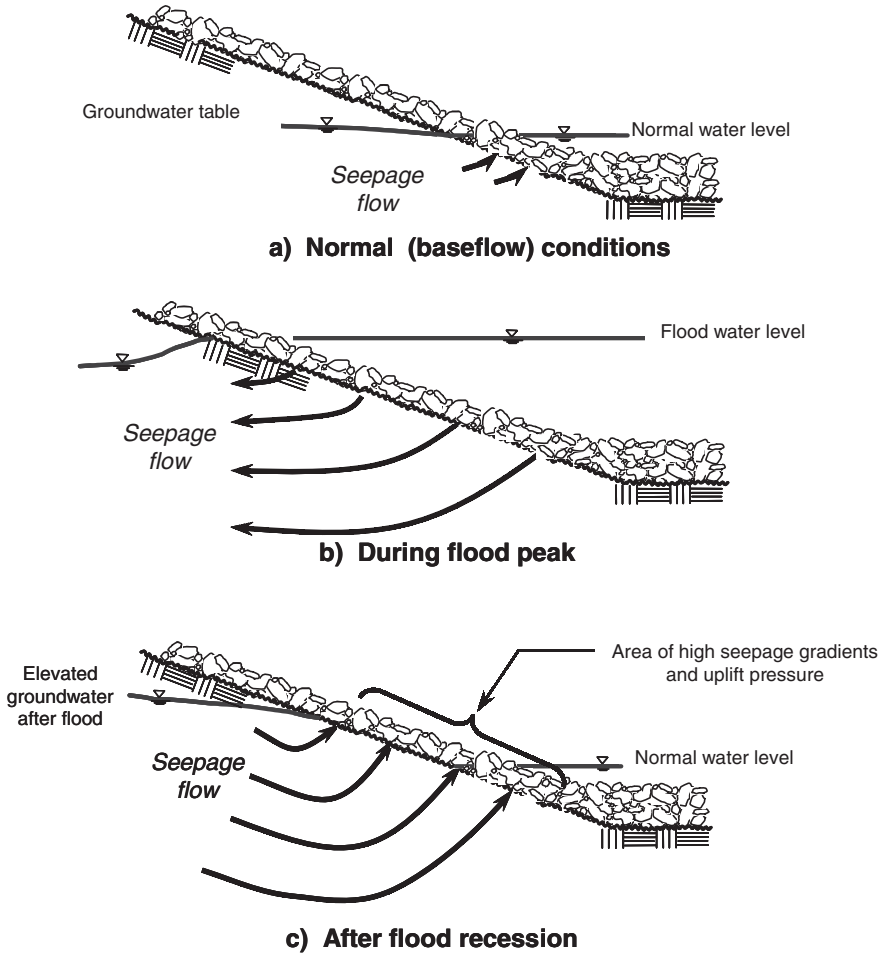


Figure 3.15. Changes in water levels and seepage patterns during a flood.

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 3.3 lists average values of porosity and permeability for alluvial soils.

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.

Table 3.3. 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)

These values are available from manufacturers. 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 riprap, these stresses do not represent the particular application environment that the geotextile will experience. While geotextiles have various properties, only those deemed most relevant to applications involving riprap installation are discussed in the following paragraphs. More information regarding standard material specifications and test methods for determining geotextile properties is provided in Section 3.2.3.

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.

Transmissivity. The transmissivity, ψ, 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 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 microstructure. Transmissivity is not particularly relevant to riprap 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. Force required to initiate a tear in the fabric when pulled in tension. 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. Force required to propagate a tear once initiated. Typically reported in Newtons or pounds.

Puncture Strength. Force required to puncture a geotextile using a standard penetration apparatus. Typically reported in Newtons or pounds.

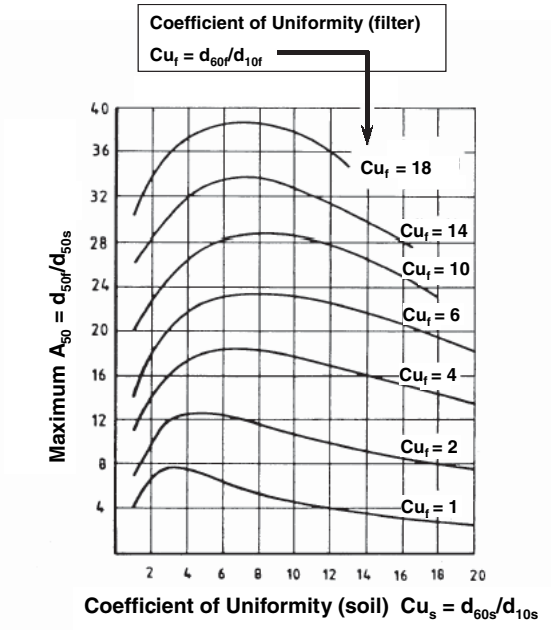
Granular Filter Properties

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. Additional information regarding standard material specifications and test methods for determining the physical characteristics of aggregates is provided in Section 3.2.3.

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 3.16 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 riprap 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,



Source: Heibaum (2004)

Figure 3.16. Filter design chart according to Cistin–Ziems.

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%.

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

Geotextile Filter Design Procedure

The suggested steps for proper design of a geotextile filter are outlined in the following paragraphs.

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 PI (required only if the base soil is more than 20% clay).

Step 2. Determine Particle Retention Criterion. A decision tree is provided as Figure 3.17 to assist in determining the appropriate soil retention criterion for the geotextile. The figure has been modified to include 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 having relatively low cohesion 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.

Note: If the required AOS is smaller than that of available geotextiles, then a granular transition layer is required, even if the base soil is not clay. 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:

$K < 1 \times 10^{-7}$ cm/s
$PI > 15$
$c > 10$ 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 Geotextile 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, up to 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.

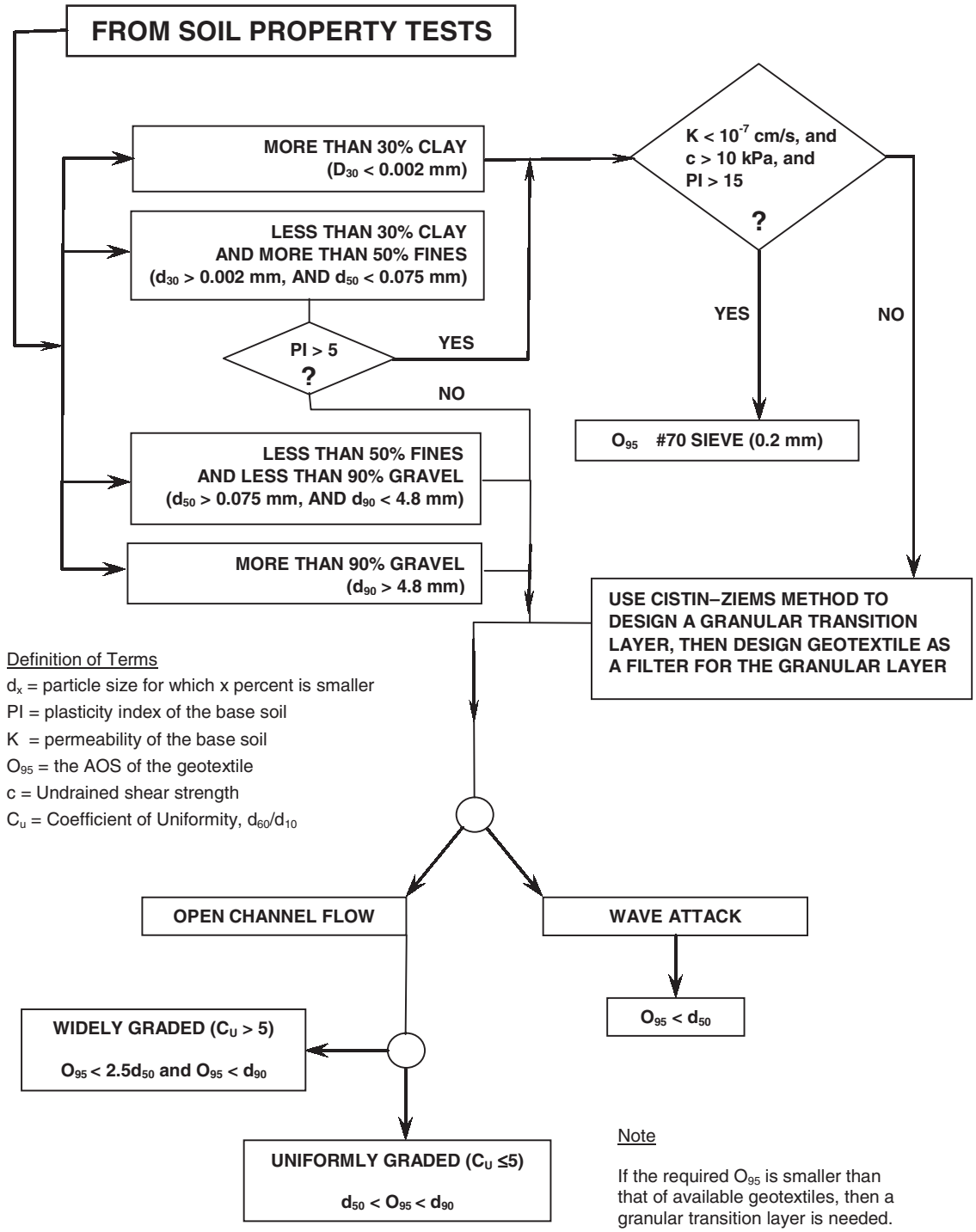
To obtain the permeability of a geotextile in cm/s, multiply the thickness of the geotextile in cm by its permittivity in s^{-1}. Typically, the designer will need to contact the geotextile manufacturer to obtain values of permittivity and thickness.

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. AASHTO M-288, "Geotextile Specification for Highway Applications," 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).

Granular Filter Design Procedure

Numerous texts and handbooks provide details on the well-known Terzaghi approach to designing a granular filter.



Source: modified from Koerner (1998)

Figure 3.17. Geotextile selection for soil retention.

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 in the following paragraphs. Note that d_s is used to represent the base (finer) soil, and df 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 PI (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 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 d_{60}/d_{10} should be determined for each candidate material.

Step 4. Determine Maximum Allowable d_{50f} for Filter. Enter the Cistin–Ziems design chart (Figure 3.16) with the Coefficient of Uniformity for the base soil on the x-axis. Find the curve that corresponds to the Coefficient of Uniformity 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 $d_{50f(max)} = A_{50max}$ 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 Compatibility with Riprap. Repeat Steps 1 through 4, considering that the filter material is now the "finer" soil and the rock riprap is the "coarser" material. If the Cistin–Ziems criterion is not met, then multiple layers of granular filter materials should be considered.

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). Single-layer thicknesses up to 15 in (38 cm) may be warranted where large riprap 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 (HEC-11 [Brown and Clyde, 1989]).

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) 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 3.18 shows a close-up photo of the SandMat™ material. Figure 3.19 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

Source: Colcrete–Von Essen Inc.

Figure 3.18. Close-up photo of SandMat™ geocomposite blanket, NCHRP Project 24-07(2).

Source: Colcrete–Von Essen Inc.

Figure 3.19. SandMat™ geocomposite blanket being unrolled, NCHRP Project 24-07(2).

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 armoring material (typically riprap, or partially grouted riprap).

Figure 3.20 shows a geotextile container being filled with sand. Figure 3.21 shows the sand-filled geocontainer being 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/m^2. 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 ft^2 (0.56 to 0.74 m^2) can be fashioned from non-woven, needle-punched geotextile having a minimum mass per unit area of 200 g/m^2, filled at the job site, and field-stitched with a hand-held machine. Figures 3.22 and 3.23 illustrate the smaller geocontainers being installed at a prototype-scale test installation for NCHRP Project 24-07(2) in a pier scour countermeasure application (for more detail see Section 2.4.6 and also Lagasse et al. [2006]).

Bearing Capacity

Geotextiles are often used to improve the bearing capacity of weak, compressible, and often-saturated soils for purposes of improving roadways and other vehicular access points. It stands to reason that the bearing capacity of weak soils can also be improved by the use of geotextiles to withstand loading by heavy rock riprap.

Source: Colcrete–Von Essen Inc.

Figure 3.20. Filling geocontainer with sand, NCHRP Project 24-07(2).

Source: Colcrete–Von Essen Inc.

Figure 3.21. Handling a 1-tonne sand-filled geocontainer, NCHRP Project 24-07(2).

a. Demonstrating puncture resistance of geocontainers

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

Figure 3.22. Small (200-lb [90.7-kg]) sand-filled geocontainers, NCHRP Project 24-07(2).

In essence, bearing capacity relies upon the ability of a soil (or reinforced-soil) substrate to effectively spread a loading from a relatively small point to a larger area, such that any potential deformation of the soil surface is counteracted by lateral and vertical forces that are mobilized in the substrate.

Improvements in bearing capacity, ranging from about 100% for loose sands to more than 700% for soft clay-like silts, using one layer of geotextile have been reported (Koerner, 1998). In the reported studies, the difference in bearing capacity was quantified using the settlement ratio ρ/B (settled distance divided by footing width) as a function of applied load, compared to a non-reinforced control. Use of multiple geotextile layers, with a specified vertical spacing, increased the bearing capacity in all cases.

Koerner identified four distinct modes of failure when using a geotextile to improve bearing capacity:

- **Excessive depth of geotextile:** Geotextile is placed deeper than about 1 ft (0.3 m) below the soil surface. Failure takes place in the soil above the geotextile.
- **Insufficient embedment length:** Geotextile does not extend far enough beyond the load point to mobilize sufficient frictional resistance against slippage.
- **Tensile failure of geotextile:** Geotextile is not strong enough to resist tensile forces without excessive elongation or outright tearing.
- **Excessive long-term (creep) settlement:** Geotextile is vulnerable to long-term, sustained forces that result in

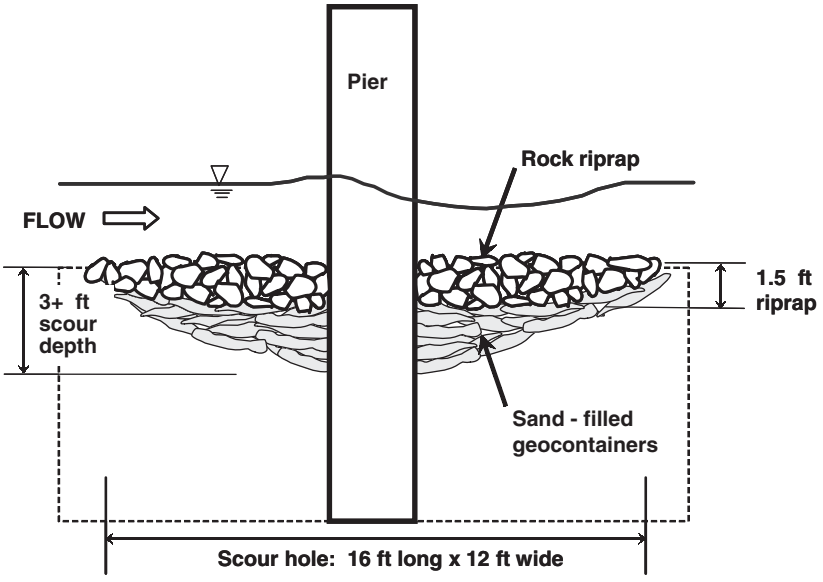


Figure 3.23. Schematic diagram of sand-filled geocontainers beneath riprap armor, NCHRP Project 24-07(2).

gradual overextension and, thus, undesirable settlement at the load point.

USACE Special Report 99-7 (Henry, 1999) provides in-depth background regarding the issue of soil bearing capacity, albeit in the context of vehicular wheel loadings on unpaved roadways. Primarily a geotechnical study, this document nonetheless provides some valuable information regarding the effect of geotextiles in improving the quality of subgrade bearing capacity, particularly with respect to load redistribution.

The design curves provided in USACE Special Report 99-7 relate the required road base aggregate thickness to the undrained shear strength of the subsoil, with and without a geotextile. In all cases, the use of a geotextile provides a significant reduction in the required amount of road base aggregate to effectively resist deformation by wheeled vehicles. Geotextile strength and elongation specifications are also provided, using existing ASTM testing standards.

The geotechnical stability analysis methodologies presented in the previously mentioned references are beyond the scope of this NCHRP report. However, it can be concluded that the use of geotextiles beneath a riprap armor layer will provide additional support to the bearing capacity of the underlying subsoils. The use of multiple layers of geotextiles, each separated by 6 to 12 in (0.15 to 0.3 m) of compatible soil or suitable granular material, will serve to increase the bearing capacity to resist either static loading from rock riprap or dynamic loading from wheeled (or tracked) maintenance vehicles. Geotextiles are often supplemented with a geogrid when bearing capacity is a significant design consideration.

3.2.3 Material and Testing Specifications

Overview

Currently, material and testing specifications for riprap available in the United States (e.g., AASHTO, ASTM) are generally adequate for determining riprap quality. However, there is little consistency in specifications for riprap gradation properties. For example, many gradation specifications can be interpreted to result in an essentially uniform rock size where a more widely graded mixture was intended by the designer. In addition, the wide variety of size designations (classes) among agencies results in confusion and, potentially, increased project cost. In this section, a methodology is developed that considers both the rock size and slope of the riprap particle distribution curve, as well as typical rock production methods.

The survey (Appendix B) indicates that very little field testing during construction or inspection is done on a programmatic basis. A simple methodology developed by the Office of Surface Mining is presented in this section to facilitate a decision to accept or reject a rock product at the quarry or on site, and a "pebble count" approach for verifying size distribution is described. In addition, standard laboratory material and testing specifications (see Tables 2.3 and 2.4) are reviewed and adapted to the riprap application.

Gradation Specifications

General. Gradation specifications for riprap prescribe a range of allowable sizes for a given riprap class. Sizes can be defined by weight or by a length dimension. Practical specification guidance must allow producers to supply rock with a range of sizes that allows reasonable, but not excessive, deviation from the "ideal" particle size distribution curve. The underlying principle in this regard is to achieve economy through standardization without sacrificing hydraulic stability. From this perspective, the specification should result in a matrix of rocks that has a majority of particles that are equal to, or larger than, the size required for stability at the design hydraulic loading. A certain amount of particles that are smaller than the stable size can be tolerated, but in much smaller proportion.

A specification that allows an excessive amount of undersized stones can result in failure by particle displacement. On the other hand, a specification that requires a large proportion of particles significantly greater than the stable stone size will result in unnecessarily high cost, both for the material itself, and for the transportation and placement of that material. Thus, there is a very real need to strike a balance between "too many small particles" on the one hand, versus "too many large particles" on the other.

In the current state of practice, many guidelines exist for specifying the allowable particle size distribution of rock riprap. Some guidelines are "loose," allowing a large range of sizes compared to the size required for stability. Others are "restrictive," requiring very tight control on the range of allowable sizes.

Many existing gradation specifications have been built around several "classes" of rock size in order to achieve economy through standardization. The rock size is designated either by a specified dimension or by a specified weight; however, neither by itself is sufficient. Typically, a minimum specific gravity requirement is included with a size category, whether that category is designated by dimension or by weight. Clearly, the specific gravity is required to convert from one system to the other.

Given a minimum allowable specific gravity, many specification methods convert between dimension and weight by assuming a particle shape halfway between a cube and a sphere. The EM 1601 (USACE, 1991) method recommends conversion based on a sphere. The relationships between the representative dimension, d, and weight, W, are summarized below:

- Sphere

$$W = \gamma_s\left(\pi d^3/6\right) = 0.52\left(d^3\gamma_s\right) \qquad (3.5)$$

- Halfway between sphere and cube

$$W = 0.5\left[\gamma_s\left(\pi d^3/6\right)+\gamma_s\left(d^3\right)\right] = 0.76\left(d^3\gamma_s\right) \quad (3.6)$$

- Cube

$$W = \gamma_s d^3 = 1.0\left(d^3\gamma_s\right) \quad (3.7)$$

The relationship between the d_{50} size of stone and its weight depends not only on its specific gravity, but also the geometric relationship between the particle d_{50} (usually taken as the intermediate or B dimension) and the total volume of the particle. Estimates range from a theoretical minimum (a sphere) to a theoretical maximum (a cube). These minimum and maximum limits can vary significantly based on the maximum allowable shape factor, as described in Section 2.4.1. Galay et al. (1987) include a figure showing riprap measured at a quarry. This figure is reproduced as Figure 3.24. Galay plotted the stone weight versus the B-axis dimension and fit a trend line to the data. Cube and sphere lines have been added to Figure 3.24 for reference. The Galay trend line very nearly plots halfway between a sphere and a cube ($W = 0.76\gamma_s d^3$) for sizes greater than 2.2 ft (0.67 m) and assuming a specific gravity of 2.65. Several particles plot above the cube line. Also shown on this figure is a line for $W = 0.85\gamma_s d^3$. From these data, riprap weight appears to be reasonably represented by $W = 0.85\gamma_s d^3$ where d is the B axis of the stone.

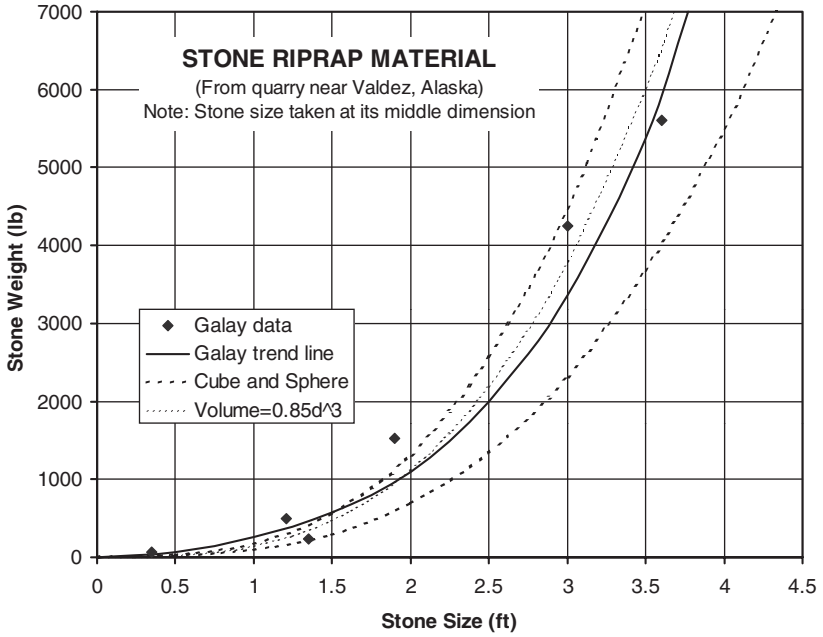
Size Distribution. Stability calculations typically yield a stone size that is represented by either the d_{50} or d_{30} of the particle size distribution (or alternatively, the W_{50} or W_{30} weight). Proper riprap specification provides a tolerance that defines acceptable limits for the percentage of rocks both larger *and* smaller than the design size.

The desired particle dimension or weight is typically expressed in the form of a size distribution curve. Such curves usually indicate the percentage of stones that are smaller than the indicated size, although the CABS (Racin et al., 2000) gradations are based on the percentage larger than the indicated size.

Whether expressed as a "larger than" or "smaller than" gradation, a size distribution curve represents the cumulative distribution function of the sample population of the various rocks that compose the matrix of particles. The d_{50} (or W_{50}) value represents the size for which half the particles are larger and half are smaller (i.e., the *median* size). The steepness of the distribution curve is a measure of the standard deviation of the particle sizes about the median and is referred to as the uniformity of the gradation. The probability function is not necessarily a normal ("bell-shaped" or Gaussian) distribution.

There are different ways to define uniformity. The most common measure of uniformity for riprap used in the United States is the Uniformity Ratio d_{85}/d_{15}, which is also widely used in Europe. Another measure of particle size uniformity is the ratio d_{60}/d_{10}, commonly referred to (in the United States) as C_u, the Coefficient of Uniformity, which is primarily used for soils, geotechnical studies, and for filter design.

Figure 3.25 shows an example of three gradation curves. All curves in this figure have a median size of 21 in (0.53 m), a maximum size of 42 in (1.06 m), and a minimum size of 3 in (0.08 m). However, the shape and steepness of the curves are



Source: modified from Galay et al. (1987)

Figure 3.24. Stone weight versus stone size for riprap.

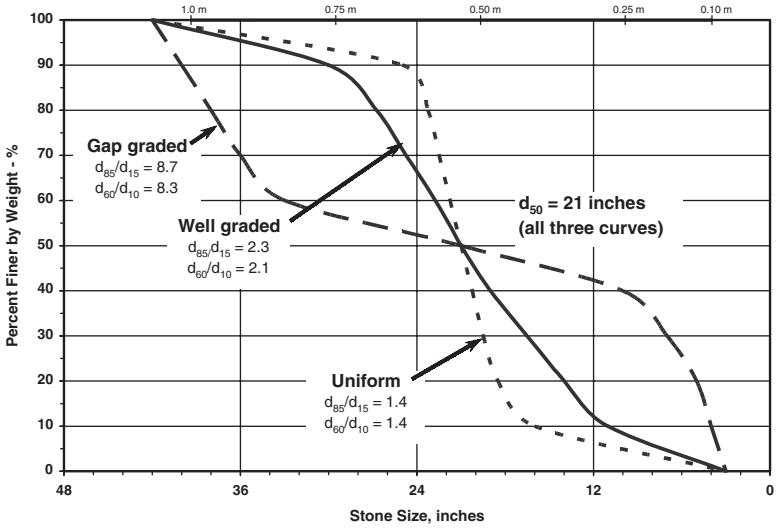


Figure 3.25. Examples of different gradation curves having the same median size d_{50} and maximum size d_{100}.

different, illustrating the concepts of a *well-graded* mixture of sizes, a *uniform* size gradation, and a *gap-graded* distribution. The Uniformity Ratio, d_{85}/d_{15}, and the Coefficient of Uniformity, d_{60}/d_{10}, for each gradation are also shown on this figure.

Defining uniformity based on the d_{85}/d_{15} ratio, CUR and RWS (1995) identifies the three categories identified in Table 3.4; note that because weight is proportional to the cube of the dimension, $W_{85}/W_{15} = (d_{85}/d_{15})^3$.

Standard Classes. Not all riprap sources are capable or willing to produce stone having a unique gradation developed for a specific project, particularly if the next project or next customer will require a different gradation. The definition and use of standardized gradation categories, or *classes*, helps mitigate the producers' concern of trying to "hit a moving target." Thus, adopting standard classes promotes cost efficiency by providing incentive for quarries to gear their blasting and processing methods towards producing a consistent product.

Specifying the use of standard classes is, therefore, preferred over the use of custom gradations. Exceptions to this rule may sometimes be warranted, if cost effective; for example, where extremely large volumes of rock warrant production of a non-standard size gradation, where a temporary dedicated quarry supplies a single project, or where a local quarry naturally produces a non-standard gradation that happens to be suitable for project-specific hydraulic stability requirements.

Using standard classes, an appropriate gradation can be specified, which in common practice means that the class that yields the size equal to or larger than the d_{50} size required for stability is selected. This practice is known as the "next larger" method of specification. This practice results in a somewhat over-designed installation, but economically, a less costly one. Added costs may result from larger volumes of material and the associated transportation and placement costs. The cost-effectiveness of using standard classes versus non-standard gradings should always be evaluated, and standard classes used unless there are compelling reasons to the contrary.

Standard classes are given names based on a characteristic size (either dimension or weight) that is nominally identified with that class. For example, HEC-11 and CABS both refer to classes that are typically named after the minimum allowable weight of the d_{50} particle, such as 2-ton, 1-ton, 1/2-ton, etc. The following summary illustrates the nomenclatures used by the six different methodologies investigated:

- HEC-11 (Brown and Clyde, 1989) identifies six weight classes based on the minimum allowable d_{50}. The classes range from "facing" (75 lbs) to 2 tons.
- CABS (Racin et al., 2000) provides nine weight classes based on the minimum allowable W_{50}. The classes range from "light" (200 lbs) to 8 tons. Because of the layered philosophy used in the CABS approach, this method also

Table 3.4. Definition of uniformity.

Description	Dimension Ratio d_{85}/d_{15}	Weight Ratio W_{85}/W_{15}
Narrow or "single sized" (e.g., "uniform")	1.2 to 1.5	1.7 to 3.4
Wide (e.g., "well graded")	1.5 to 2.5	3.4 to 16
Very Wide (nominally, "quarry run")	Greater than 2.5	Greater than 16

Source: modified from CUR and RWS (1995)

provides three classes of "backing" stone, all having a minimum allowable W_{50} less than 75 lbs.

- EM 1601 (USACE, 1991) defines 12 classes using the d100 particle size. The classes are identified in 3-in increments ranging from 12 in to 54 in. For each class, the minimum and maximum allowable W_{100}, W_{50}, and W_{15} are specified.
- HDS 6 (Richardson et al., 2001) establishes an "ideal" gradation curve that uses the designer's d_{50} size and establishes the remainder of the distribution using multipliers of d_{50}. In essence, this method results in a "custom" gradation specification. Reference to USACE procedures for establishing upper and lower limiting curves are made in HDS 6; unfortunately, that guidance does not produce the same gradations that are established by the standard classes of EM 1601.
- ASTM Standard Practice D 6092 (ASTM, 2003b) provides six weight classes based on the minimum allowable W_{50}. The classes range from "R-20" (20 lbs) to "R-1500" (1,500 lbs). The standard also provides conversions to equivalent size using the following shapes: cube, sphere, prolate sphere, and average of cube and sphere. Specific gravities ranging from 2.60 to 2.75 are considered.
- European practice as reflected in EN 13383-1 (CEN, 2002) divide a total of 15 standard classes (called "gradings") into three categories:
  - Heavy gradings: five classes based on weight ranging from 0.3 to 15 metric tons
  - Light gradings: five classes based on weight ranging from 5 to 300 kg
  - Fine gradings: five classes based on dimension ranging from 45 to 180 mm

Illustrative Example. The following example is provided to illustrate the similarities and differences, both qualitative and quantitative, among existing methods currently in use. The example problem is stated here:

> Assume that a riprap sizing procedure has determined that a median stone size d50 of 20 in at a specific gravity of 2.65 is required. Using a shape factor of 85% that of a cube, a corresponding median weight W50 of 650 lbs is required. From the particle size distribution guidelines of the 6 methods described previously, the "next larger" method of specification is to be used to determine the allowable riprap gradation.

For comparison purposes, the results from the various methods that are based on weight have been converted to the equivalent dimension based on a shape factor of 85% that of a cube, using the intermediate or B axis (see Figure 3.24). Table 3.5 provides a summary of the size distribution characteristics resulting from the six methods investigated.

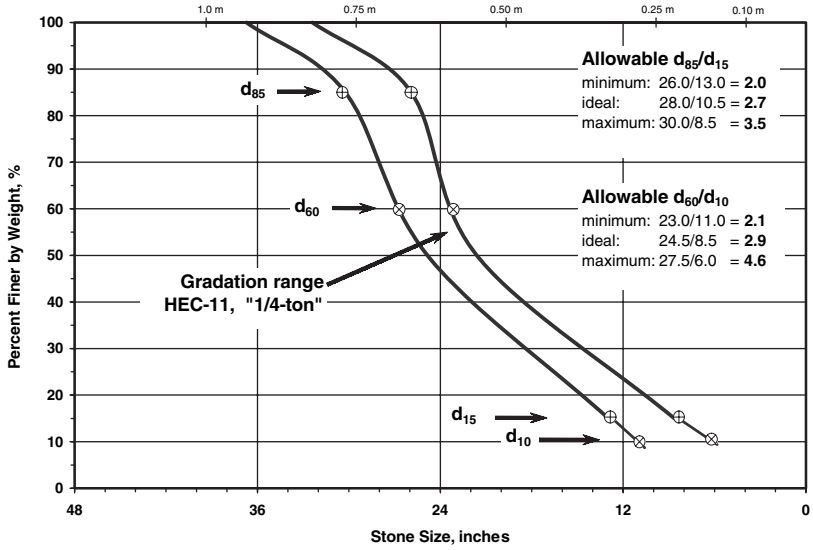
From Table 3.5, the CEN method is seen to yield the most conservative specification in terms of the median rock size requirement. Also from Table 3.5, the CABS and CEN methods are seen to result in the most "stringent" gradations based on the preferred or "ideal" uniformity ratio d_{85}/d_{15} (1.5 or less), while HDS 6 suggests a size distribution that is much more widely graded ($d_{85}/d_{15} = 4.2$).

Figure 3.26 shows the limiting curves that result from the six existing specification methods given a median size d_{50} of 20 in (specific gravity of 2.65, shape 85% that of a cube, and $W_{50} = 650$ lbs).

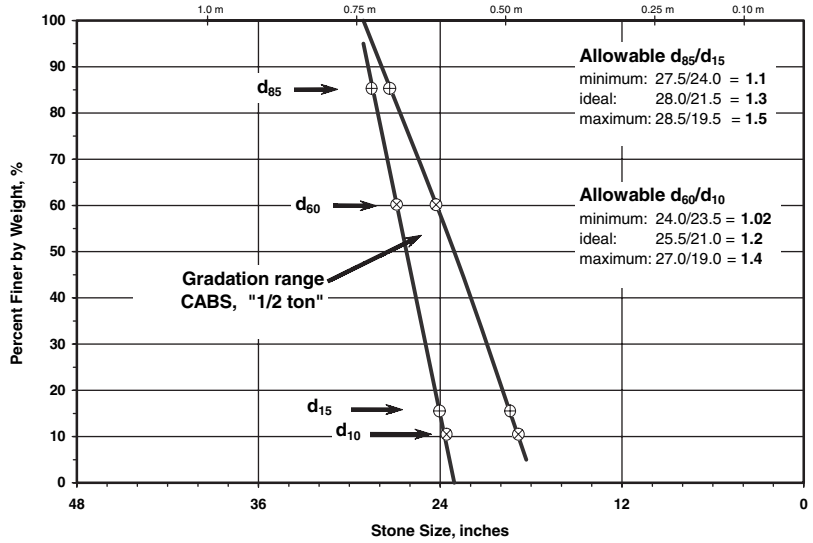
Effect of Uniformity on Stability. Considerable differences of opinion exist with respect to the degree of uniformity that is most appropriate for riprap applications in riverine environments. The little information that is available tends to come primarily from laboratory studies, although a few qualitative field observations are also available. For more

Table 3.5. Summary of standard riprap class characteristics for example specification (d_{50min} = 20 inches, W_{50min} = 650 lb).

Method	Standard Class Designation	Median Particle Size d_{50} (in)			Uniformity Ratio d_{85}/d_{15}			Coefficient of Uniformity d_{60}/d_{10}		
		Maximum Allowable	"Ideal"	Minimum Allowable	Maximum Allowable	"Ideal"	Minimum Allowable	Maximum Allowable	"Ideal"	Minimum Allowable
HEC-11	"1/4 ton"	24.5	23	21.5	3.5	2.7	2.0	4.6	2.9	2.1
CABS	"1/2 ton"	26	24.5	23	1.5	1.3	1.1	1.4	1.2	1.0
EM 1601	"42 inch"	24	22.5	21	2.3	1.7	1.3	2.0	1.5	1.2
HDS 6	custom	22	21	20	4.9	4.2	3.9	4.2	3.6	3.1
ASTM	"R-1500"	26.5	23.5	20.5	2.1	1.6	1.3	2.1	1.5	1.2
CEN	"$HMA_{300/1000}$"	27.5	26	24.5	1.6	1.4	1.3	1.4	1.3	1.2
Summary Statistics for the Six Methods										
Minimum Value		22.0	21.0	20.0	1.5	1.3	1.1	1.4	1.2	1.0
Mean Value		25.1	23.4	21.8	2.6	2.2	1.8	2.6	2.0	1.6
Maximum Value		27.5	26.0	24.5	4.9	4.2	3.9	4.6	3.6	3.1
Standard Deviation		2.0	1.7	1.7	1.3	1.1	1.1	1.4	1.0	0.8



a. HEC-11 method ("1/4 ton" class)



b. CABS method ("1/2 ton" class)

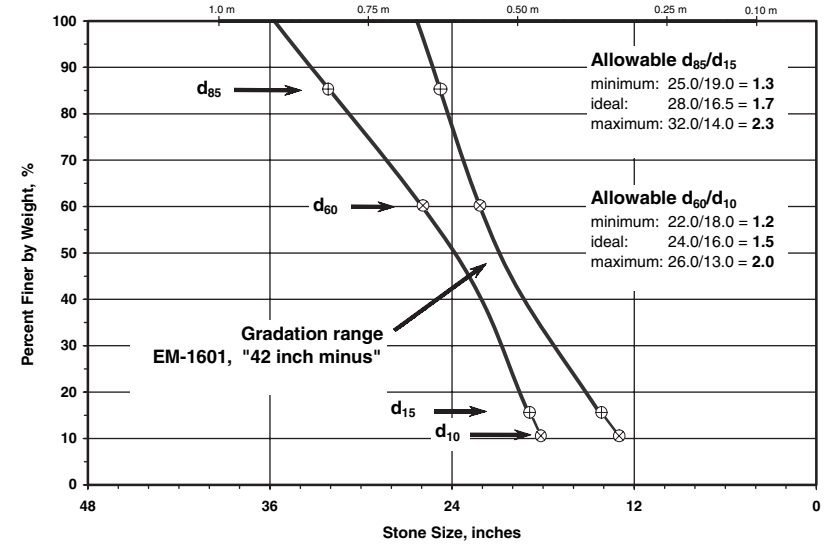
Figure 3.26. Recommended gradations for riprap using the "next larger" method of specification for $d_{50\ min}$ = 20 in, $W_{50\ min}$ = 650 lb.

information, see the discussion in Section 2.4.1. Although there is not complete agreement on the subject of riprap gradation, river applications tend to favor a well-graded distribution, while coastal (wave attack) applications benefit from the use of a more uniform distribution.

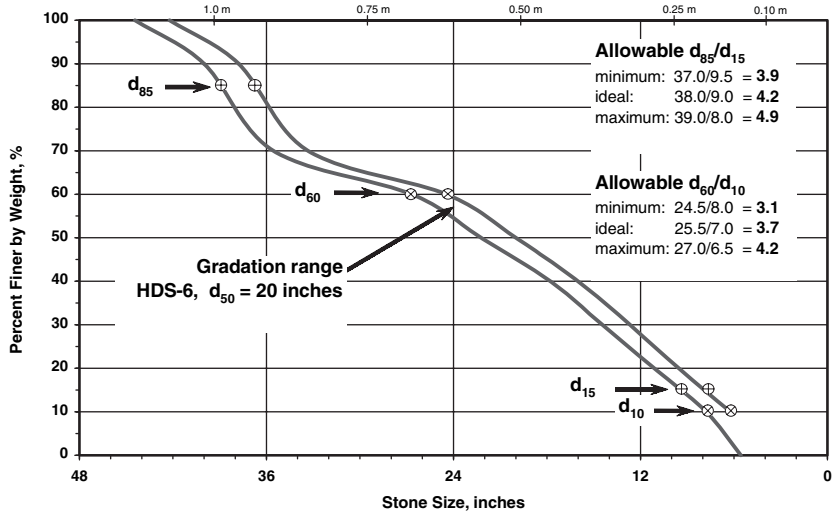
In addition to the physical forces relating uniformity with stability, consideration must also be given to the practical issues of production. Both very uniform gradations and very wide gradations are more expensive to produce compared to intermediate gradations, because of the processing (screening and/or blending) that must be performed, either at the quarry or at the project site.

HEC-11 states, "The stone should be reasonably well graded throughout the riprap layer thickness," although no quantitative guidance is given. HDS 6 maintains that, "A uniformly graded riprap with a median size d_{50} scours to a greater depth than a well-graded mixture with the same median size. . . . With a distributed size range, the interstices formed by the larger stones are filled with the smaller sizes in an interlocking fashion, preventing the formation of open pockets."

Giroud (1982) indicates that a Coefficient of Uniformity of about 3.0 results in the greatest density and thus the greatest degree of interlocking, which indicates that the smaller particles effectively fill the voids between the larger particles. If the ratio exceeds 3.0, the wide distribution of sizes tends to decrease the effectiveness of interlocking. This observation seems to be consistent with the HEC-11 guidelines.



c. EM 1601 method ("42-in minus" class)



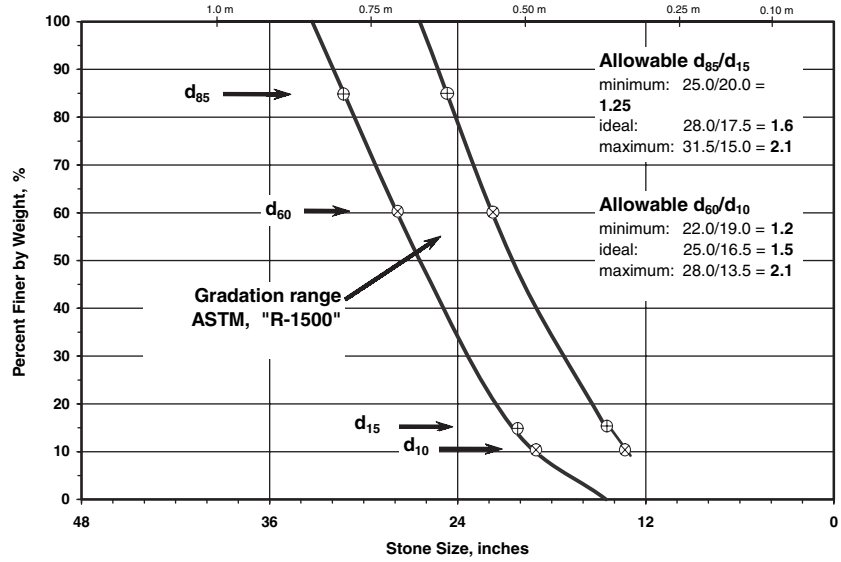
d. HDS 6 method

Figure 3.26. Recommended gradations for riprap using the "next larger" method of specification for $d_{50\ min}$ = 20 in, $W_{50\ min}$ = 650 lb (continued).

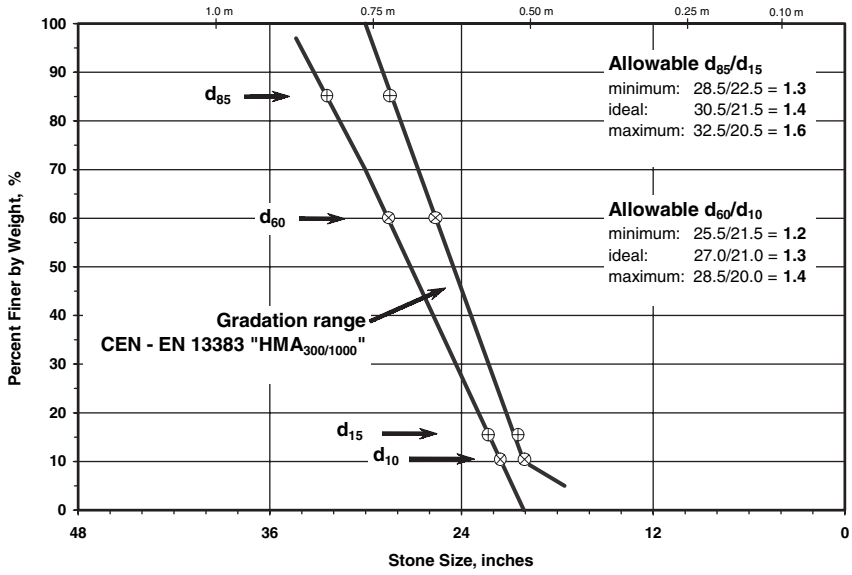
Abt et al. (1988) conducted research sponsored by the Nuclear Regulatory Commission regarding the long-term stability of containment designs for low-level radioactive waste. Wittler and Abt (1990) summarized these studies as well as one by Ahmed (1989) on this topic. They concluded that uniform riprap is more stable under hydraulic loading because of a more efficient transfer of stress from particle to particle via a "three bearing point" distribution of forces. They postulated that the effect of smaller particles is to orient the stresses tangentially to the larger particles, rather than through their centroids. That study also concluded, however, that failure of uniform riprap tends to occur very suddenly and with little potential for self-healing compared to the gradual, particle-by-particle displacement and subsequent rearrangement exhibited by well-graded stone. Anderson et al. (1970) also found that more uniform gradations exhibited somewhat greater hydraulic stability with regard to movement of individual particles.

CABS (Racin et al., 2000) requires very uniform gradation for riprap specification. One or more intermediate layers of successively smaller class stone are typically required to transition between the outer (armor) layer and the geotextile that is placed against the subgrade soil. The CABS design process proceeds logically from the outer (uniformly graded) layer, based on hydraulic loading, to the inner (also uniformly graded) layers that provide the transition to the subgrade soil.

The CABS method also acknowledges, in a qualitative sense, the related issues of particle shape (angular to subangular vs. rounded) and layer thickness as they relate to particle



e. ASTM method ("R-1500" class)



f. EN 13383-1 method ("$HMA_{300/1000}$" class)

Figure 3.26. Recommended gradations for riprap using the "next larger" method of specification for $d_{50\ min}$ = 20 in, $W_{50\ min}$ = 650 lb (continued).

interlocking and overall stability of the riprap installation. CABS (Racin et al., 2000) observes, "As confirmed by field evaluations in . . . this report, small rock in the outside layer of RSP is very loosely held and typically does not interlock well. Small rocks are ultimately washed out of the revetment by impinging flow or during rapidly receding stages. Filling voids in the outside layer with quarry run material is also expensive, especially if rock is measured and paid by weight and not by volume."

Recommended Gradation

Based on the CUR definitions of uniformity (Table 3.4), four of the six gradations (CABS, EM 1601, ASTM, and CEN) allow uniform riprap (d_{85}/d_{15} <1.5) and two of the six (HEC-11 and HDS 6) result in riprap ranging from very well graded to quarry run (d_{85}/d_{15} > 2.5). Recommended gradation criteria were developed as part of this project based on a target d_{50} and a target uniformity ratio that produces riprap that is well graded. For the recommended gradation, the range of acceptable d_{50} is 5% smaller to 15% larger than the target value, which results in a range of acceptable W_{50} of approximately minus 15% to plus 50%. The target uniformity ratio (d_{85}/d_{15}) is 2.0, and the range is from 1.5 to 2.5 (±25%). Using the requirements from the prior example, the recommended gradation is illustrated in Figure 3.27.

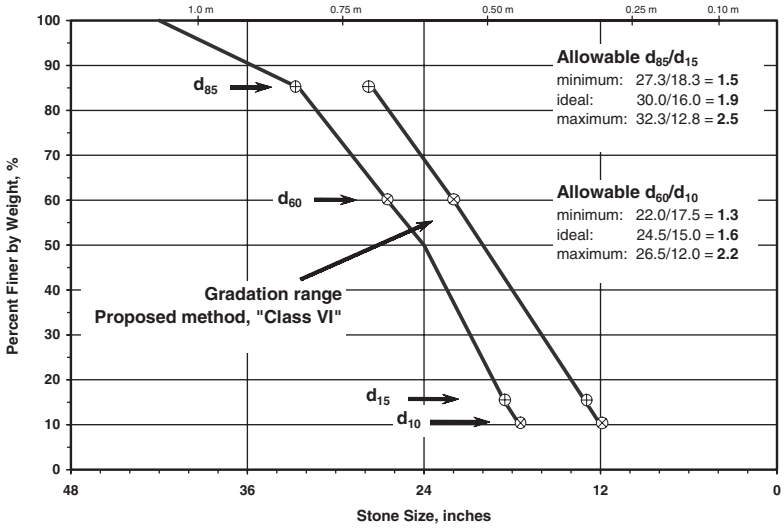


Figure 3.27. Recommended "well-graded" riprap with target d_{50} = 20 in.

The following equations produce the allowable ranges for the d_{10}, d_{15}, d_{50}, d_{60}, and d_{85} sizes:

$$d_{10min} = 0.58d_{50target} \quad (3.8)$$
$$d_{10max} = 0.84d_{50target} \quad (3.9)$$
$$d_{15min} = 0.61d_{50target} \quad (3.10)$$
$$d_{15max} = 0.87d_{50target} \quad (3.11)$$
$$d_{50min} = 0.95d_{50target} \quad (3.12)$$
$$d_{50max} = 1.15d_{50target} \quad (3.13)$$
$$d_{60min} = 1.05d_{50target} \quad (3.14)$$
$$d_{60max} = 1.25d_{50target} \quad (3.15)$$
$$d_{85min} = 1.30d_{50target} \quad (3.16)$$
$$d_{85max} = 1.54d_{50target} \quad (3.17)$$
$$d_{100max} = 2.0d_{50target} \quad (3.18)$$

From the above equations, 10 standard classes of riprap are proposed. Particle sizes based on the intermediate (B) axis range from 6 in (nominal 20-lb stone) to 42 in (3-ton stone). Tables 3.6 and 3.7 provide the recommended allowable range of dimensions and weights, respectively, for the 10%, 15%, 50%, 60%, and 85% finer sizes. The maximum allowable stone size, d_{100}, is also shown in the tables and is based on a dimension that is twice the nominal or "target" d_{50} particle size.

Using this gradation recommendation, the d_{30} size is related to the d_{50} size by $d_{50} = 1.20(d_{30})$, for example when using the EM 1601 procedure for sizing revetment riprap.

Field Tests

OSM Test. In the early 1980s, The U.S. Department of Interior, OSM, developed design procedures for drainage facilities at active and reclaimed surface mines (OSM, 1982). Primarily oriented towards the coal mining industry, the design procedures include riprap-lined surface water diversions where riprap sources are developed on site from sedimentary rocks composing overburden strata. A method whereby an onsite assessment of the suitability of various types of rock for use as riprap could be rapidly conducted by engineers, geologists, or inspectors was developed as part of the design procedures.

Table 3.6. Minimum and maximum allowable particle size in inches.

Nominal Riprap Class by Median Particle Diameter		d_{15}		d_{50}		d_{85}		d_{100}
Class	Diameter	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

Table 3.7. Minimum and maximum allowable particle weight in pounds.

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: Weight limits for each class are estimated from particle size by $W = 0.85(d^3 \gamma_s)$ where d corresponds to the intermediate (B) axis of the particle, and particle specific gravity is taken as 2.65.

The method requires only a geologist's hammer, knife, and 10-power hand lens. Figures 3.28 through 3.30 provide simple, easy-to-use flow charts for field assessment of sandstone, siltstone, and limestone, respectively, as recommended in the "Design Manual for Water Diversions on Surface Mine Operations" (OSM, 1982).

Each flow chart results in a recommendation to either "accept," "reject," or "lab test." The last reflects the presence of one or more indicators, characteristic of sedimentary rocks, which could cause the rock to be less than desirable in a riprap application and that should be investigated further. Note that rocks composed of appreciable amounts of clay—such as shales, mudstones, and claystones—are never acceptable for use as riprap.

For igneous and metamorphic rocks, a classification system to define durability and weathering characteristics was developed by the U.S. Forest Service (Clayton and Arnold, 1972). The seven classes are identified in the following paragraphs:

Class 1, Unweathered Rock. Unweathered rock will ring from a hammer blow and cannot be dug by the point of a rock hammer; joint sets are the only visible fractures; no iron stains emanate from biotites; joint sets are distinct and angular; biotites are black and compact; feldspars appear to be clean and fresh.

Class 2, Very Weakly Weathered Rock. Very weakly weathered rock is similar to Class 1, except for visible iron stains that emanate from biotites; biotites may also appear "expanded" when viewed though a hand lens; feldspars may show some opacity; joint sets are distinct and angular.

Class 3, Weakly Weathered Rock. Weakly weathered rock gives a full ring from a hammer blow and can be broken into hand-sized rocks with moderate difficulty using a hammer; feldspars are opaque and milky; there is no root penetration; joint sets are subangular.

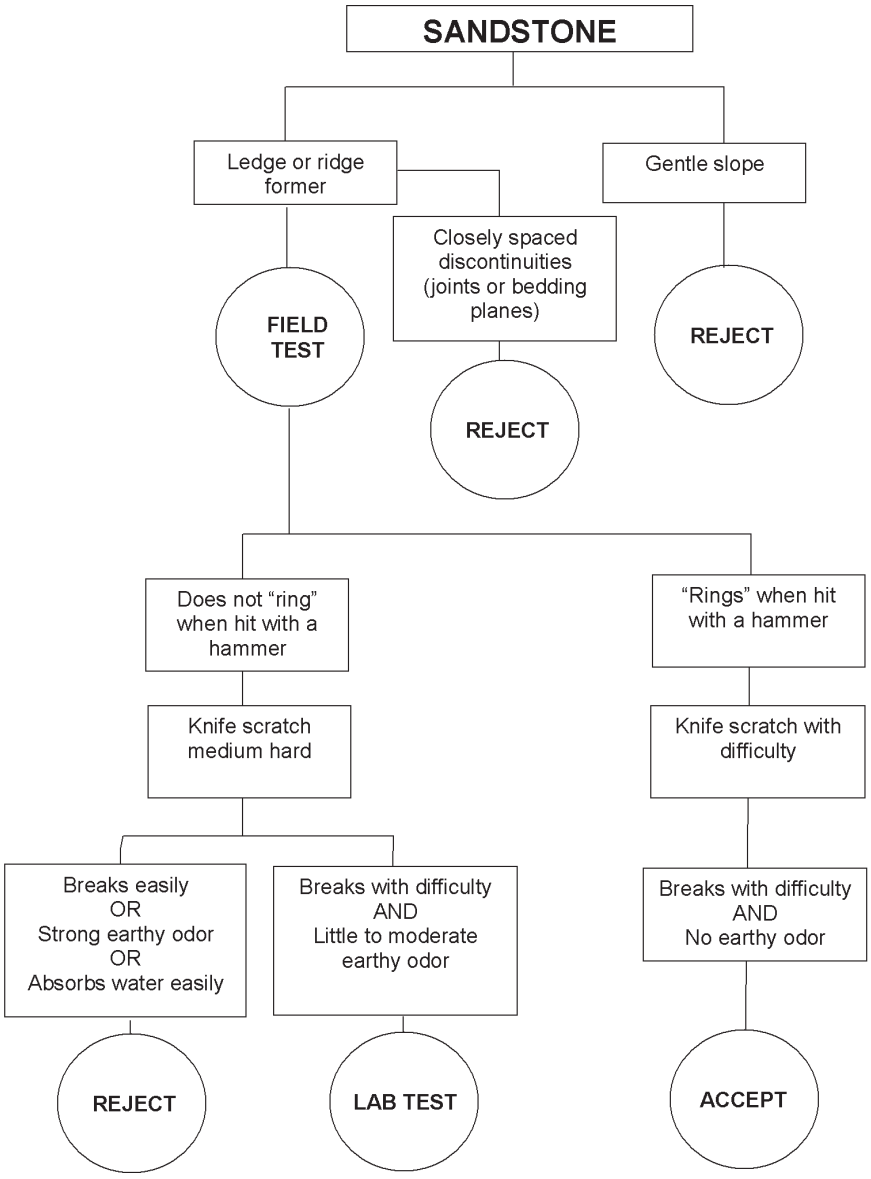
Class 4, Moderately Weathered Rock. Moderately weathered rock may be weakly spalling; except for the spall rind, if present, rock cannot be broken by hand; hammer blow yields no ring or dull ring; feldspars are opaque and milky; biotites usually have a golden yellow sheen; joint sets are indistinct and rounded to subangular.

Classes 5, 6, and 7, Moderately Well-Weathered to Very Well-Weathered Rock. Moderately to Very well-weathered rock can be broken by hand; feldspars are powdery and weathered to clay minerals; biotite appears silver or white; joints are weakly visible, well rounded, or hard to identify; there is root penetration within fractures or throughout rock mass.

Additional Field Tests. The Wolman count method and Galay transect approach are designed to determine a size distribution based on a random sampling of individual stones within a matrix. Both methods are widely accepted in practice and rely on samples taken from the surface of the matrix to make the method practical for use in the field. Details of the methods can be found in Bunte and Abt (2001), Galay et al. (1987), and Wolman (1954). In general, these three references provide detailed descriptions of sampling methods, as well as analysis and reporting procedures for determining the size distribution of rock samples. The Wolman count method is illustrated in this section. The Galay transect approach is discussed in Section 3.2.5 as a quality control method.

Material gradations for sand size and small gravel materials are typically determined through a sieve analysis of a bulk sample. The weight of each size class (frequency by weight) retained on each sieve is measured and the total percentage of material passing that sieve is plotted versus size (sieve opening). The Wolman (1954) count method measures frequency by size of a surface material rather than a bulk sample. The intermediate dimension (B axis) is measured for randomly selected particles on the surface.

One field approach for cobble size and larger alluvial materials is to select the particle under one's toe after taking a step with eyes averted to avoid bias in particle selection. Another field approach is to stretch a survey tape over the material and measure each particle located at equal intervals along the



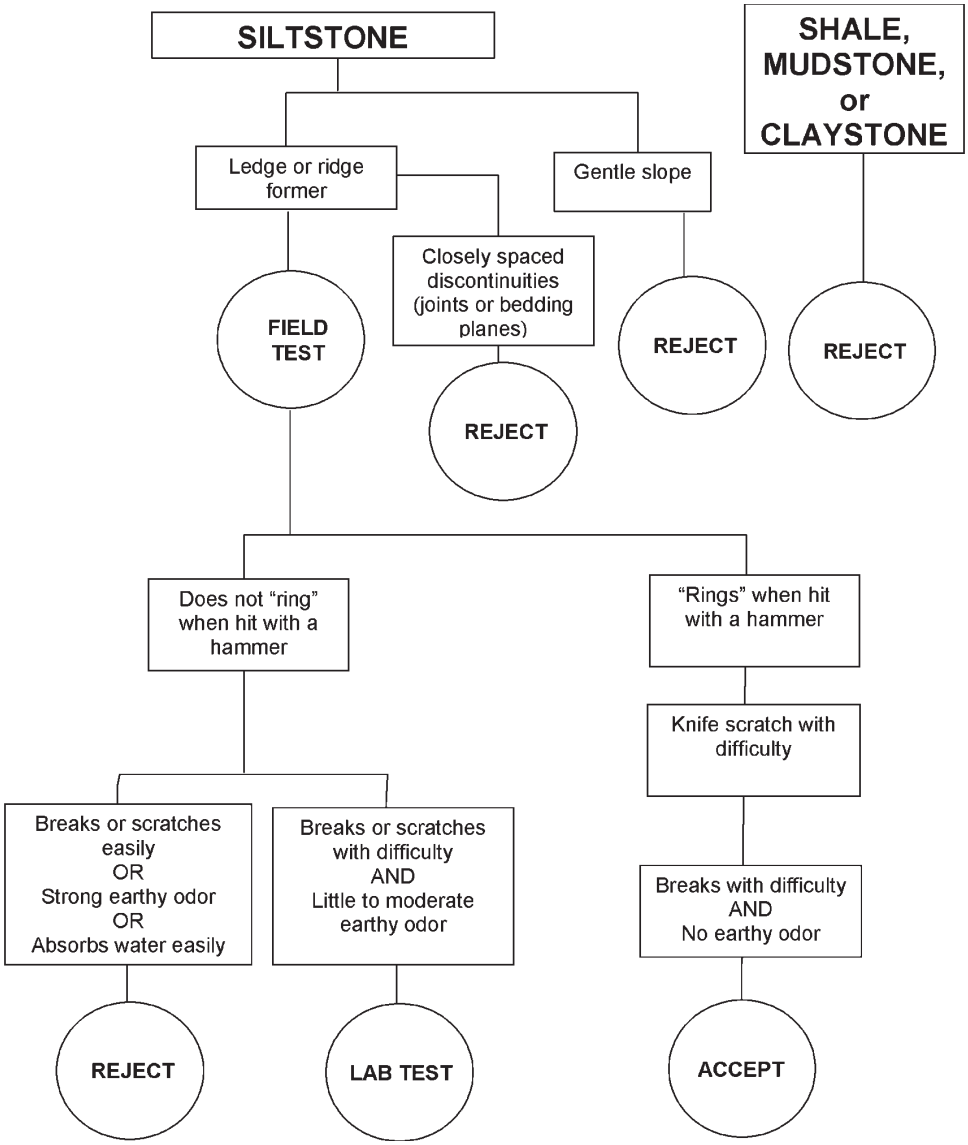
Source: modified from Office of Surface Mining (1982)

Figure 3.28. Rock durability decision chart for field testing of sand-

tape. The equal-interval method is recommended for riprap. The interval should be at least 1 ft for small riprap and increased for larger riprap. The B axis is then measured for 100 particles. The longer and shorter axes (A and C) can also be measured to determine particle shape. Kellerhals and Bray (1971) provide an analysis that supports the conclusion that a surface sample following the Wolman method is equivalent to a bulk sample sieve analysis. 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. An interval large enough that this situation occurs infrequently should probably be selected.

Once 100 particles have been measured, the frequency curve is developed by counting the number of particles less than or equal to specific sizes. To obtain a reasonably detailed frequency curve, the sizes should increase by $(2)^{1/2}$. For uniform riprap, the sizes may need to increase by $(2)^{1/4}$ to obtain a detailed frequency curve. The starting size should be small enough to capture the low range of sizes, with 64 mm being adequate for most riprap. This process should be repeated to obtain several samples at the riprap installation.

Figure 3.31 shows one of two riprap stockpiles that were sampled using a Wolman count to determine whether the sizes met the design criteria of d_{50} equaling 6 and 12 in (0.15 and 0.3 m). Three samples of 100 stones were measured at each pile and gradations curves were developed for each of the six samples. Table 3.8 includes the data and results for sample number 1 on the 12-in (0.3 m) stockpile. The B axis was measured to



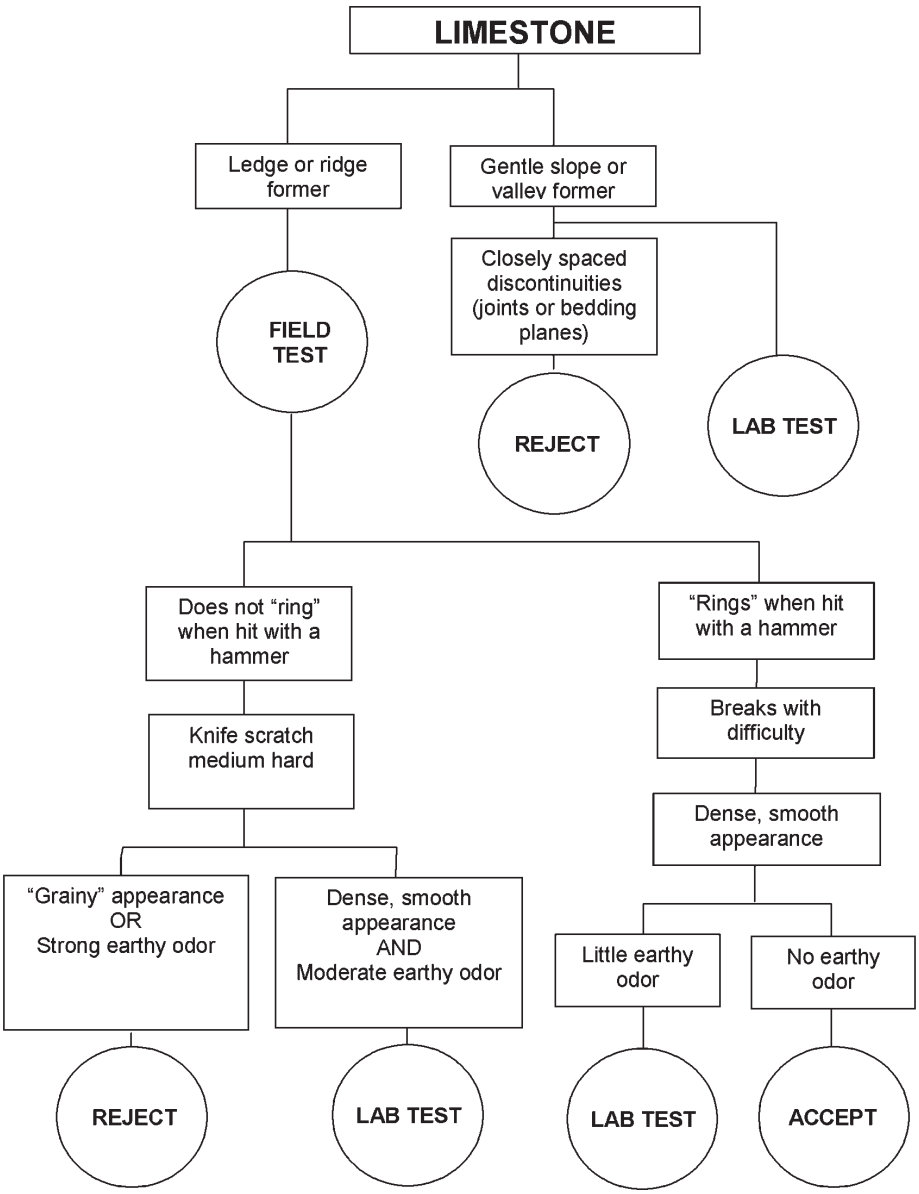
Source: modified from Office of Surface Mining (1982)

Figure 3.29. Rock durability decision chart for field testing of siltstone or shale.

the nearest 10 mm and the percentage less than or equal to each size was computed. The starting size of 64 mm was used and size classes increased by $(2)^{1/2}$ (64 mm, 91 mm, 128 mm . . .). For 100 stones, the "percent passing" is equal to the number of stones less than or equal to a given size.

Figure 3.32 shows the results of the gradation measurements of the two stockpiles. The average gradation was developed by averaging the three samples. The target d_{50} was achieved for the average sample for each stockpile. Also shown is the target or allowable range of sizes based on the recommended gradation discussed earlier. The recommended gradation is based on a target d_{50} and uniformity ratio ($S_t = d_{85}/d_{15}$) ranging from 1.5 to 2.5, which are the limits identified by CUR as "well-graded" riprap (Figure 3.32). The average curve for the 6-in (0.15-m) material meets this gradation target but the 12-in (0.3-m) material exceeds the target maximum d_{84} by 10%. This indicates that the 12-in (0.3-m) material is approaching "quarry run" with the uniformity ratio for the 12-in (0.3-m) material of $d_{85}/d_{15} = 510/187 = 2.7$. One solution to correcting this slight deficiency is to exclude the largest particles during placement. However, that would also reduce d_{50} so the smallest particles should also be excluded from the stockpile.

An alternative to the size-based method described above is to weigh all individual particles from a 10,000- to 15,000-lb (4536- to 6804-kg) sample. A platform scale at the quarry or at the job site can then be used to determine a weight-based gradation. A typical test of this kind takes 4 to 6 hours to complete.



Source: modified from Office of Surface Mining (1982)

Figure 3.30. Rock durability decision chart for field testing of limestone.

Figure 3.31. Riprap stockpile.

Table 3.8. Example gradation measurement using Wolman count method.

Count	mm	Count	mm	Count	mm	Count	mm
1	540	26	560	51	500	76	400
2	510	27	670	52	480	77	340
3	180	28	550	53	180	78	470
4	250	29	220	54	450	79	450
5	250	30	290	55	300	80	280
6	530	31	400	56	420	81	340
7	450	32	320	57	200	82	940
8	170	33	270	58	360	83	600
9	200	34	520	59	290	84	530
10	180	35	650	60	650	85	230
11	520	36	550	61	600	86	400
12	520	37	380	62	400	87	220
13	360	38	180	63	520	88	180
14	300	39	200	64	300	89	300
15	400	40	190	65	320	90	540
16	390	41	340	66	300	91	530
17	170	42	420	67	220	92	270
18	330	43	440	68	260	93	280
19	600	44	300	69	320	94	210
20	380	45	420	70	160	95	200
21	340	46	510	71	470	96	230
22	300	47	540	72	730	97	300
23	280	48	600	73	470	98	390
24	330	49	180	74	200	99	710
25	450	50	290	75	200	100	500

Size (mm)	Percent Passing
64	0
91	0
128	0
181	9
256	24
362	52
512	77
724	98
1024	100
1448	100
2048	100

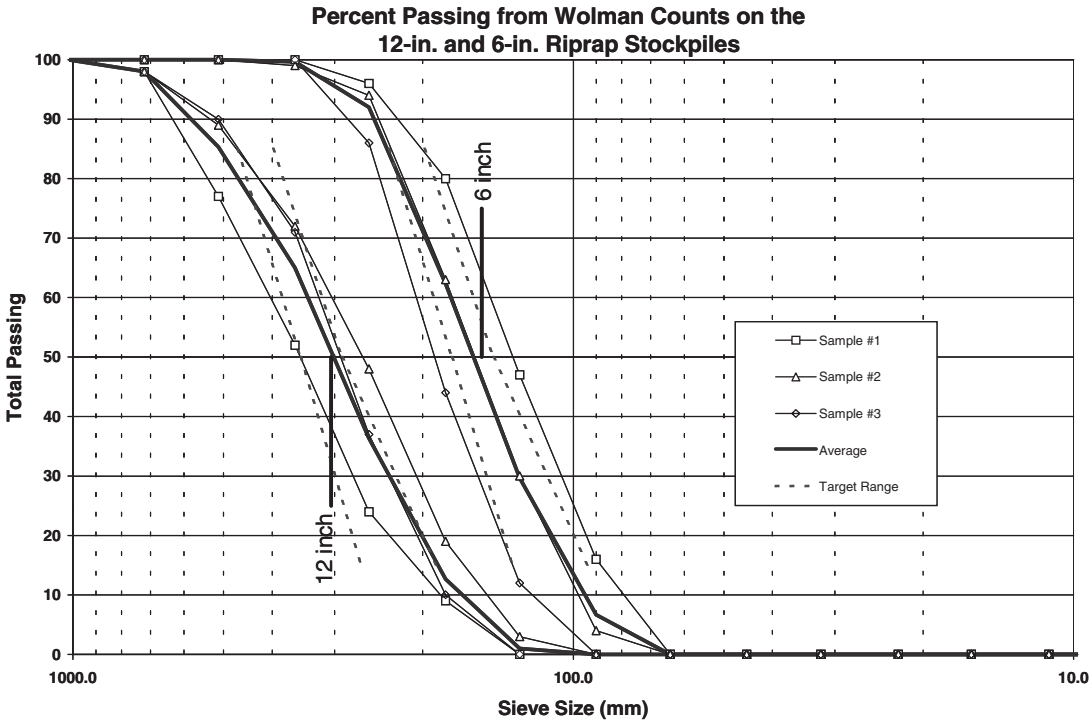


Figure 3.32. Example gradations from 6- and 12-in (0.15- and 0.31-m) d_{50} stockpiles.

Laboratory Tests

In contrast to field testing procedures, laboratory test methods typically yield a numerical value (or values) that provide a measure of the property of interest (size, weight, abrasion resistance, etc.). Test methods do not, in and of themselves, specify what minimum or maximum value is required for acceptance of the material. These "pass-fail" thresholds must be specified by the design engineer. Most state DOTs and other owner agencies provide recommended values appropriate for specific geographic settings and climate.

Relevant standards published by AASHTO and ASTM relating to material type, characteristics, and testing of rock riprap and aggregate material associated with riprap installations (e.g., filter and/or bedding layers) are summarized in Table 2.3. Table 2.4 identifies standards for geotextiles used in conjunction with riprap installations.

In this section, these tables are abbreviated, revised, and reformatted to include additional information relevant to the riprap application (Tables 3.9 and 3.10). Material specifications and test methods not directly related to rock riprap or aggregates have been removed from the tables as presented in Chapter 2. In addition, the discussion of the scope and purpose of each standard has been expanded. The applicability of each with respect to source material certification, testing in the laboratory, testing in the field, and usage in the design/specification process is also indicated.

California Highway Research Report No. M&R 632561, "Investigation of Rock Slope Protection Material," (State of California DPW 19\67) reports on a study to identify tests that should be considered essential in determining rock quality for riprap, compared to other types of tests that may be of only limited use. The report provides guidance for discriminating between tests that correlate well to predicting actual field performance of rock for riprap and those tests that do not. In general, the study found that all the tests would reject obviously unsuitable rock; however, some tests rejected

Table 3.9. Standard specifications and test methods from AASHTO and ASTM for riprap and aggregate.

Designation	Type		Title	Scope	Purpose	Application[1]			
	Material Spec	Test Method				Quarry Cert.	Lab Test	Field Test	Design
AASHTO Standards for Rock and Aggregate									
M 43	✓		Specification for Sizes of Aggregate for Road and Bridge Construction	Defines the size designations and ranges for 19 standard classes of coarse aggregate up to 4 inches in size.	This specification provides standardized size-gradation categories for use by producers, designers, and specifiers. It facilitates the selection of an aggregate or combination of aggregates that is compatible with both the in-situ soil and the armor stone. It is typically used for specifying granular filter material or bedding stone for use beneath riprap.				✓
TP 61		✓	Method of Test for Determining the Percentage of Fracture in Coarse Aggregate	Determines the percentage by weight that consists of fractured particles meeting certain requirements.	For stability and interlocking, rock for riprap should not be rounded, but instead should exhibit angular surfaces. This test consists of a visual determination of fractured particles and results in a quantitative value representative of the sample.	✓	✓	✓	
T 85		✓	Method of Test for Specific Gravity and Absorption of Coarse Aggregate	Determines the specific gravity of the stone and the amount of water absorption after 15 hours of soaking. This test distinguishes between bulk specific gravity, bulk specific gravity (saturated surface-dry), and apparent specific gravity.	The density of rock is a fundamental design parameter in all riprap sizing equations. Rock must be substantially more dense than water to remain stable under hydraulic loading and the force of buoyancy. Also, rock should not be so porous that it absorbs an excessive amount of water when saturated. This test provides a quantitative measure of these properties. For acceptance, the rock must typically exhibit an apparent specific gravity greater than a specified minimum value, and an absorption less than a specified maximum value.	✓	✓		
T 103		✓	Method of Test for Soundness of Aggregates by Freezing and Thawing	Determines the degradation of rock samples in an environment that simulates accelerated weathering; specifically, the weight loss due to disintegration by repeated freezing and thawing.	Rock should not readily weather into smaller pieces when subjected to freezing and thawing. When freezing conditions are expected in the field application, this test provides a quantitative measure of the suitability of rock proposed for use. It is similar to ASTM D 5312.	✓			

Table 3.9. Standard specifications and test methods from AASHTO and ASTM for riprap and aggregate (continued).

Designation	Type		Title	Scope	Purpose	Application[1]			
	Material Spec	Test Method				Quarry Cert.	Lab Test	Field Test	Design
AASHTO Standards for Rock and Aggregate (continued)									
T 104		✓	Method of Test for Soundness of Aggregate by Use of Sodium Sulfate or Magnesium Sulfate	Determines the degradation of rock samples in an environment that simulates accelerated weathering; specifically, the weight loss due to disintegration by repeated cycles of immersion in solution followed by drying.	This test is often used a surrogate for T 103, as it simulates freeze-thaw action by the expansion upon re-hydration of salt crystals deposited in pore spaces during previous immersion cycles. It is similar to ASTM D 5240.	✓			
TP 58		✓	Method of Test for Resistance of Coarse Aggregate to Degradation by Abrasion in the Micro-Deval Apparatus	Determines the resistance of aggregate to degradation by mechanical abrasion using the Micro-Deval Apparatus.	Similar to AASHTO T 210; includes steel balls as part of the abrasive charge.	✓	✓		
ASTM Standards for Rock and Aggregate									
D 4992			Practice for Evaluation of Rock to be Used for Erosion Control	Provides guidance to aid in assessing the suitability of rock for riprap using field observations and measurements. Recommends quantitative test methods that are performed either in the field or in the laboratory.	This standard is neither a test method nor a material specification. However, it is extremely relevant for assessing source materials at an existing or proposed new production site. It provides recommended procedures for investigating and characterizing lithologic formations at the quarry source, as well as the properties of individual rock particles. Includes a valuable summary of various test procedures. This standard does not provide suggested values for pass-fail criteria.	✓			
D 5240		✓	Test Method for Testing Rock Slabs to Evaluate Soundness of Riprap by Use of Sodium Sulfate or Magnesium Sulfate	Determines the degradation of rock samples in an environment that simulates accelerated weathering; specifically, the weight loss due to disintegration by repeated cycles of immersion in solution followed by drying.	This is a weathering test that is a surrogate for freeze-thaw testing. It is very similar to AASHTO T 104, but specifically deals with relatively large samples of cut (sawn) rock (2.5 inches x 5 inches x 5 inches)	✓	✓		

much satisfactory material, partly because of discrimination against certain rock types.

Of particular note is the so-called "Los Angeles Rattler" test, originally developed for determining the abrasion resistance of concrete aggregates. This test proved to be poorly correlated to performance of larger stone used for riprap. Conversely, both the specific gravity and the water absorption tests were found to be reliable indicators of field performance. The minimum recommended specific gravity is 2.5 (CEN recommends that an average value from 10 samples must exceed 2.3), and the maximum recommended allowable water absorption is 2.0% for these tests (CEN recommends that an average value from 10 samples not exceed 0.5%). The sodium sulfate soundness test was also found to be a reliable indicator of field performance, provided that the maximum allowable weight loss was increased from 5% to 10%.

Last, wet-dry and freeze-thaw tests were found to be impractical because of the cost and length of time required to perform these tests, and because the sodium sulfate soundness and water absorption tests were reliable and less expensive surrogates. In other words, materials that would fail the expensive wet-dry and freeze-thaw tests would typically fail the much less expensive sodium sulfate soundness and water absorption tests as well.

With respect to rock and aggregate quality, Table 3.9 summarizes recommendations for laboratory testing. Based on the discussion above, several tests normally associated with riprap have been deleted. Specifically, tests not recommended include:

- ASTM C 535, Test Method for Resistance to Degradation of Large-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine
- ASTM 5312, Test Method for Evaluation of Durability of Rock for Erosion Control Under Freezing and Thawing Conditions

Table 3.10 illustrates the standards for geotextiles used with riprap applications.

Table 3.9. Standard specifications and test methods from AASHTO and ASTM for riprap and aggregate (continued).

Designation	Type		Title	Scope	Purpose	Application[1]			
	Material Spec	Test Method				Quarry Cert.	Lab Test	Field Test	Design
ASTM Standards for Rock and Aggregate (continued)									
D 5519		✓	Test Method for Particle Size Analysis of Natural and Man-Made Riprap Materials	Determines the size and mass gradation of rock particles greater than 3 inches in size.	This is a test method intended for use in conjunction with a material specification such as ASTM D 6092, or other gradation classification systems. Can be performed at the quarry, laboratory, or job site. Unlike AASHTO M 43, this test is specifically designed for use with larger particles of rock. Therefore, it is a more appropriate test for characterizing armor stone (riprap).	✓	✓	✓	
D 5779		✓	Test Method for Field Determination of Apparent Specific Gravity of Rock and Manmade Materials for Erosion Control	Determines apparent specific gravity by weighing rock particles in air (W_a) and in water (W_w).	Similar to AASHTO T 85, except this test is designed to be performed in the field (quarry or job site) using a simple apparatus. Apparent specific gravity calculated as $S.G. = \frac{W_a}{W_a - W_w}$	✓		✓	
D 5873		✓	Test Method for Determination of Rock Hardness by Rebound Hammer	Determines the "rebound hardness" of a rock specimen as a dimensionless number indicating relative hardness.	Hardness is a desirable characteristic of rock used in riprap applications. Hardness is related to other characteristics of rock, such as durability and resistance to weathering or mechanical degradation. The test uses a spring-driven steel hammer, and is rapid and easy to use. Can be used in the lab or in the field. Sometimes referred to as the Schmidt Hammer method.	✓	✓	✓	
D 6092	✓		Practice for Specifying Standard Sizes of Stone for Erosion Control	Provides recommended gradation ranges for six standardized classes of riprap.	This specification provides standardized size- and weight-gradation categories for use by producers, designers, and specifiers. It facilitates the selection of an appropriate class of armor stone. The document includes useful conversion charts for weight to equivalent size for various specific gravities, assuming stone shape is midway between a sphere and a cube				✓
D 6825			Guide for Placement of Riprap Revetments	Provides guidance for placement of rock as well as other riprap system components such as granular filters or geotextiles	This standard is neither a test method nor a material specification. However, it is extremely useful for the planning and designing of riprap installations under a variety of conditions. It provides recommended procedures and includes equipment requirements and recommendations for earthwork and subgrade preparation				✓

[1]Quarry Cert. = Producer certification of rock and/or aggregate properties at point of production Field Test = Jobsite test for compliance during construction
Lab Test = Laboratory test for compliance during construction Design = Design and specification guidance

In Table 3.10, the AASHTO standard M 288, Geotextile Specification for Highway Applications, requires some interpretation and clarification. Current state of the practice for specifying a geotextile as a filter beneath riprap relies heavily on AASHTO standard M 288, in conjunction with FHWA HI-95-038, "Geosynthetic Design and Construction Guidelines" (Holtz et al., 1995). *Few people realize that M 288 in and of itself is NOT a design or construction specification*. It is a material specification intended to facilitate and standardize the purchasing of geotextiles used in highway applications and covers not only erosion control but a wide variety of applications including subsurface drainage, subgrade separation, subgrade stabilization, sediment retention, and paving fabrics for asphaltic cements.

The primary objective of M 288 is to provide strength requirements for geotextiles such that stresses incurred during installation do not damage the fabric. Strength requirements are provided for three classes of geotextiles; the severity of installation conditions dictates the required geotextile class. Class 1 is specified for more severe or harsh installation conditions where there is a greater potential for geotextile damage. Classes 2 and 3 are specified for less severe conditions.

Table 5 of M 288 provides guidance on strength requirements for geotextiles used with rock riprap and other types of armor revetment. In addition, Table 5 provides recommendations for selecting the AOS and permittivity (a property related to permeability) of the fabric as a function of the percentage of in-situ soil passing the 0.075-mm sieve. Lastly, Appendix A of M 288 provides some simple construction/installation guidelines for placement of the geotextile. Section A.4 of M 288 deals specifically with geotextiles used in erosion control applications beneath revetments.

The M 288 material standard together with the design guidance provided in FHWA HI-95-038 forms a comprehensive methodology for selecting and specifying geotextiles in erosion control applications. However, the issue of clogging as it relates

Table 3.10. Standard specifications and test methods from AASHTO and ASTM for geotextile components of riprap installations.

Designation	Type		Title	Scope	Purpose	Application[1]			
	Material Spec	Test Method				Mfg. Cert.	Lab Test	Field Test	Design
AASHTO Standards for Geotextiles									
M 288	✓		Geotextile Specification for Highway Applications	Provides recommended values for the material properties of geotextiles. The specification covers a wide variety of construction applications, including the use of geotextiles as a filter under riprap.	This is primarily a materials purchasing specification that provides guidance on minimum strength requirements. However, it also includes installation guidelines as well as recommended geotextile properties for a range of installation environments and soil characteristics. This specification references various AASHTO and ASTM test methods for determining the strength, elongation, permittivity, effective pore size, open area, and porosity characteristics of geotextiles.	✓			
ASTM Standards for Geotextiles									
D 4759	✓		Practice for Determining the Specification Conformance of Geosynthetics	Provides guidance for determining if the properties of geosynthetic fabrics conform to specifications.	Contains information on QA/QC procedures for ensuring that geotextiles at the jobsite meet the design specifications.	✓	✓		

[1]Mfg. Cert. = Vendor certification of geotextile properties at point of manufacture
Lab Test = Laboratory test for compliance during construction
Field Test = Jobsite test for compliance during construction
Design = Design and specification guidance

to long-term performance of geotextiles is not adequately addressed by FHWA HI-95-038. Specifically, Section 3.3-3 of that manual states,

> Since erosion control systems are often used on highly erodible soils with reversing and cyclic flow conditions, severe hydraulic conditions often exist. Accordingly, designs should reflect these conditions, and soil-geotextile filtration tests should always be conducted. Since these tests are performance-type tests and require project site soil samples, they must be conducted by the owner or an owner representative and not by the geotextile manufacturers or suppliers.

Problematic soils require site-specific testing to support the design and specification of geotextiles for use with riprap revetment. Such soils include very fine non-cohesive soils with uniform particle size, gap-graded soils, laminated soils, and dispersive clays. However, sufficient guidance exists such that soil-geotextile filtration tests do *not* need to be performed for *all* erosion control applications. When the designer/specifier encounters problematic soils such as those just described, or has difficulty finding a geotextile that exhibits the proper balance between permeability and soil retention, laboratory tests should be run using actual samples of site-specific soil in conjunction with several candidate geotextiles. The two most common tests are the gradient ratio test (ASTM D 5101) and the hydraulic conductivity ratio test (ASTM D 5567). For additional discussion, see Section 3.2.2.

3.2.4 Construction/Installation Guidelines

Overview

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 geotextile fabric. The basis of the protection afforded by the riprap is the mass 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, from screening oversized rock from earth borrow pits, from collecting rock from fields or from talus deposits. Screening borrow pit material and collecting field rocks present different problems such as rocks too large or with unsatisfactory length to width ratios for riprap. Quarry stones are generally the best source for obtaining large rock for riprap. However, not all quarries can produce large stone because of rock formation characteristics or limited volume of the formation. Because quarrying generally uses blasting to fracture the formation into rock suitable for riprap, cracking of the large stones may only become evident after loading, transporting,

and dumping at the quarry, after moving material from quarry to stockpile at the job site, or from the stockpile to the final placement location.

In most cases, the production of rock material takes place at a quarry that is relatively distant from the construction site. Therefore, the rock typically must be hauled to the job site where it is either placed directly, stockpiled, or loaded onto waterborne equipment.

Quarry operations typically produce rock for riprap that falls into one of three broad categories based on gradation limits: (1) quarry run, (2) graded (blasted or plant run), and (3) uniform riprap.

- Quarry run riprap sizing is established by controlling the borehole spacing and blasting technique. Some sorting may be required at the shot pile or a rock breaker may be used to reduce oversized rock to within the maximum size allowed.
- Graded riprap sizing is established by controlling the borehole spacing and blasting technique, along with removal of small sizes by running the material over a grizzly or by sizing it through a crusher. This material is more expensive.
- Uniform riprap is produced by removing the over- and undersized material by a series of grizzlies. This process produces a one-sized gradation within a narrow size limit as dictated by the size of the grizzlies. Of the three types of riprap discussed here, this material is the most expensive to produce.

The objectives of construction of a good riprap structure are (1) to obtain a rock mixture from the quarry that meets the design specifications and (2) to place that mixture on the slope of the bank in a well-knit, compact, and uniform layer without segregation of the mixture. The best time to control the gradation of the riprap mixture is during the quarrying operation. Sorting and mixing later in stockpiles or at the construction site is not satisfactory. In the past, control of the riprap gradation at the job site has almost always been carried out by visual inspection. 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 (see Additional Field Tests in Section 3.2.3).

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. 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. Before construction, the contractor should provide a quality control plan to the owner (for example, see USACE ER 1180-1-6, Construction Quality Management [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 can be found in manuals of most governmental agencies involved in construction (FHWA, 1981; USACE, 1991; Racin et al., 2000; BAW, 1993b). These documents provide recommended requirements for the stone, including the tests necessary to ensure that the physical and mechanical properties meet the requirements of the project specifications. **Detailed design and specifications guidelines resulting from this study are presented in Appendix C for the range of riprap applications considered**. Field tests can be performed at the quarry and/or on the job site, or representative samples can be obtained for laboratory testing (see Sections 3.2.3 and 3.2.5).

Gradations are specified and plan sheets show locations, grades, and dimensions of rock layers for the revetment. Additional drawings clarify features at the toe, at the end of the revetment, at transitions, or at other unusual changes in the structures. 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.

Stone size and riprap layer thickness are related. Layer thickness is generally defined as not less than the spherical diameter of the upper limit W_{100} stone or not less than 1.5 times the spherical diameter of the upper limit of the W_{50} stone, whichever results in the greater thickness. Typically, project specifications call for a 50% increase in layer thickness if the riprap is to be placed underwater. Riprap is placed on bedding stone and/or geotextile filter material.

Onsite 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. 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 (see Section 3.2.5).

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
- Whether placement is 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 provide some basic information and description of fundamental equipment, techniques, and processes involved in the construction of riprap revetment works.

Equipment Overview

Although some riprap has been or may be hand-placed, placement is generally accomplished with mechanical equipment and machinery. Equipment to be used in the construction of a riprap structure must be evaluated for each specific site. A brief overview of equipment used to load, transport, and place riprap follows.

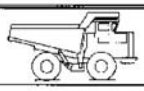
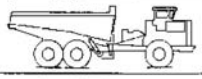
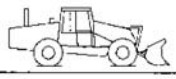
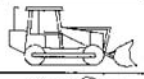
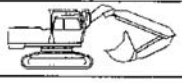
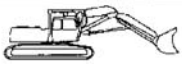
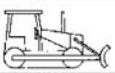
A good resource for a more in-depth discussion of equipment and construction techniques can be found in Chapter 9 of the *Manual on the Use of Rock in Hydraulic Engineering*, prepared jointly by Netherlands' CUR and the Dutch RWS. The manual was published in 1995 by A. A. Balkema and updates a 1991 publication, *Use of Rock in Coastal and Shoreline Engineering*, which was a collaborative project between the United Kingdom's Construction Industry Research and Information Association and the Netherlands CUR.

Riprap may be placed from either land-based or water-based operations and can be placed under water or in the dry. Land-based operations generally use equipment associated with construction of highways and dams. Water-based operations may require specialized equipment for deep-water placement, in some cases, or can use land-based equipment loaded onto barges for near-shore placement.

Transport and handling of stones inevitably cause wear and tear on construction equipment. Repair areas should be established to conduct routine maintenance as well as to fix relatively minor equipment problems. Specific measures such as low earth berms should be employed to contain minor spills of petroleum, oil, or lubricants.

Transportation of Riprap

Hauling riprap at the quarry and at the job site can involve off-highway and highway-rated trucks. Figure 3.33 from CUR and RWS (1995) gives capacities of some off-highway and

type		capacity (m^3)	weight (ton)	wheel load (ton) ground pressure	width (m)
(off higway) dump truck		20 - 90	empty: 30 - 110 loaded: 60 - 270	front/rear (ton) empty: 15/15 - 50/60 loaded: 20/40 - 90/180	wheel base 3.7 - 5.7
articulated dump truck		12 - 27	empty: 20 - 40 loaded: 40 - 90	front/rear (ton) empty: 10/10 - 20/20 loaded: 14/26 - 30/60	wheel base 5.7 - 6.8
wheel loader		2.5 - 9	15 - 86		bucket width 2.7 - 4.7
track loader		2.5 - 3	25	60 - 90 kPa	bucket width 2.7
backhoe crane		0.5 - 15	15 - 200	40 - 150 kPa	track gauge 2 - 5
front shovel		2 - 15	40 - 200	70 - 190 kPa	track gauge 2 - 5
bulldozer		blade width 2.5 - 5 m	10 - 80	50 - 100 kPa	track gauge 2 - 3

Source: CUR and RWS (1995)

Figure 3.33. Equipment to transport and place riprap.

highway-rated trucks, loaders, cranes, and bulldozers that may be used to transport and place riprap. Off-highway trucks include dump and articulated dump trucks with capacities to 150 tons. Width of up to 20 ft and wheel loads prevent highway use of these trucks. Their primary purpose is for relatively short hauls in quarry and mining operations. This equipment is capable of hauling larger sized riprap than standard highway-rated trucks. In the United States, highway-rated trucks are generally 18-wheel, semi-trailer, end-dump units with a gross vehicle weight (GVW) limited to about 80,000 lbs. These trucks can haul up to about 24 tons of riprap, based on wheel base and tare weight.

Highway-rated trucks, railroads, and barges are transportation methods for long hauls from quarry to job site. Standard dump trucks and end-dump, tractor-trailer rigs are used from quarry to job site as well as at the construction area. Haul capacities are about 32 tons for tandem axle trucks and for trailer rigs. For large individual stones, flatbed trucks can be used to transport several at a time. Railroad hopper and side-dump cars can haul 75 to 150 tons of riprap. Side-dumping barges have capacities of 500 to 1,500 tons for the larger vessels and flatbed barges using bulldozers to discharge the riprap have capacities to 5,000 tons. These barges are used for transport and positioned for direct placement of the riprap.

Loading Riprap

Crushing and screening should be performed at the quarry before loading for transport to the site. Because of the size and relative immobility of this type of equipment, locating such activities at the construction site is impractical for all but the largest projects. Unless dumping directly from the quarry haul vehicles, a stockpile at the site will be required. Planning for the required size of the stockpile area depends on the production and transport capacity of the quarry, and the placement rate of the various pieces of equipment to be employed at the working face of the riprap installation. Because of uncertainty of equipment downtime, both at the quarry and at the construction site, flexibility should be programmed into the construction schedule.

Stockpiles may be raised to a considerable height, particularly if footprint area for staging operations is limited. In this case, multiple lifts of rock are placed in layers. CUR and RWS (1995) recommend that maximum slopes of the access roads leading to the top of the stockpile be no steeper than 10%.

Wheeled and track loaders may be used at the quarry or at a stockpile to fill trucks with riprap with an equivalent diameter of about 2 to 3 ft (0.6 to 0.9 m). Larger stones may require a large backhoe or front shovel loader. A crane with a clamshell or orange peel bucket may be required for very large individual stones. In extreme cases, eyebolts may be epoxy-grouted into the individual large stones to provide a lifting point for a crane.

Because loading involves dropping large rocks into the bed of a truck, some equipment operators will load smaller rocks from the stockpile to provide a cushion for the larger riprap. Although this procedure may reduce damage to the truck container, it often results in material not meeting specifications for the gradation, or weight, or for the required d_{50}.

Placing Filter and Riprap

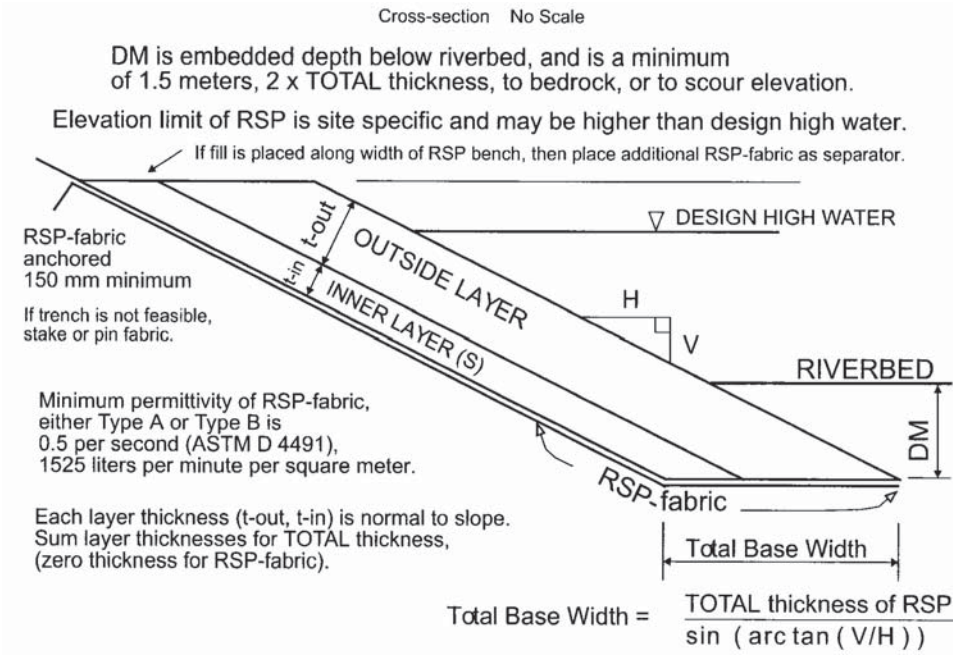
Overview. Construction of a riprap revetment on a river or channel bank begins with the design based on the soils in the bed and bank, the discharge and/or velocity of the stream, planform and bank geometry, and characteristics and availability of suitable rock. The revetment is founded on the natural soil graded to the correct slope. A filter of granular material or geotextile fabric then is placed on the slope with the final layer of riprap bedded on the filter layer. Figure 3.34 is taken from CABS (Racin et al., 2000) and shows a schematic diagram of a revetted streambank with an embedded toe.

Because most riprap failure results from scour or undermining at the toe of the slope, the filter and riprap must extend below the anticipated scour depth (Lagasse et al., 2001). In situations where riprap cannot be installed below the bed level, then sufficient riprap must be stockpiled at the toe to be available to be launched into the scour hole as it develops. Stockpiling riprap at the toe of the revetment should be used only in special situations or until the toe can be excavated and riprap installed to the required depth below the bed. Figure 3.35 shows a schematic diagram of a revetted streambank with a mounded toe.

Placement of Filter. Whether the filter is composed of one or more layers of granular material or 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.

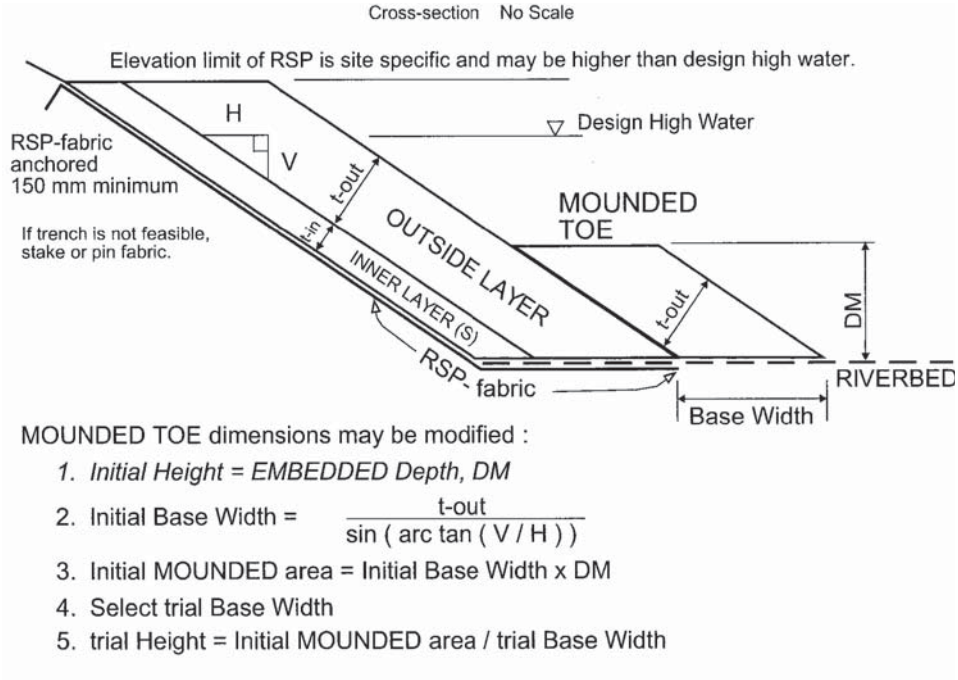
Bank revetment and its underlying filter are often placed both above and below the waterline. Construction is typically conducted during low-flow periods, when the water level is at its seasonal low and flow velocity is relatively mild. After grading and compaction activities have been completed, and any voids filled and organic material (e.g., tree stumps, peat layers) removed, the filter is placed.

When placing a granular filter, front-end loaders are the preferred method for placing and spreading the material on slopes milder than approximately 4H:1V. Steeper slopes typically require that material be dumped on the upper slope and spread with a long-reach Grade-All or backhoe equipped with a wide grading blade instead of a digging bucket as demonstrated in



Source: Racin et al. (2000)

Figure 3.34. Cross section of riprap revetment showing an embedded toe.



Source: Racin et al. (2000)

Figure 3.35. Cross section of riprap revetment showing a mounded toe.

Source: CUR and RWS (1995)

Figure 3.36. Equipment for slope trimming.

Figures 3.36 and 3.37. 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.

Contractors in the United States generally place stone filters and rock riprap using a controlled placement technique with an excavator (typically a hydraulic trackhoe) equipped with a wrist attachment, which allows placing and finishing in one operation. Riprap is placed with the maximum drop onto the geotextile generally limited by specification to 2 ft (0.6 m). Long-reach extended booms are used as conditions dictate. Cranes equipped with a dragline or clamshell bucket are used where the required reach exceeds that of a long-reach trackhoe.

Source: CUR and RWS (1995)

Figure 3.37. Slope trimming ahead of riprap placement.

When placing a geotextile, it should be rolled or spread out directly on the prepared area, in intimate contact with the subgrade, and free of folds or wrinkles. 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. Placement of the overlying rock or stone should be conducted as soon as practicable, so that the geotextile is not exposed to ultraviolet radiation for unnecessary durations. Placement of the rock should be started at the toe and progress up the slope wherever possible. The dumping of riprap on filter material should generally be limited to drop heights of less than 1 ft (0.3 m) to minimize displacement of granular filter media, or tearing of geotextile fabrics.

Along the bankline, the geotextile should be placed so that the upstream strips of fabric overlap downstream strips, and so that upslope strips overlap downslope strips. Overlaps should be in the direction of flow wherever possible and should be at least 1.0 ft (0.3 m) when working on dry ground, and twice that amount when placement is under water. The geotextile should extend to the edge of the revetment within the top, toe, and side termination points of the revetment. If necessary to expedite construction and to maintain the recommended overlaps, anchoring pins or 11-gauge steel, 6- by 1-inch U-staples may be used; however, weights (e.g., sand-filled bags) are preferred so as to avoid the creation of holes in the geotextile.

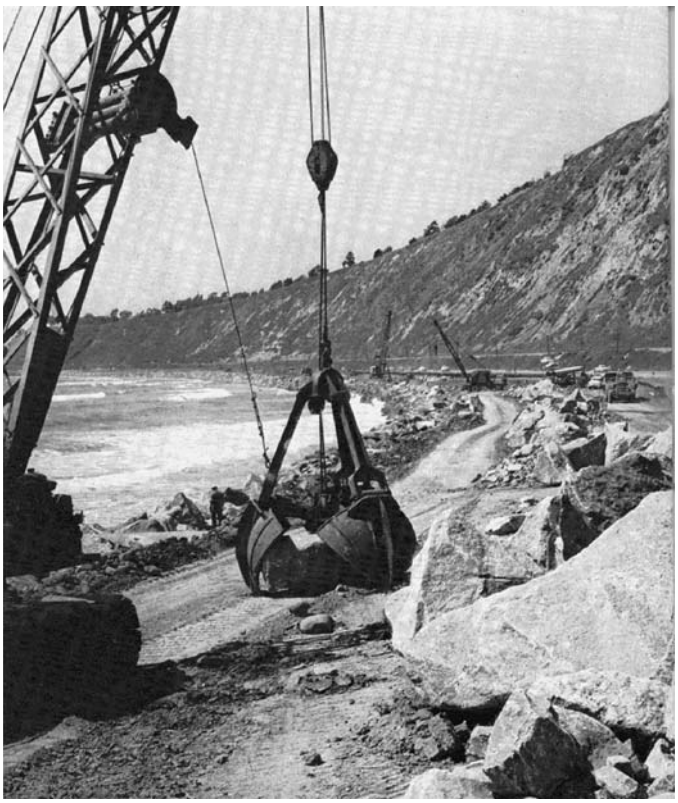
Riprap Placement Equipment and Techniques. There are two fundamental distinctions that must be made when describing the placement of riprap: **direct dumping** of the bulk material as contrasted with **controlled placement** of individual stones or groups of stones. In addition, distinction must be made between **land-based operations** versus **water-based operations;** note that land-based operations can include the placement of stone below the waterline using equipment located on the bank or shore.

CUR and RWS (1995) provides information on different kinds of construction equipment and the typical site conditions and applications for which each is suitable:

- For direct dumping using land-based equipment, dump trucks are typically used in combination with bulldozers, loaders, or trackhoes that are used to rearrange and distribute the stone after it is dumped. This method can be advantageous when little room is available for staging and stockpiling; however, load ratings on local roads must be observed. Direct dumping means placing each sequent lift immediately on the previous lift with relatively little rearrangement of the rocks to avoid segregation. Using chutes or dumping rock at the top of the slope and pushing down the slope with a dozer or front-end loader is not

acceptable. On long slopes where construction is from the top of the slope, cables may have to be attached from the dump trucks to a tractor winch to lower the truck down the slope. In other cases, riprap may be dumped at the toe and a dozer used to move it up the slope. The procedure would depend on the quality of the rock and whether the continuous traffic of the dozers would break down the rock. If the rock is hard and not affected by the traffic, the riprap may be consolidated into a more tightly interlocked mass.

- For controlled placement using land-based equipment, backhoes or cranes are preferred. Clamshells or orange-peel grapples can achieve precise placement of relatively small quantities of stones per cycle, as well as large individual stones. Figure 3.38 shows an orange-peel grapple picking up a large stone during construction of riprap revetment. Rock is usually stockpiled near the installation area and replenished by dump truck as placement proceeds. Lifting capacity of two typical cranes is shown in Figure 3.39 taken from CUR and RWS (1995).
- Direct dumping from water-based operations is typically performed using barges. Vessels are typically of the bottom-door or "split barge" varieties. These vessels place large amounts of bulk material and are most often associated with the construction of the core portions of breakwaters or other structures in large, open areas. Contractors in the United States often drag material off of standard deck barges with a hydraulic trackhoe. A spacer is placed between the loaded barge and an unloading spud barge, and the trackhoe walks the length of the spud barge while pulling the riprap off the side of the loaded barge. Location and movement of the spud barge is controlled using three to five anchors.
- Where the water depth is sufficient to position barges close to the work site, direct controlled placement of riprap is performed with a long-reach hydraulic trackhoe or dragline located on the bank. In shallow water, off-road trucks can be used to haul rock to the placing site. If dikes are being constructed, off-road trucks can use the rock dike as a roadbed. The dike is then finished using a dozer or trackhoe.
- Controlled dumping using water-borne equipment, on a scale slightly less than that of direct dumping, can be performed using so-called side-dump barges where the rock is pushed over the side using conveyor belts, sliding shovels, or bulldozers. Better control over underwater placement is achieved with barge- or pontoon-mounted cranes. Very precise placement can be achieved using "flexible fall-pipe" vessels, which utilize a concept much like a very large-diameter tremie pipe. Rock falling through the pipe does not segregate, the placement is controlled by the feed rate and ship speed, and deposition of the rock is precisely controlled by telemetered positioning sensors at the discharge end of the pipe.



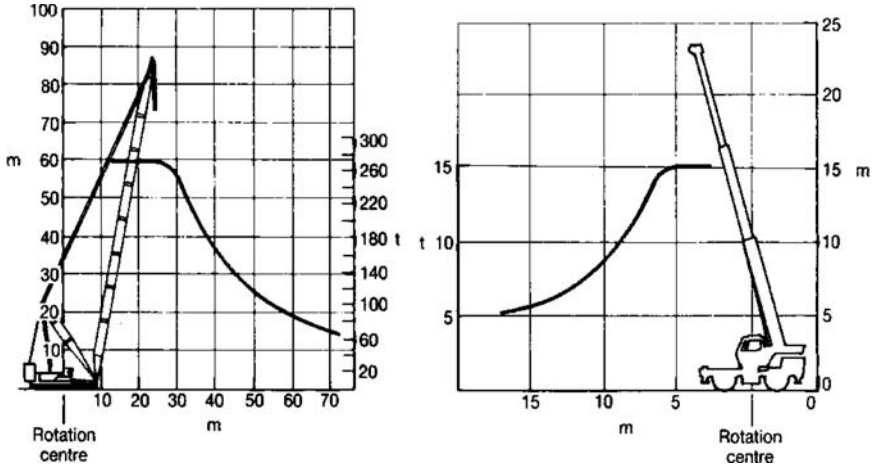
Source: State of California DPW (1960, reprinted 1970)

Figure 3.38. Orange-peel grapple picking up an individual large stone.

Construction of riprap structures under the water line is always problematic because of depth of water and direction and magnitude of the current. Excavation, grading, and placement of riprap and filter under water require additional measures. For installations of a relatively small scale, the stream can be diverted around the work area 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 temporarily suspended by buoys around the work area to minimize environmental degradation during construction. Typically, riprap thickness is increased by 50% when placement must occur under water.

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 (ROV) can provide some information about the riprap placement under water. Even in the best of circumstances, underwater inspection is difficult and expensive.

Termination and Transitions. Termination details for revetment riprap installations typically include edge



Source: CUR and RWS (1995)

Figure 3.39. Lifting capacity of two heavy cranes.

treatment (upstream and downstream) and a toe trench. Bank revetment is typically placed up the slope to extend above the design high-water level with adequate freeboard allowance. Therefore, at the upper bank, the top of the riprap layer is typically feathered into the existing bank line or is terminated at the top of the bank (for additional discussion, see Section 3.8).

Note that the survey questionnaire responses (see Appendix B) indicated that destabilization of the toe of the bank slope was the number one cause of revetment riprap failure. Many respondents indicated that larger stone was typically used at the toe, in either a trenched or buttressed configuration, to provide additional stability in this critical region, but there was little quantitative information supplied in this regard. HEC-11 provides suggested configurations for edge treatment (also known as "turndowns") and toe details. Typically, dimensions of edge and toe terminations are given as a function of the riprap layer thickness (see also Figures 3.34 and 3.35).

In many cases where additional bed scour is anticipated at or near the toe of the bank, an extra quantity of rock is placed on the streambed next to the toe. This extra rock forms a thick apron, the intent of which is to progressively launch riprap into the scour zone as scour is occurring. Often stone in this overlying layer have been oversized. Although some may use this technique as a conservative approach to guard against uncertainty in calculated scour, if and when scour actually occurs, this practice results in an uncontrolled placement of rock, with no underlying filter. If the scour is temporary and is expected to refill with streambed sediment during the receding limb of the design flood, this technique may be a cost-effective approach. However, when toe scour is anticipated, it should be considered in the design, and the toe trench sized and constructed accordingly.

Other Site Considerations. The delivery of sediment that erodes from the staging and work areas to drainageways, streams, or other receiving water bodies must be minimized in accordance with the requirements of the National Pollutant Discharge Elimination System (NPDES) program. Areas of bare soil that have been cleared, grubbed, and graded must be managed according to an approved Erosion and Sediment Control (ESC) Plan. The ESC Plan is a site-specific document, including drawings and specifications, that is developed prior to construction. It calls for Best Management Practices (BMPs) to be employed in a manner suitable for minimizing the escape of sediment beyond the site boundaries. BMPs used most often for construction areas typically include erosion and sediment control products, materials, and management techniques such as

- Silt fences,
- Temporary drainage ditches and berms,
- Sediment ponds,
- Brush and mulch filter barriers,
- Wood fiber or straw wattles and/or temporary blankets, and
- Vehicle wheel-wash pits.

Measurement and Payment

Riprap satisfactorily placed can be paid for on the basis of either volume or weight. When using a weight basis, commercial truck scales capable of printing a weight ticket including time, date, truck number, and weight should be used. When using a volumetric basis, 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 his own expense.

3.2.5 Inspection and Quality Control

Post-Construction/Post-Flood Inspection

Only limited guidance is available for quality control or for post-construction/post-flood inspection of riprap. The following tips for inspecting riprap at bridges are presented in the National Highway Institute (NHI) training course on "Stream Stability and Scour at Highway Bridges for Bridge Inspectors" (NHI #135047):

- 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.)
- Riprap should have a **granular or synthetic geotextile filter** between the riprap and the embankment to prevent loss of embankment material.
- 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.
- **Revetment riprap** must have an adequate burial depth at the toe (toe down) to prevent it from being undermined. Toe down should be deeper than the expected long-term degradation and contraction scour. Additional material should be provided to launch into any scour hole that develops.
- For **piers and abutments**, 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.
- When inspecting riprap, the following would be strong indicators of problems:
  - Riprap has been **displaced** downstream.
  - Angular riprap blanket has **slumped** down slope.
  - Angular riprap material has been **replaced** over time by smoother river run material.
  - Riprap material has physically **deteriorated, disintegrated**, or been **abraded** over time.
  - There are **holes** in the riprap blanket where the filter has been exposed or breached.

The Survey of Current Practice (Appendix B) indicates that, in general, state DOTs and other agencies have not developed specific guidance for post-construction/post-flood inspection. Most DOTs indicated that riprap condition was an inspection item for the biennial bridge inspections conducted under the NBIS (U.S.DOT, 2004), but no inspection guidance specific to riprap had been developed.

Inspections of underwater installations are often hampered by conditions that afford little or no visibility. Underwater inspections must be performed by qualified and experienced divers who may have to rely on feel only.

Based on the discussion and case studies of riprap failures presented in Section 3.7, a suggested riprap inspection code is included in Appendix D. This code parallels Item 113 "Scour Critical Bridges" of the NBIS and would be applicable to all riprap installations including revetments, piers, abutments, and countermeasures.

The code provides a numeric ranking scheme based on both the observed condition of the entire riprap installation as well as the condition of the riprap particles themselves. The code is intended to serve for underwater inspections as well as for installations that can be observed in the dry, with the exception that divers would not be expected to perform a Wolman count for determining particle size distribution (see Section 3.2.3, Additional Field Tests). Action items associated with the coding guidance are also provided with the inspection code.

Quality Control

In reporting on Canadian practice, Galay et al. (1987) notes that, typically, stone material used in the construction of riprapped banks and aprons is specified for design as a gradation on a by-weight basis. If the stones being placed were required to be monitored during construction, hypothetically, a volumetric sample of the stone would need to be obtained and passed through a set of sieves. The accumulated weight retained on each sieve would then be plotted as a percentage of the total sample weight in relation to the grid sizes of each sieve. A volumetric or bulk sample in this instance would involve removal of all placed stones to total riprap layer depth within a specified surface area, or all stones within one or more truckloads being transported to the project site.

As this procedure is not practicable, a variety of methods have evolved to check the size gradation of stones being placed as riprap. Generally, the approach has been to assess stone sizes visually while having some impression of what the maximum, minimum, and average sizes of stone look like. This impression is sometimes obtained by actually weighing stones to find typical examples of these three sizes. For projects where extremely large amounts of stone are

involved, inspectors sometimes go to the extent of dumping randomly selected truckloads of stone and sorting the stones into several piles of different size ranges. Each of these piles is weighed and related to the total sample weight and a typical size of stone for each pile (Galay et al., 1987).

Recently, there has been an effort to develop a simple but effective means of monitoring gradations of stone riprap material (Galay et al., 1987). Basically, what has evolved is a surface-sampling technique, whereby stones exposed on the surface of a completed riprap layer are measured with respect to their sizes. The riprap is sampled in such a way that the measured stones give a representative picture of the proportional area occupied by various sizes. Rather than analyzing the distribution of the sample sizes on a by-weight basis, a by-number analysis is used instead. A gradation curve is then drawn relating stone sizes and frequency distribution. Since riprap specifications are typically provided in terms of stone weight, a link has to be established between stone size and weight. Several methods have been used to describe stone size, including (1) a single measurement of a stone's intermediate dimension and (2) relating a stone's volume to an equivalent spherical diameter. In any case, a sample set of stones is weighed and size dimensions determined so that the stone size versus weight relationship can be determined (see Section 3.2.3).

Various agencies involved with stone riprap construction were contacted by Galay et al. (1987). A summary of their approaches for monitoring riprap gradations during construction is presented in Table 3.11.

Table 3.11. Summary of findings: monitoring of stone riprap gradations during construction.

Agency	Comments
Alberta Environment Construction and Geotechnical Division, Canada	Areal or surface sample from finished riprap blanket. Middle dimension of stone measured for all stones exposed in a selected surface area or stones found at random points along taped line laid along the surface. After collection of 20 or more stone dimensions, distribution of sizes is assessed on a by-number basis and distribution points are plotted on probability paper. This method is assumed to provide a distribution that is reasonably the same as volumetric sample analyzed on a by-weight basis.
Prairie Farm Rehabilitation Administration, Canada	Uses same approach as above. Adopted during construction of large dam 15 years ago. Method has been tested thoroughly to ensure its compatibility with a volumetric by-weight analytical approach.
Colorado Division of Water Resources	Generally assesses stone sizes visually, but for larger projects, several truckloads are selected at random and dumped for a check on gradation. Stones are segregated visually by size into various separate piles, and each pile is then weighed; as well, a representative size from each pile is weighed. The distribution of this volumetric sample is then plotted on a by-weight basis.
USACE, Seattle District	Generally uses same method as above, but admitted that it really has not come to grips with monitoring stone riprap sizes during construction. It sometimes uses grizzly to ensure that proper sizes are being hauled to the construction site. In EM 1601 (USACE, 1991; p. 46), it suggests that, "provision should be made in the specifications for testing in an in-place sample of riprap material as soon as a representative section of revetment has been completed." Unfortunately, it does not indicate whether this sample should be a volumetric or surface sample, or how the sample should be analyzed. It further suggests that selected in-transit truckloads of riprap should be tested.
California State Highways, District 4	Generally uses same method as outlined above for Colorado Division of Water Resources. Specifications are given on a by-weight basis. Stones of proper weight are selected at the quarry that meet the lower and upper limits, and mean sizes of stone. These are marked and set aside so that the loader operator has some means of judging which stones will meet specification. The loader operator is responsible for ensuring that a well-graded assortment of stones is hauled to the construction site.

Source: modified from Galay et al. (1987)

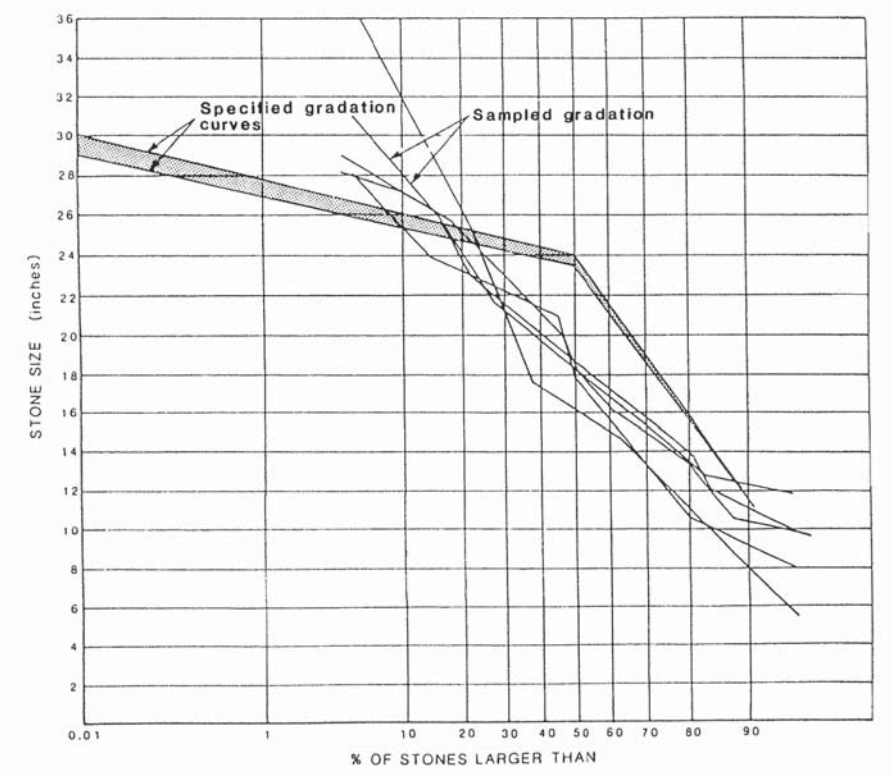
For example, Galay et al. found that the Alberta Environment Construction and Geotechnical Division (Canada) has used a surface-sampling technique for the previous 5 or so years. Generally, its approach has been to take line samples (that is, stretch a measuring tape across the riprap surface and select stones at even intervals) or an areal sample (select every surface stone within a randomly established boundary). The intermediate dimension of each sample stone is measured and the distribution plotted on a by-number basis in relation to stone size. A predetermined relationship between a stone's size and weight is then used to establish the gradation in terms of weight.

The remaining agencies contacted (Galay et al., 1987) follow field-testing procedures related to a visual interpretation of the stone weights that are being placed. Some stones are weighed so that the inspector can gain some appreciation of what minimum, mean, and maximum stone sizes look like. Frequently, this set of stones is marked and set aside at the quarry or the project site for reference by the loader operator and inspector. Rarely, it appears, are large volumetric or bulk samples collected so that individual stones can be weighed and the total sample analyzed on a by-weight basis. Occasionally, bulk samples are collected and sizes segregated into several piles. Each pile is then weighed and a representative size established for each pile; the distribution is then plotted on a by-weight basis.

Basic to the argument that an analysis of surface samples can be considered reasonably equivalent to analysis of bulk sample is a paper by Kellerhals and Bray (1971). Although the subject of interest in the paper is sampling of river bed gravels, the conclusions presented are assumed to apply to all coarse materials, including riprap stone: specifically, "grid sampling with frequency analysis by number is the only sampling procedure capable of describing a surface layer one grain thick, in equivalence with customary bulk sieve analysis" (Galay et al., 1987).

Figure 3.24 presents a plot of sampled stone sizes and their respective measured stone weight, which were selected from a quarry site in Alaska. During placement of stones from this quarry, line samples were collected and their distributions were plotted on a by-number basis. Figure 3.40 shows the results plotted for five samples in relation to the specified gradation envelope curves.

In this instance, stone placement was determined to be unsatisfactory; production procedures were subsequently revised in an attempt to increase the gradation. This revision required an inspector to be present at the quarry, continually working with the equipment operators to ensure that more stones in the middle and lower range were being loaded and hauled to the site (Galay et al., 1987). A similar field test, the Wolman count, suitable for both quality control and post-construction/post-flood inspection of riprap is discussed in Section 3.2.3.



Source: Galay et al. (1987)

Figure 3.40. Stone riprap gradations: specified and sampled.

3.3 Bridge Pier Riprap

There have been a wide range of recent studies for sizing pier riprap using a variety of parametric groupings with significant variation in recommended stone size (see Section 2.4.2). In this section, the preliminary sensitivity analysis of Figure 2.5 (modified from Lauchlan [1999]) is revisited and expanded. The FHWA HEC-18/HEC-23 equation, which was derived from Parola et al. (1989), is compared to several equations including the New Zealand (Lauchlan, 1999) equation using three laboratory data sets.

The laboratory results and design recommendations from NCHRP Project 24-07(2) are evaluated (see Section 2.4.6) regarding filter requirements, riprap extent, and other construction/installation guidelines for pier riprap. Specifically, guidelines for the use of geotextile containers as a means of placing a filter for pier riprap developed under NCHRP 24-07(2) are of particular interest. The Melville and Coleman (2000) construction/installation guidelines, summarized in Section 2.4.2, have been considered in conjunction with current FHWA guidelines in HEC-23. Constructability issues are investigated, including dumping versus controlled placement, underwater versus dry installation, and buried versus mounded placement.

3.3.1 Sensitivity Analysis for Design Equations

Table 3.12 provides a summary of a variety of pier riprap sizing equations from Melville and Coleman (2000) with the addition of the Ruff and Fotherby (1995) equation (see Table 2.5). The Ruff and Fotherby equation is intended for Toskane design but also can be used for riprap design. Table 3.12 also shows the equations reduced to a common, dimensionless form where riprap size divided by flow depth is shown as a function of

Table 3.12. Equations for sizing riprap at bridge piers.

Reference	Equation	Standard Format (for comparison)	Comments
Bonasoundas (1973)	d_{r50} (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)	$N_{sc} = 1.14\left(\frac{d_{r50}}{y}\right)^{-0.2}$	$\frac{d_{r50}}{y} = \frac{0.85}{(S-1)^{1.25}}Fr^{2.5}$	N_{sc} = critical stability number = $V^2/[g(S_s\text{-}1)d_{r50}]$ 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)	$V = 4.8(S_s-1)^{0.5}d_{r50}^{1/3}y^{1/6}$	$\frac{d_{r50}}{y} = \frac{1.65}{(S_s-1)^{1.5}}Fr^3$	
Austroads (1994)	$\frac{d_{r50}}{y} = \frac{0.58K_pK_v}{(S_s-1)}Fr^2$	$\frac{d_{r50}}{y} = \frac{0.58K_pK_v}{(S_s-1)}Fr^2$	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)	$d_{r50} = \frac{0.692(f_1 f_2 V)^2}{(S_s-1)2g}$	$\frac{d_{r50}}{y} = \frac{0.346f_1^2f_2^2}{(S_s-1)}Fr^2$	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)	$d_{r50} = \frac{0.168}{\sqrt{y}}\left(\frac{V}{U_*\sqrt{(S_s-1)g}}\right)^3$	$\frac{d_{r50}}{y} = \frac{0.168}{(S_s-1)^{1.5}U_*^3}Fr^3$ $U_* = \frac{0.3}{K_dK_y}$	$K_y = 0.783\left(\frac{y}{b}\right)^{0.322} - 0.106$ $0 \le (y/b) < 3$ $K_y = 1$ $(y/b) \ge 3$ $K_d = 0.398\ln\left(\frac{b}{d_{r50}}\right) - 0.034\left[\ln\left(\frac{b}{d_{r50}}\right)\right]^2$ $1 \le (b/d_{r50}) < 50$ $K_d = 1$ $(b/d_{r50}) \ge 50$ K_y = flow depth factor K_d = sediment size factor

Table 3.12. Equations for sizing riprap at bridge piers (continued).

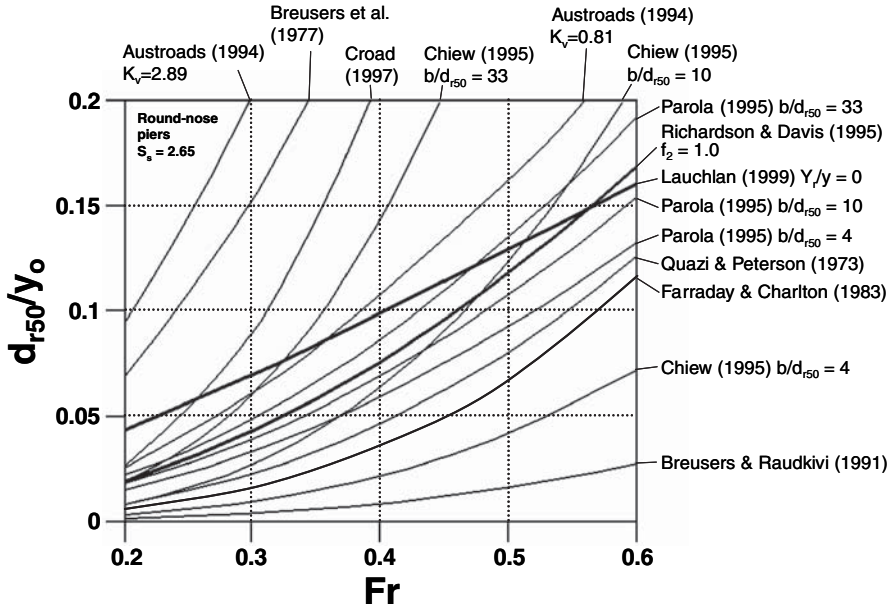
Reference	Equation	Standard Format (for comparison)	Comments
Parola (1993, 1995)	Rectangular: $N_{sc} = 0.8$ $20<(b_p/d_{r50})<33$ $N_{sc} = 1.0$ $7<(b_p/d_{r50})<14$ $N_{sc} = 1.0$ $4<(b_p/d_{r50})<7$ Aligned Round-Nose: $N_{sc} = 1.4$	$\frac{d_{r50}}{y} = \frac{f_1 f_3}{(S_s - 1)} Fr^2$	b_p = projected width of pier f_1 = pier shape factor; $f_1 = 1.0$ (rectangular), 0.71 (round-nose if aligned) f_3 = pier size factor = $f(b_p/d_{r50})$: $f_3 = 0.83$ $4<(b_p/d_{r50})<7$ $f_3 = 1.0$ $7<(b_p/d_{r50})<14$ $f_3 = 1.25$ $20<(b_p/d_{r50})<33$
Lauchlan (1999)	$\frac{d_{r50}}{y} = 0.3 S_f \left(1 - \frac{Y_r}{y}\right)^{2.75} Fr^{1.2}$	$\frac{d_{r50}}{y} = 0.3 S_f \left(1 - \frac{Y_r}{y}\right)^{2.75} Fr^{1.2}$	S_f = safety factor, with a minimum recommended value of 1.1 Y_r = placement depth below bed level
Ruff and Fotherby (1995)	$d_u = \frac{0.255 V_v}{(S_s - 1)} \sqrt{\frac{b}{g}}$	$\frac{d_u}{y} = \frac{0.383 C_l C_s C_h}{(S_s - 1)} \sqrt{\frac{b}{y}} Fr$	b=projected pier width C_l =location factor (0.9 near bank in straight uniform channel, 1.2 for flow in bend) C_s = shape factor (0.9 for streamlined pier, 1.0 for circular pier, 1.1 for square pier) C_h= riprap height factor (1.0 for riprap surface at bed, 1.2 for riprap surface above bed)

Source: modified from Melville and Coleman (2000)

Froude number, rock specific gravity, and correction factors. Figure 3.41 shows a comparison of the various equations for a range of Froude numbers from 0.2 to 0.6 assuming round-nose piers and riprap particle specific gravity of 2.65 (see Figure 2.5). These figures indicate that there is a wide range of predicted riprap sizes for any given flow condition. Lauchlan (1999), Melville and Coleman (2000), and Lauchlan et al. (2000a) compare these equations in detail. Because there is a lack of consistency among the methods, Melville and Coleman (2000) recommend the use of the HEC-18 (Richardson and Davis, 1995), which is now included in the HEC-23 manual (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.

Laboratory Data

Three sets of laboratory data were used to evaluate these equations: data reported by Quazi and Peterson (1973), Parola (1991, 1993), and Ruff and Fotherby (1995). These data were collected for laboratory-scale conditions using uniform-size gravel material (riprap) placed with the surface of the gravel flush with the channel bed around circular and square piers. Each data set includes the particle specific gravity, velocity, and



Source: modified from Lauchlan (1999)

Figure 3.41. Comparison of equations for sizing riprap.

Table 3.13. Range of laboratory data.

Variable	Minimum	Maximum
Particle size (mm)	2.6	25.6
Pier size (in, mm)	2.5,64	9,229
Velocity (ft/s, m/s)	0.89,0.27	4.33,1.32
Depth (ft, m)	0.21,0.064	1.57,0.48
Froude number	0.21	0.78
Pier size/particle size	3.0	24.6
Depth/pier size	0.74	6.3
Particle specific gravity	2.56	2.92

depth for incipient motion conditions for the material protecting the pier. The Parola study focused on two conditions. The first condition was riprap mounded around the pier to a thickness approximately two to three times the riprap size. The second condition was riprap lining a preformed scour hole. Only four of the Parola runs included riprap that was nearly flush with the bed. The Ruff and Fotherby data focused on Toskanes (concrete armor units) but included 26 runs using gravel. The Quazi and Peterson data include 41 runs using gravel. The three data sets comprise a total of 71 measurements of pier riprap stability at laboratory scales. Table 3.13 shows the range of conditions of the three data sets.

Sensitivity Analysis

Each of the equations in Table 3.12 was tested by computing the riprap size for each laboratory hydraulic condition. For the equations that do not include a correction for pier shape, the velocity was increased by 1.13 (1.7/1.5) for the four runs with square piers (Parola, 1991). No other assumptions were required to apply the equations. For each equation, the ratio of predicted size to the actual size was computed for all of the runs. If the ratio was less than 1.0 then the computed size would have failed for the laboratory condition. If the ratio was greater than 1.0, then the computed size would be stable for the laboratory condition. Figure 3.42 presents "box and whiskers" plots showing the distribution of the computed ratios (d_{50}pred/d_{50}obs) for each equation. In Figure 3.42, the interval between each symbol contains 25% (one quartile) of the data. The box includes 50% of the data with one quartile above the plus sign and one quartile below the plus sign. The whiskers above and below each box indicate the range of the upper and lower quartiles. For each equation, any portion of the box and whiskers that plot below a value of 1.0 indicate that the equation underpredicted the required riprap size. Therefore, the "ideal" equation would have no portion of the box and whiskers below a value of

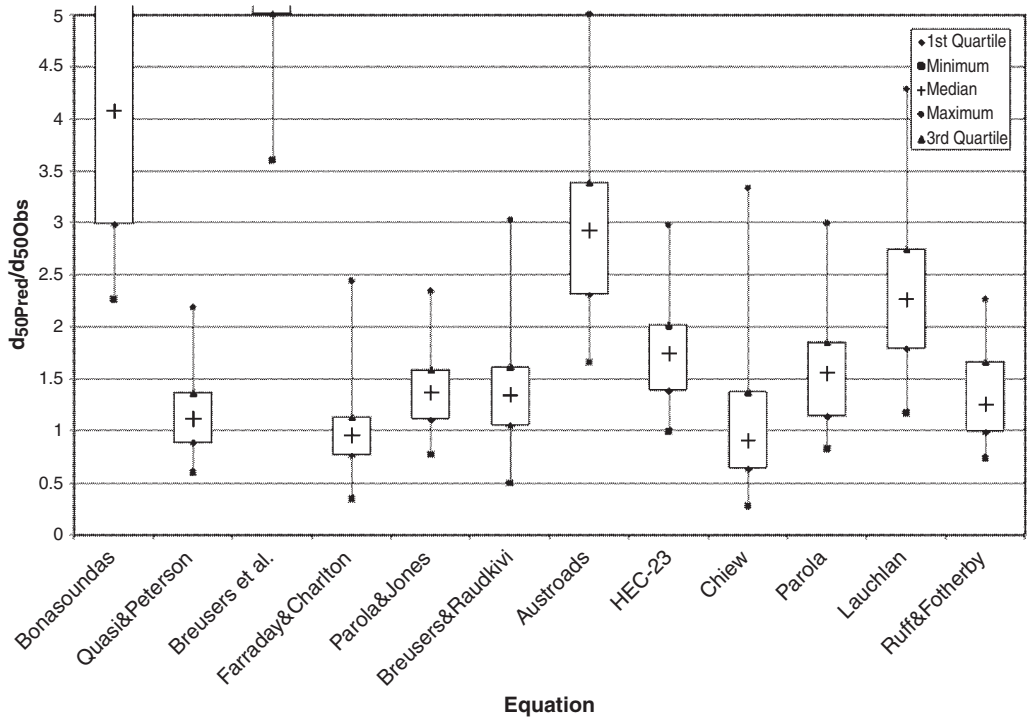


Figure 3.42. "Box and whiskers" plot for pier riprap equations.

1.0 and the entire box and whiskers would be above and as close to 1.0 as possible.

The plot clearly indicates that several equations predict sizes much larger than required: Bonasoundas, Breusers et al., Austroads, and Lauchlan. Several other equations predict sizes that are too small more than 25% of the time for the 71 laboratory runs: Quasi and Peterson, Farraday and Charlton, Breusers and Raudkivi, Chiew, and Ruff and Fotherby. The three remaining equations (Parola et al., HEC-23, and Parola) underpredicted the size of riprap in 9, 1, and 3 of the 71 laboratory runs, which represent 12.7%, 1.4%, and 4.2% of the data. Because these equations are intended for design application, having any underpredictions of size is undesirable. It is also undesirable to grossly overpredict the required size. **Therefore, for use in design, the HEC-23 and Parola equations provide the best balance between the desire to rarely (if ever) undersize riprap and the desire to not be overly conservative. As these equations are very similar, the HEC-23 equation is recommended for design practice**.

Figure 3.43 shows the HEC-23 equation and the laboratory data. The HEC-23 equation envelops the data indicating that it is conservative and probably does not require any factor of safety for application, unless there is considerable uncertainty in the design velocity value.

It is important that the velocity used to size riprap at piers is representative of conditions in the immediate vicinity of the piers, including the constriction caused by the bridge. Appendix C provides guidance on the selection of an appropriate velocity for design.

3.3.2 Filter Requirements

Based on results obtained during the laboratory testing phases of NCHRP Projects 24-07(1) and 24-07(2), granular filters were found to perform poorly where bed forms are present. Specifically, when dune troughs that are deeper than the riprap armor travel past the pier, the underlying finer particles of a granular filter are rapidly swept away. The result is that the entire installation becomes progressively destabilized beginning at the periphery and working in toward the pier. **For this reason, it is strongly recommended that only geotextile filters be used at bridge piers in riverine systems where dune-type bed forms may be present during high flows**.

A second finding relates to the extent of the geotextile filter. In NCHRP Project 24-07(1), a finding suggested that extending the geotextile from the pier to about two-thirds of the way to the periphery of the riprap would result in better performance. This suggestion was considered during Project 24-07(2), and the finding was confirmed. **Geotextile filters at piers should not be extended to the periphery of the riprap, but instead should terminate at two-thirds the riprap extent**.

With these two exceptions, the remainder of the guidance provided for filters in Section 3.2.2 of this report is appropriate for riprap installations at bridge piers. Photographs from NCHRP Project 24-07(2) that illustrate these findings are provided in Figures 3.44 and 3.45.

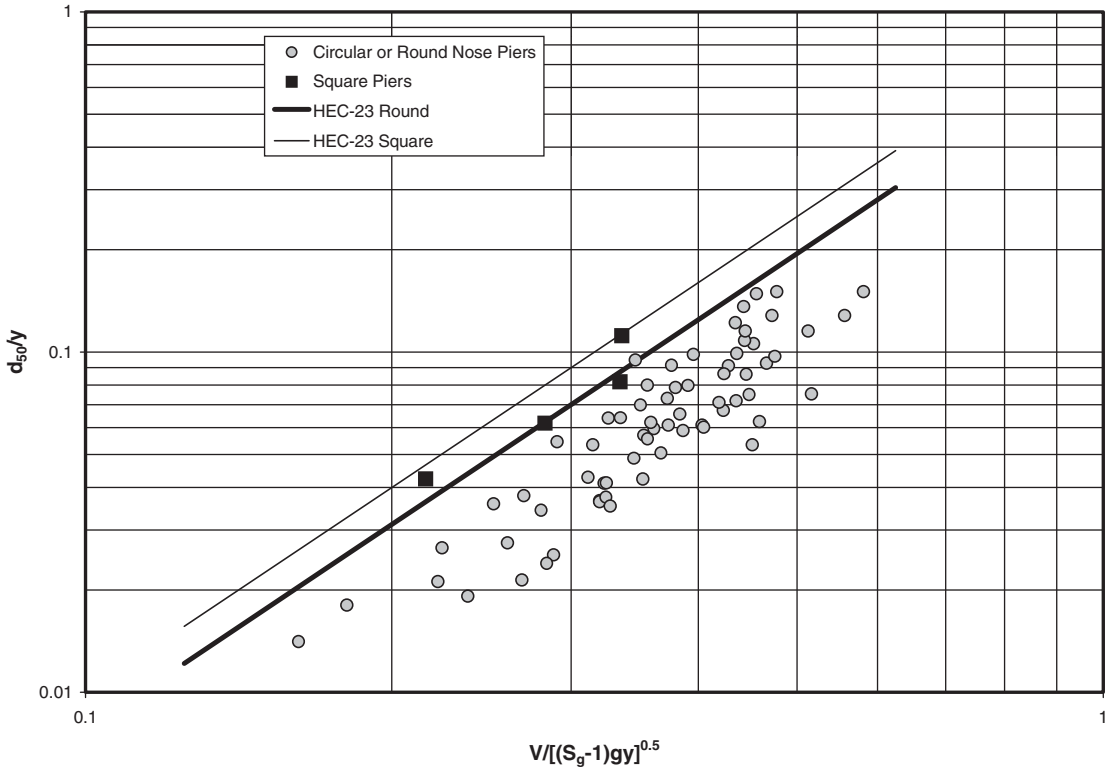


Figure 3.43. Comparison of HEC-23 equation to laboratory data.

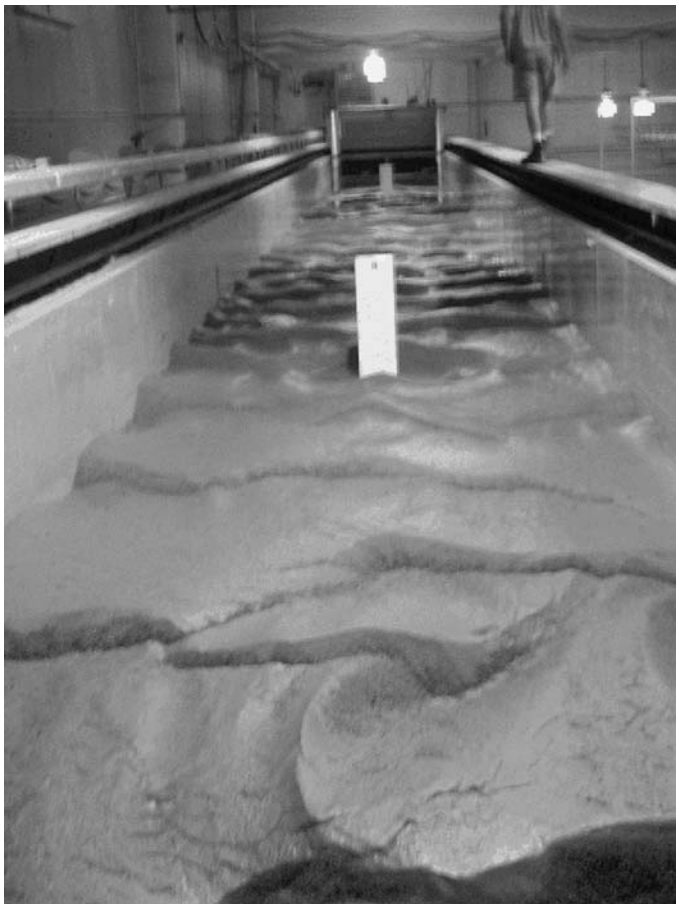


Figure 3.44. Square (8-in) piers in the 8-ft wide, indoor flume (looking upstream) used for NCHRP Project 24-07(2).

3.3.3 Material and Testing Specifications

The requirements for the quality and characteristics of riprap materials, and the associated tests to support them, are presented for revetment riprap installations in Section 3.2.3. These requirements are suitable for use with pier riprap as well.

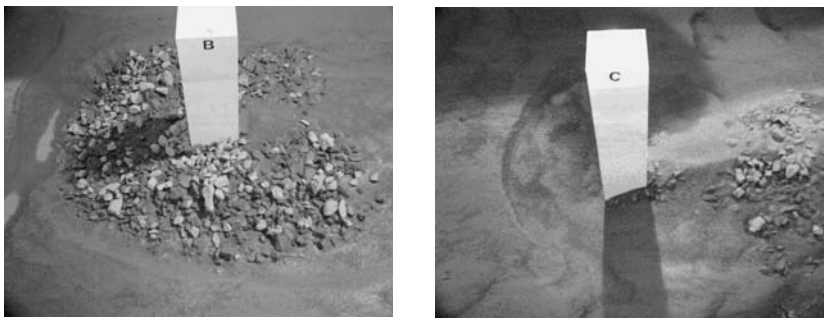
3.3.4 Construction/Installation Guidelines

Guidelines for constructing and installing revetment riprap and filters are provided in Section 3.2.4. The guidelines for riprap installations at piers are similar both for placement in the dry as well as underwater. Some modifications to construction/installation guidance for pier riprap follow:

- Placement of the geotextile around the structure (e.g., pier, footer, pile cap) must ensure that a good seal to the structure is established. This seal will prevent the loss of finer bed material from any gaps that might otherwise occur next to the structure. In underwater situations, sand-filled geocontainers can be used for this purpose, as well as for filling any local scour holes around the pier before placement of the riprap armor (see Figure 3.23).
- The top of the riprap surface should be flush with the ambient bed level, which may require pre-excavation before geotextile and riprap placement (see Figure 2.4).
- Geotextile is preferred over granular material for use as a filter for riprap at piers. The geotextile should extend two-thirds the distance from the pier to the periphery of the riprap (see Section 3.3.2).
- The riprap armor should extend a distance of two times the pier width (as measured perpendicular to the approach flow) around all sides of the structure.
- The minimum thickness of the riprap layer should be three times the d_{50} riprap size. Thickness should be increased to include contraction scour, long-term degradation, and bed form troughs if any of these conditions exist during flood flows.
- Thickness should be increased by 50% when placing riprap under water.

3.3.5 Inspection and Quality Control

Guidance provided for inspection and quality control for revetment riprap installations in Section 3.2.5 is appropriate



a. Test 5d, riprap with geotextile filter.

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

Figure 3.45. Riprap as a pier scour countermeasure, NCHRP Project 24-07(2).

for pier riprap installations. The inspection code included in Appendix D accommodates pier riprap as well as other types of riprap applications.

3.4 Bridge Abutment Riprap

Based on the synthesis contained Section 2.4.3, only the abutment riprap sizing approach as developed by FHWA and presented in HEC-23 appears to be a candidate for further investigation. The approach consists of two equations: one for Froude numbers less than 0.8 and the other for higher Froude numbers. There are no field data available to test these equations and the only available laboratory data set was used to develop the equations (see Section 2.4.3). The FHWA equations rely on an estimated velocity, known as the characteristic average velocity, at the abutment toe. Rather than evaluating these equations using the same laboratory data set used to develop them in the first place, the method for estimating the velocity at the abutment is investigated in detail. Two-dimensional (2-D) modeling was performed to evaluate the flow field around an abutment using FESWMS-FST2DH version 3.1.5 (Froehlich, 2003).

In addition to riprap sizing, the HEC-23 manual (Lagasse et al., 2001) provides information on abutment riprap design including freeboard, extent, layer thickness, and the use of a riprap apron. The recommended apron extent from the toe of slope is two times the flow depth in the overbank area, but not to exceed 25 ft. Melville et al. (2006) indicate that there are conditions where an apron extent of twice the flow depth may not be adequate.

Filter requirements, material and testing specifications, construction and installation guidelines, and inspection and quality control procedures are discussed in the following sections with reference to the guidelines and specifications for revetment riprap.

3.4.1 Design Equations

The flow field around an abutment cannot be estimated directly using one-dimensional (1-D) models such as HEC-2, WSPRO, or HEC-RAS, because each of these models distributes flow at any cross section, whether the cross section is natural or the constricted bridge opening, based on the distribution of conveyance within that cross section. The assumptions that are used include level water surface for the entire cross section and equal energy slope throughout the cross section. These assumptions result in a computed velocity at the abutment much lower than anywhere else within the cross section when in fact the velocity may be much higher. The FHWA abutment riprap equation was developed using the actual velocity as measured at abutments in laboratory investigations. Appropriate estimation of velocity from standard 1-D modeling results is critical to the application of these equations. The method for estimating abutment flow velocities, herein call the Set Back Ratio (SBR) method, is described in detail in Section 2.4.3. The SBR is the distance the abutment toe is set back from the channel bank divided by the average flow depth in the channel. If the SBR is less than (or equal to) 5, then the velocity used in the abutment riprap equations (characteristic average velocity) is the average velocity in the entire bridge opening. If the SBR is greater than 5, then the velocity is estimated by dividing the total upstream (unconstricted) floodplain flow by the flow area between the channel and the abutment (set back area). It should be emphasized that the SBR method is intended to estimate the actual velocity at the toe of the abutment and does not infer that this velocity exists over the entire setback area between the abutment and channel bank.

Figure 3.46 shows the base FESWMS-FST2DH model used to test the SBR method. The topography (a), finite element network (b), and material coverage (c) are shown. The model is a relatively simple geometry that includes a straight channel on a straight floodplain and significant roadway embankment encroachment. Flow is from top to bottom so the right (looking downstream) floodplain embankment has a spill-through abutment and the left floodplain embankment has a guide bank. An extremely detailed finite element network was developed to very accurately simulate the flow fields around the abutment and guide bank. Even though this 2-D representation is very detailed (some elements around the abutment and guide bank are less than 2 ft [0.6 m] long), the flow field simulation is still not exact because vertical velocity and acceleration components are excluded and hydrostatic pressure is assumed. Even with these limitations, the model results are deemed to be a reasonable representation of the flow conditions. Figure 3.47 shows the topography and finite element network within the bridge opening including the guide bank.

Figure 3.48 shows the results of the base model simulation. The highest velocities occur in the channel and at the abutment. Slightly lower velocities occur at the head of the guide bank, along the guide bank, and at the bridge exit at the downstream end of the guide bank. Flow separation occurs along the abutment and on the embankment downstream of the guide bank. All of the model boundaries along the embankments, abutment, and guide bank are sloped at 2H:1V. There are velocity "hot spots" at the top of each slope at the abutment, guide bank head, and at the downstream end of the guide bank. These velocities were not used for comparison purposes because they may be numerical artifacts rather than accurate representations of real flow. (They may be numerical artifacts because, for very shallow flow, the true Manning n is expected to be higher than for deeper flow conditions.) Therefore, the highest velocity at the toe of each slope was used for this evaluation. Also, the toe of slope is the location of initial riprap failure observed in the laboratory and at prototype scales (H.J. Verheij personal communication, 2005).

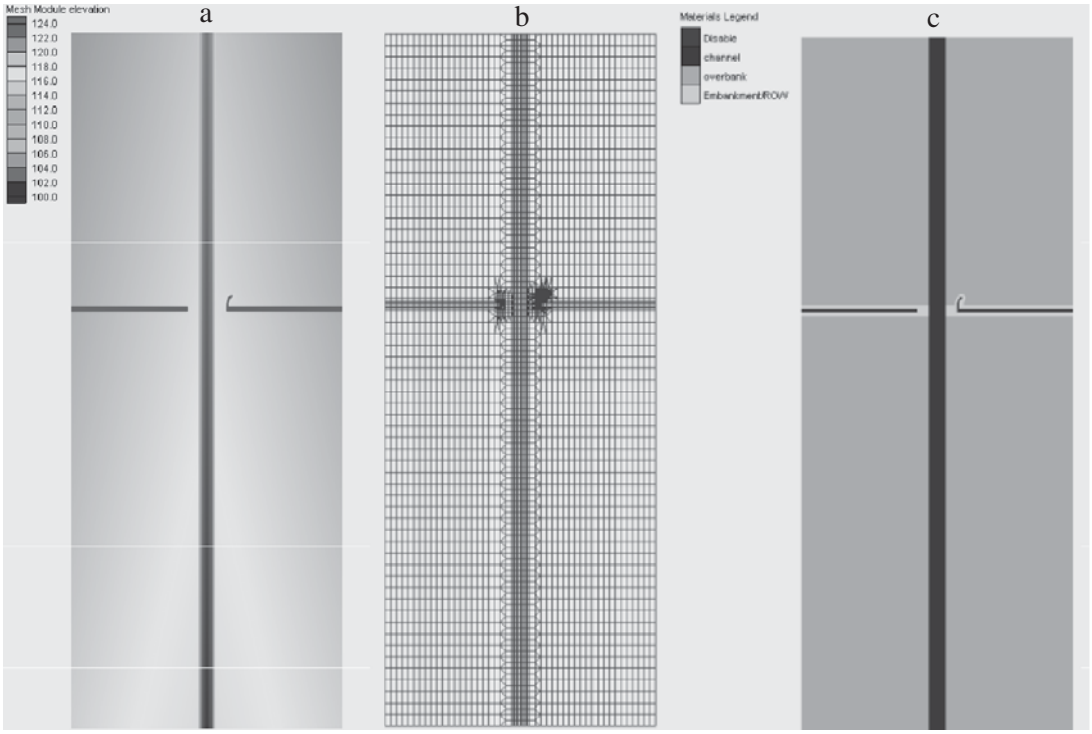


Figure 3.46. Base model for abutment flow simulations.

The base model was modified using six different variables to produce a wide range of unobstructed and constricted flow conditions. These modifications were (1) total discharge, (2) floodplain Manning n, (3) guide bank length, (4) abutment set back distance, (5) floodplain width, and (6) longitudinal slope. Twenty-one different variations of the base model were produced for a total of 22 different simulations. Table 3.14 shows the conditions simulated in the 22 models. The downstream

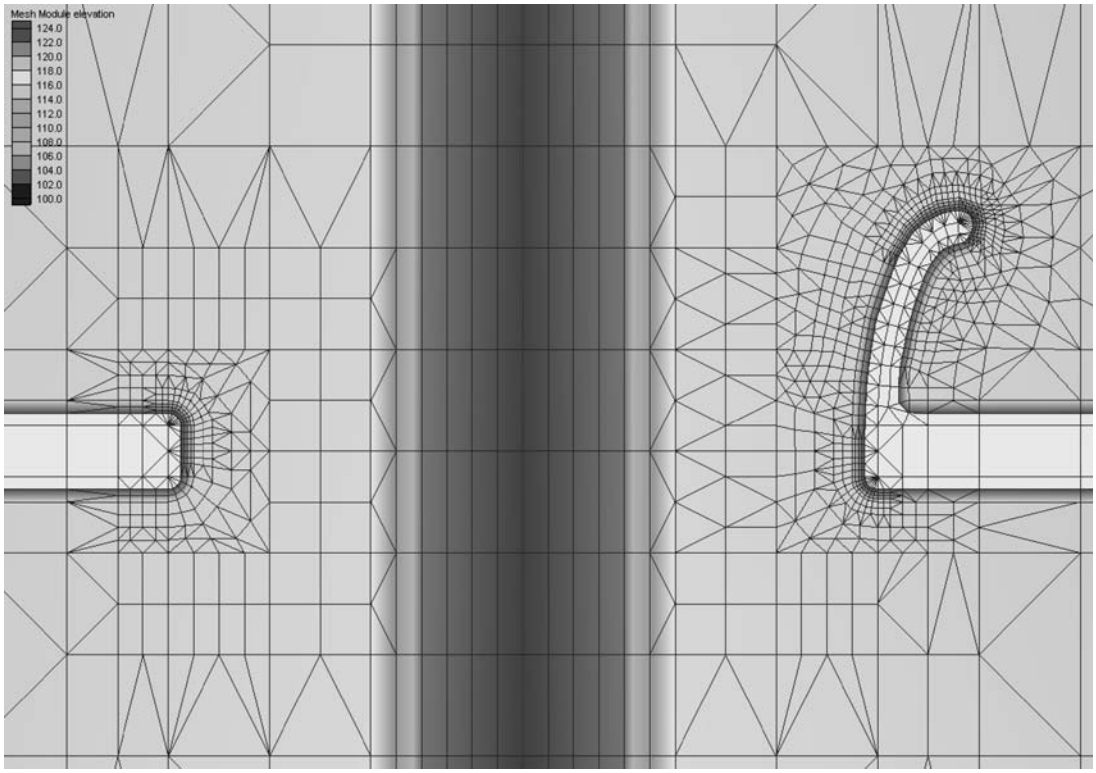


Figure 3.47. Bridge opening area for the base model.

Figure 3.48. Flow field in the base model bridge opening.

Table 3.14. Two-dimensional models for abutment velocities.

Model Run	Discharge (cfs)	Floodplain Manning n	Guide Bank Length (ft)	Abutment Setback (ft)	Floodplain Width (ft)	Channel and Valley Slope
Base Model	55000	0.12	200	175	2200	0.0002
Model 1	30000					
Model 2	80000					
Model 3		0.06				
Model 4		0.09				
Model 5		0.15				
Model 6			150			
Model 7			250			
Model 8				25		
Model 9				75		
Model 10				125		
Model 11				225		
Model 12					1000	
Model 13					3000	
Model 14						0.0001
Model 15						0.0004
Model 16			150	75	1000	
Model 17	80000	0.09			3000	
Model 18	80000	0.06		125	3000	
Model 19		0.15	150	75	1000	
Model 20		0.15	150	125	1000	
Model 21		0.15	150		1000	

Notes: Blanks indicate base model values.
Channel Manning n was 0.025 for all runs.
Right-of-Way Manning n was 0.35 for all runs.

water surface for each model was set at normal depth. For each model, the simulated velocity (produced by the model) at the toe of slope of the abutment was recorded and compared with the velocity computed by the SBR method.

Table 3.15 shows the numerical model results and the computed velocities using the SBR method. In general, the SBR method performed well in predicting the abutment flow velocity. The SBR method uses either the average velocity in the bridge opening (SBR ≤ 5) or the upstream floodplain discharge divided by setback area (SBR > 5). The velocity that meets the SBR criteria for each simulation is identified in **bold**. The 2-D model velocity (highlighted column) is also shown in Table 3.15 and is considered the best estimate of the actual velocity at the abutment. Also shown in Table 3.15 is the Froude number at the abutment, the required riprap size using Equation 2.20 or 2.21, as well as the results for the guide bank.

Whenever the SBR is less than 5, the average velocity in the bridge opening provides a good estimate for the velocity at the abutment. In four cases, the SBR method overestimates velocity by more than 20% (Models 3, 10, 17, and 18). Each of these cases has an SBR of greater than 5 so the method of dividing upstream floodplain flow by the set back area was used. For the extreme case (Model 18), this calculation predicts a velocity of 26.1 ft/s (8 m/s) when the model velocity was 12.9 ft/s (3.9 m/s). Therefore, one final adjustment to the SBR method is warranted. **When the SBR is greater than 5, the recommended adjustment is to compare the velocity from the SBR method to the maximum velocity in the channel within the bridge opening and select the lower velocity**. Using this modified SBR method, significant overestimations of abutment velocity are avoided. Figure 3.49 shows the computed abutment velocity using the SBR and modified SBR methods plotted versus the "observed" velocity at the abutment from the 2-D models. In the five cases where the modified SBR method was used (maximum velocity in the channel was less than the SBR method velocity), the estimate was better and in three cases significantly better.

In conclusion, the SBR method is well suited for estimating velocity at an abutment if the estimated velocity does not exceed the maximum velocity in the channel. Table 3.15 shows the computed Froude number and required riprap size based on the observed 2-D model velocities using Equations

Table 3.15. Comparison of abutment and guide bank velocities.

Model Run	Set Back Ratio	Average velocity for bridge opening (ft/s)	Velocity computed for setback area (ft/s)	2-D model Velocity at abutment toe of slope (ft/s)	Maximum velocity in channel under bridge (ft/s)	Froude Number at abutment toe of slope	Required riprap size for spill-through abutment (in)	2-D model Velocity at guide bank head (ft/s)	Froude Number at guide bank toe of slope	Required riprap size for guide bank using spill-through abutment equation (in)
Base Model	8.6	6.4	**7.7**	8.2	9.5	0.53	13.4	5.8	0.38	6.7
Model 1	10.6	4.9	**3.5**	3.5	6.8	0.34	2.5	2.6	0.25	1.4
Model 2	7.5	7.5	**11.6**	10.2	11.0	0.60	21.0	7.8	0.46	12.2
Model 3	9.3	7.3	**12.5**	8.5	9.9	0.63	14.4	7.1	0.53	10.1
Model 4	8.9	6.7	**9.4**	8.0	9.6	0.55	12.7	6.1	0.42	7.6
Model 5	8.4	6.1	**6.5**	7.4	9.3	0.46	11.0	5.5	0.34	6.1
Model 6	8.6	6.4	**7.7**	7.6	9.4	0.50	11.7	6.1	0.40	7.5
Model 7	8.6	6.4	**7.7**	7.6	9.5	0.50	11.7	5.4	0.35	5.8
Model 8	1.2	**8.6**	46.4	9.1	11.0	0.54	16.6	7.0	0.42	9.9
Model 9	3.7	**7.7**	17.4	8.3	10.2	0.53	13.9	6.7	0.43	9.0
Model 10	6.2	6.9	**10.7**	7.9	9.8	0.51	12.6	6.2	0.40	7.6
Model 11	11.1	5.9	**6.0**	7.4	9.2	0.48	10.9	5.5	0.36	6.0
Model 12	8.0	5.7	**4.6**	6.2	8.5	0.36	7.7	4.5	0.26	4.0
Model 13	8.8	6.6	**8.9**	8.0	9.8	0.53	12.7	6.1	0.41	7.5
Model 14	7.6	5.2	**6.7**	7.2	7.8	0.39	10.4	5.3	0.29	5.6
Model 15	9.6	7.7	**8.1**	8.3	11.0	0.65	13.9	5.8	0.46	6.8
Model 16	3.5	**7.0**	10.5	7.1	9.3	0.41	10.2	5.5	0.32	6.1
Model 17	8.1	8.4	**14.2**	11.2	11.8	0.67	25.3	8.9	0.53	16.0
Model 18	6.1	10.0	**26.1**	12.9	12.7	0.83	31.5	10.7	0.69	22.8
Model 19	3.4	**6.7**	8.8	6.7	9.1	0.37	9.0	5.1	0.28	5.1
Model 20	5.6	6.0	**5.4**	6.3	8.8	0.35	7.9	4.6	0.26	4.2
Model 21	7.9	5.6	**4.6**	6.2	8.4	0.36	7.6	4.5	0.26	4.1

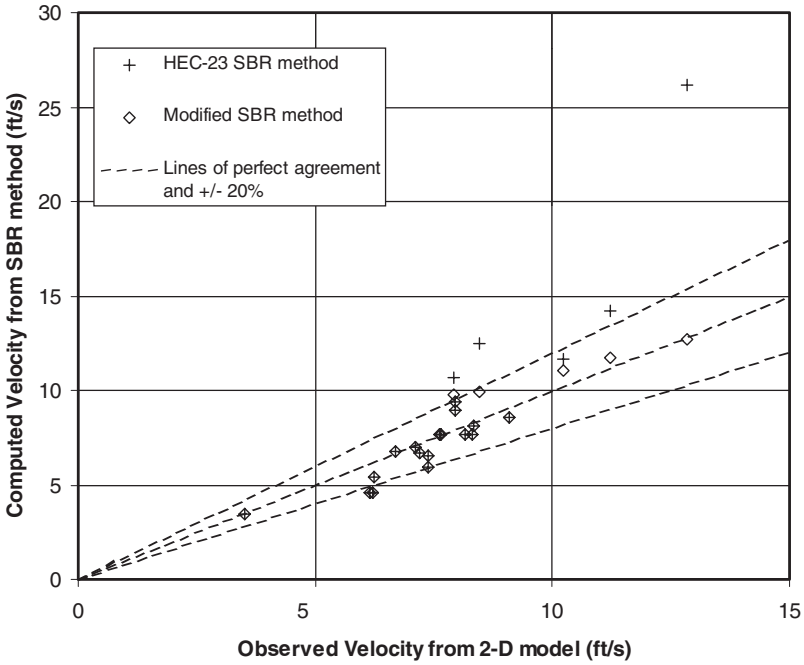


Figure 3.49. Comparison of abutment flow velocities from 2-D modeling versus SBR methods.

2.20 and 2.21. In only one case was the Froude number greater than 0.8 (Model 18 with a Froude number of 0.83), which required the use of Equation 2.21. The riprap sizes appear reasonable (2.5 to 31.5 in [64 to 800 mm]) given the range of flow velocities encountered (3.5 to 12.9 ft/s [1.1 to 3.9 m/s]).

3.4.2 Filter Requirements

The guidance provided for filters in Section 3.2.2 is generally appropriate for riprap installations at bridge abutments located on floodplains and set back from the main channel.

In the case where the abutment is integral with the bank or extends into the main channel and a riprap apron is installed, the same concern regarding the use of granular filters exists as discussed in Section 3.3.2 for pier riprap (see also the discussion of NCHRP Project 24-18A in Section 2.4.6). That is, if dune troughs passing the abutment are deeper than the riprap apron thickness, the underlying finer particles of a granular layer can be rapidly swept away. The result is that the entire riprap installation becomes progressively destabilized beginning at the periphery and working in toward the abutment. **For this reason, it is strongly recommended that only geotextile filters be used at bridge abutments in riverine systems where dune-type bed forms may be present during high flows, and where the abutment and/or abutment riprap apron extend into the main channel**. In addition, the geotextile filter should not be extended to the periphery of the riprap apron, but instead should terminate at two-thirds the riprap extent.

3.4.3 Material and Testing Specifications

The requirements for the quality and characteristics of riprap materials, and the associated tests to support those requirements, are presented for revetment riprap installations in Section 3.2.3. These requirements are suitable for use with riprap at bridge abutments as well.

3.4.4 Construction/Installation Guidelines

Guidelines for constructing and installing revetment riprap and filters are provided in Section 3.2.4. Generally, construction of an abutment that encroaches into the main channel is not desirable. If abutment protection is required at a new or existing bridge that encroaches into the main channel, then riprap toe down or a riprap key should be considered (see Figures 3.34 and 3.35). The guidelines for riprap installations at abutments are similar both for placement in the dry as well as underwater.

3.4.5 Inspection and Quality Control

Guidance provided for inspection and quality control for revetment riprap installations in Section 3.2.5 is appropriate for abutment riprap installations. The inspection code included in Appendix D accommodates abutment riprap as well as other types of riprap applications.

3.5 Riprap for Countermeasures

In general, design guidelines and specifications for riprap for countermeasures are similar to those for bankline revetment or abutments. In this section, recommendations for revetment riprap are adapted to the countermeasure application. Guidance for sizing and placing riprap at zones of high stress on countermeasures (e.g., the nose of a guide bank or spur) is investigated. The feasibility of using an abutment-related characteristic average velocity for countermeasure riprap sizing is evaluated, and a recommended equation for sizing riprap under overtopping conditions on the embankment portion of a countermeasure is provided. Failure of riprap under overtopping flow conditions on linear countermeasures is discussed in Section 3.7.5. Guidance from USACE is cited for sizing riprap for spurs.

Filter requirements, material and testing specifications, construction and installation guidelines, and inspection and quality control procedures are discussed with reference to guidelines and specifications for revetment riprap.

3.5.1 Design Equations

Parallel Flow on Guide Banks

Section 3.4.1 describes the 2-D modeling that was used to evaluate the SBR approach to computing characteristic average velocities for flow around abutments. The characteristic average velocity is an estimate of the actual flow velocity around an abutment due to severe roadway embankment encroachment on a floodplain. Figure 3.48 shows the model flow field around the abutment and guide bank for the base model of a set of 22 models that were used to evaluate the SBR method. Table 3.15 shows the results of these models. In general, the two areas of high velocity that the modeling shows at guide banks are at the head of the guide bank and at the downstream end of the spill-through slope at the exit of the bridge. The velocities at these two locations are, on average, approximately 77% of the velocity at the abutment on the other end of the bridge. Figure 3.50 shows the model velocities at the two guide bank locations plotted versus the velocity at the abutment. This figure indicates that the modified SBR method provides a reasonable estimate of guide bank flow velocity when it is reduced by a factor of 0.77.

Because guide banks are designed to protect abutments from deep scour by providing a smooth flow transition through the bridge, it is reasonable to use the abutment riprap equations (Equations 2.20 and 2.21) for guide banks. **It is recommended that the riprap size be computed using 0.85 times the characteristic average velocity computed using the modified SBR approach discussed in Section 3.4.1**. This reduced velocity results in a 28% reduction in riprap size and envelops the data (+10% line) in Figure 3.50. Table 3.15 shows the computed velocity, Froude number, and required riprap size for the guide banks simulated in the 22 2-D models. It is interesting to note that the amount of backwater produced by the guide bank side of the model was slightly higher than the backwater produced by the abrupt abutment side even though the guide bank produced a smooth flow transition through the bridge.

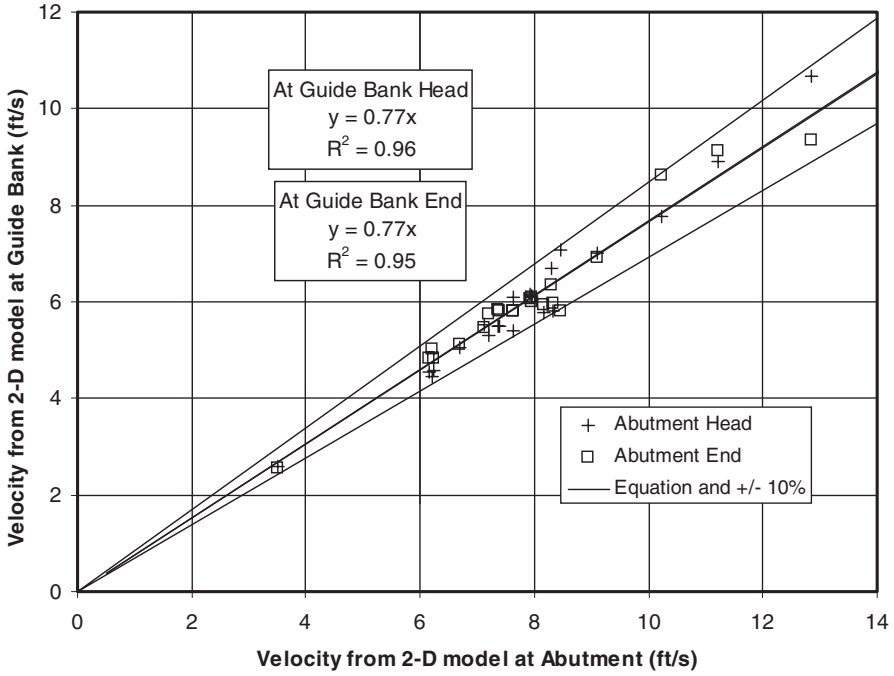


Figure 3.50. Comparison of velocity at an abutment versus velocity at a guide bank.

Overtopping Flow on Embankments

Section 2.4.4 summarizes findings on riprap design under overtopping flow conditions. Under a 1991 cooperative agreement, the U.S. Bureau of Reclamation (USBR) and Colorado State University (CSU) built a near-prototype size embankment overtopping research facility with a 50% slope (2H:1V). Angular riprap tests were conducted in the summers of 1994, 1995, and 1997 on this facility (Mishra, 1998). Failure was defined as removal of the riprap by erosion and movement of rock until bedding material was exposed.

The first two riprap test sections covered the full width of the chute and extended 60 ft (18.29 m) down the slope from the crest. The first test (1994) consisted of an 8-in (203-mm) thick gravel bedding material with a 2-ft (0.61-m) overlay of large riprap with a d_{50} of 15.2 in (386 mm) (see Figure 3.51). The second test (1995) utilized the first test bed with a second layer of approximately 2-ft (0.61-m) thick riprap with d_{50} of 25.8 in (655 mm).

The third test (1997) covered the full width of the chute and extended 100 ft (30.48 m) from the crest down the slope to the toe of the facility. An 8-in (203-mm) thick gravel bedding material with a d_{50} of 1.8 in (48 mm) was overlayed with a main riprap layer of thickness 21 in (533 mm) with a d_{50} of 10.7 in (271 mm). A berm was built at the bottom of the flume to simulate toe treatment at the base of the embankment. The configuration of the test setup in 1997 is given in Figure 3.52.

Figure 3.51. Test set up for 1994 (d_{50} = 15.2 in [386 mm]).

Figure 3.52. Test set up for 1997 (d_{50} = 10.7 in [271 mm]).

For all the tests, a gabion composed of the same rocks used on the slope was placed at the crest of the embankment, to provide a smooth transition of water from the head box to the embankment and to prevent premature failure of the riprap at the transition between the concrete approach at the crest of the embankment and the concrete chute. The top surface of the gabion was horizontal (see Figure 3.53).

The test series provided the opportunity to gather important data regarding flow through large-size riprap. Observations provided information on aeration, interstitial flow, stone movement, and the failure mechanism on the slope. Data were collected on discharge flowing down the chute through the riprap, the head box depth for overtopping heads, manometer readings for depth of flow down the chute and the pressure heads, and electronic recording of electrical conductivity versus time to determine interstitial velocities.

Estimating flow through rockfill can be a useful procedure for designing riprap. The velocity of water flowing through the rock voids helps determine the depth of water flowing through the riprap, which could be the governing factor in the riprap design for overtopping flow. In some cases, determining how much water can flow through the

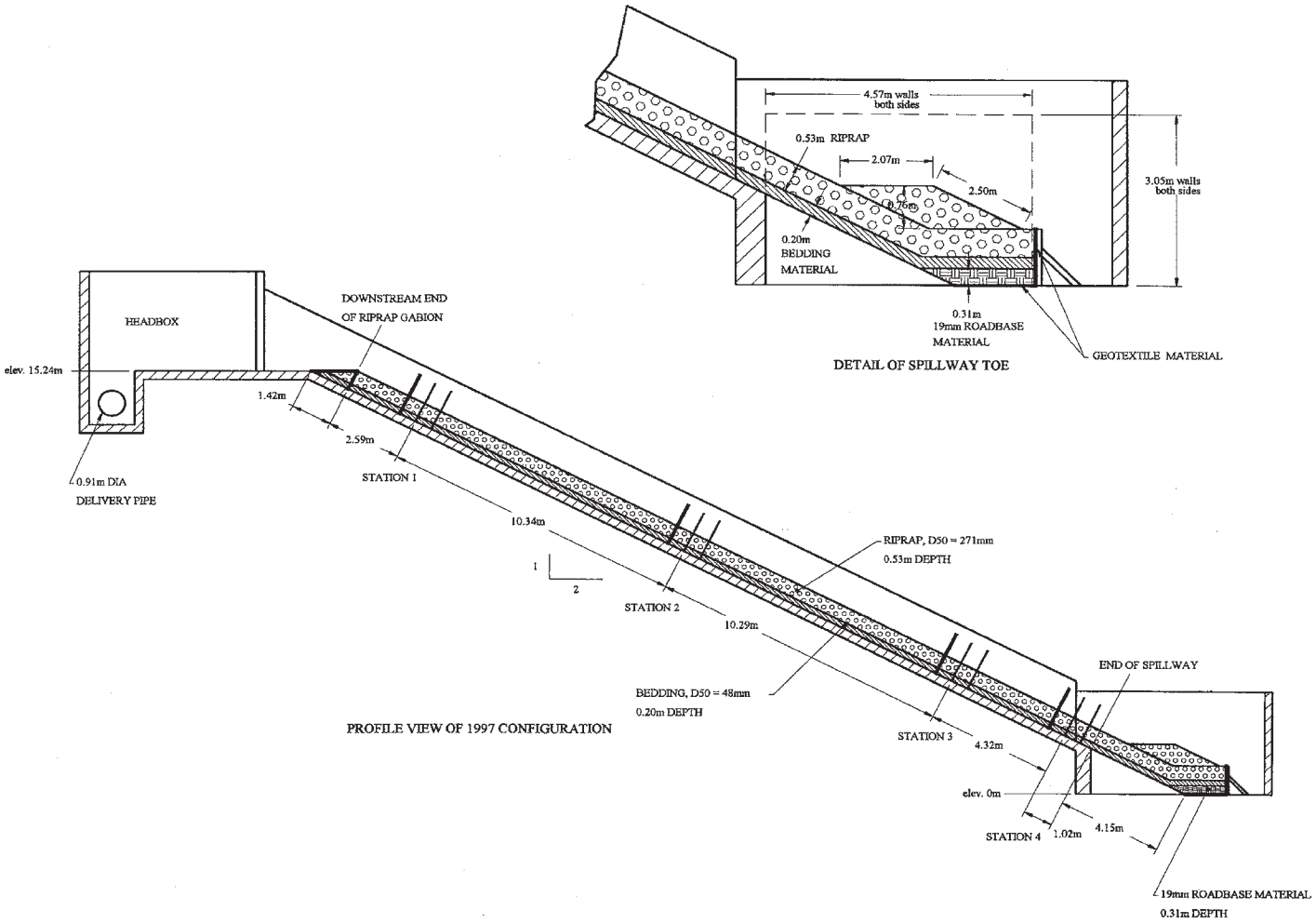


Figure 3.53. Riprap configuration in 1997.

riprap layer is necessary in order to determine the amount of water that will flow on the surface of the riprap. Consequently, accurate prediction of the interstitial velocity of water flowing through a rockfill is important. A predictive equation developed by Abt et al. (1991) considers the size of the riprap and the slope of the embankment for predicting the interstitial velocity of water through the riprap; however, it does not consider the effect of the rock layer gradation.

The interstitial velocity of water is strongly influenced by the void sizes inside the rock layer (Figure 3.54). The void sizes are determined by the gradation of the rock. For this application the coefficient of uniformity C_u (d_{60}/d_{10}) provides a good representation of the rock gradation and should be a factor in the predictive equation for the interstitial velocity of water. The USBR/CSU studies, which took into account the data obtained from previous studies, did show that the predictive equation by Abt et al. (1991) underpredicts the interstitial velocity for large riprap (Mishra 1998). The equation for interstitial velocity developed in the USBR/CSU study is

$$\frac{V_i}{\sqrt{(gd_{50})}} = 2.48C_u^{-2.22}S^{0.58} \qquad (3.19)$$

where

V_i = Interstitial velocity, ft/s (m/sec)
d_{50} = Median rock size diameter, ft (m)
C_u = Coefficient of uniformity given by d_{60}/d_{10}
S = Slope of embankment, ft/ft (m/m)

Figure 3.54. Interstitial flow through rock layer.

g = Acceleration due to gravity = 32.2 ft/s^2 (9.81 m/s^2)

Well-graded rock was established to have a better stability in situations of overtopping than uniformly graded rocks not only because the interlocking mechanism is better in well-graded riprap, but also because the interstitial velocity of water is much higher in a uniformly graded rock layer.

A universal riprap design equation was derived based on the original Shield's parameter taking into consideration the effects of the gradation of the rock layer, slope of the embankment, and the unit discharge (Equation 3.20). The riprap design equation was found to satisfactorily predict the size of the riprap to be used for a specified unit discharge and a given embankment slope. The comparison of the experimental data obtained by different research groups with the design equation curves is shown in Figure 3.55.

$$d_{50}C_u^{0.25} = K_u(q_f)^{0.52}S^{-0.75} \times \left(\frac{\sin\alpha}{(S_s\cos\alpha - 1)(\cos\alpha\tan\varphi - \sin\alpha)}\right)^{1.11} \quad (3.20)$$

d_{50} = Median rock size diameter, ft (m)
C_u = Coefficient of uniformity given by d_{60}/d_{10}
K_u = 0.525 English units
= 0.55 SI units
q_f = Unit failure discharge in ft^3/s/ft (m^3/s/m)
S = Slope of the embankment, ft/ft (m/m)
S_s = Specific gravity of rock (2.65 for most cases)
α = Slope of the embankment, degrees
φ = Angle of repose of the riprap material

The USBR/CSU study resulted in a step-by-step method, by which not only the size of the riprap, but also the thickness of the riprap layer, can be designed in a rational method utilizing the predicted interstitial velocity through the rock layer along with the universal overtopping riprap design equation. The method is illustrated in Section 6 of Appendix C with example problems.

Riprap on Spurs

Riprap for spurs must be designed to account for the higher flow velocity and scour that occur around the nose of the spur. FHWA provides guidance on spur design in HEC-23 (Lagasse et al., 2001) and in a report titled "Design of Spur-Type Streambank Stabilization Structures" (Brown, 1985a). HEC-23 and Brown provide guidance on spur type, location, spacing, and orientation. Brown provides no guidance on sizing riprap for use on spurs, but states that the worst-case location is at the nose of the spur and the worst-case condition is before spur overtopping. Because spur crests are at or below the channel bank, the design condition would be an in-channel flow. HEC-23 suggests that a revetment equation could be used for riprap sizing. If a revetment equation is used, then either the factor of safety should be increased or a higher velocity (than the channel average) should be used in the design. The EM 1601 equation can be used to size riprap at spurs by selecting a C_v value of 1.25 (Equation 3.1). Use of the abutment riprap equation may also be reasonable, but only if a good estimate can be made of the local velocity at the nose of the spur. There is no equivalent to the SBR method (as is recommended with guide banks) for estimating this local velocity, so either judgment, physical modeling or 2-D modeling would have to be used to estimate this velocity.

Figure 3.56 shows a portion of a 2-D model to illustrate the velocity increase around the end of a spur. The 2-D model was developed using FESWMS to simulate hydraulic conditions for a river geometry that includes an eroding

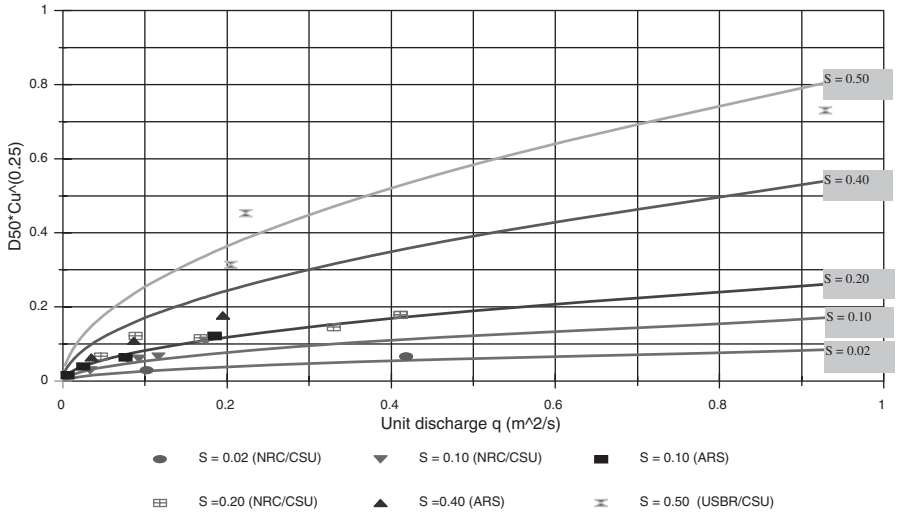


Figure 3.55. Comparison of experimental data with design curves.

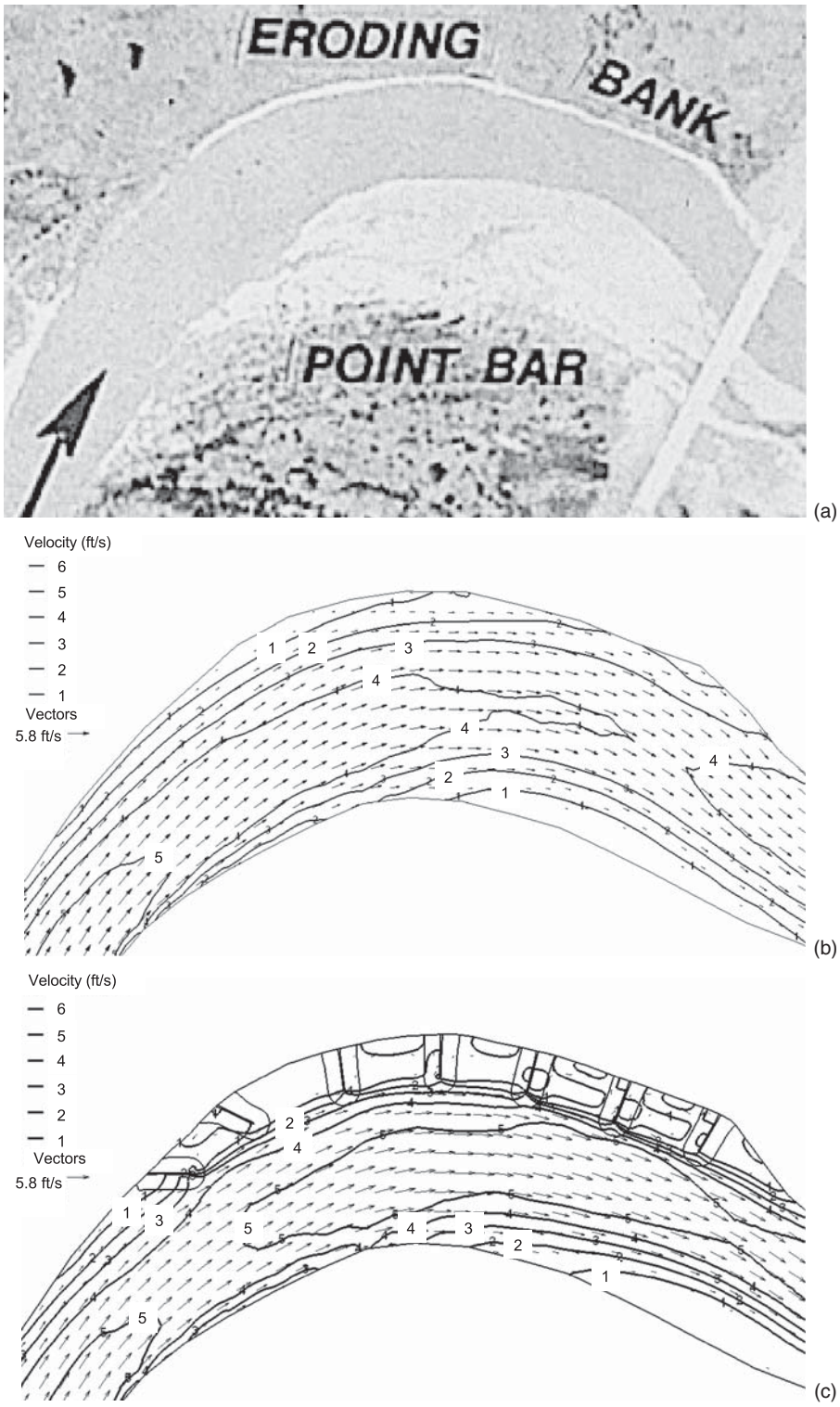


Figure 3.56. Two-dimensional analysis of flow along spurs: (a) river aerial photograph, (b) flow field without spurs, and (c) flow field with spurs (velocity contours in ft/s)

bank where spurs would be an effective countermeasure. The model was run for bankfull flow as this is the worst-case condition identified by Brown (1985a). Figure 3.56 shows three plan views of the river upstream of a bridge that is being adversely impacted by upstream bank erosion and channel lateral migration. The plan views are an aerial photograph of the channel upstream of a threatened bridge (Figure 3.56(a)), the baseline 2-D model results (without

spurs) showing velocity contours and velocity vectors (Figure 3.56(b)), and the 2-D model results with spurs (Figure 3.56(c)). The spurs produce lower flow velocities along the eroding bank and align the flow to the bridge opening. Higher flow velocities, which would trim back the point bar and cause scour, are produced along the channel centerline and along the nose of each spur.

For these simulations, the maximum computed flow velocity at the nose of a spur approaches 5 ft/s (1.5 m/s), which exceeds the maximum flow velocity computed in the bend of the baseline model. Although this model is insufficient for developing any specific guidance on estimating design velocities for spurs (such as the SBR method for guide banks), it does illustrate that spurs are subjected to high flow velocity and scour and that they are effective at reducing flow velocity along the bank line. The models illustrate that a locally high velocity is expected at the nose of a spur and that the riprap size and/or volume would need to be increased to withstand this velocity and the resulting local scour.

3.5.2 Filter Requirements

The guidance provided for filters in Section 3.2.2 of this document is generally appropriate for countermeasures constructed of or armored by riprap, such as guide banks, spurs, or bendway weirs.

3.5.3 Material and Testing Specifications

The requirements for the quality and characteristics of riprap materials, and the associated tests to support those requirements, are presented for revetment riprap installations in Section 3.2.3. These requirements are suitable for use with riprap used to construct or armor scour countermeasures.

3.5.4 Construction/Installation Guidelines

Guidelines for constructing and installing revetment riprap and filters are provided in Section 3.2.4. The guidelines for riprap-based countermeasures are similar both for placement in the dry as well as under water.

3.5.5 Inspection and Quality Control

Guidance provided for inspection and quality control for revetment riprap installations in Section 3.2.5 is appropriate for riprap-based countermeasures. The inspection code included in Appendix D accommodates countermeasure riprap as well as other types of riprap applications.

3.6 Riprap Design Variables

Each of the riprap size equations depends on the results of a hydraulic analysis to provide velocity and depth as input. Velocity is the primary hydraulic variable for determining riprap size, with depth having little or no effect on the computed size. The level of hydraulic analysis should be commensurate with the importance of the facility. The complexity of the hydraulic conditions may also require the use of more advanced 2-D or physical hydraulic models to reduce the uncertainties related to the design. The following subsections provide guidance on determining the hydraulic input for riprap sizing.

A set of riprap design (sizing) examples was developed for revetment, bridge pier, bridge abutment, guide bank, and spur riprap. These design examples use the software/spreadsheet reference data sets from Section 3.9.2 as given data and are presented in Appendix C, Guidelines for the Design and Specification of Rock Riprap Installations. In addition, a filter design example and an overtopping flow design example are included in Appendix C.

3.6.1 Design Flood Frequency

The design flood frequency for new bridge facilities varies based on the type of roadway (i.e., secondary, primary, interstate) and the volume of traffic. The level of service is often defined as the flood frequency when road overtopping occurs, although there may be some amount of freeboard for the road grade above this water surface elevation. The design frequency used for foundations may differ from other bridge and roadway components because scour is computed based on the 100- and 500-year floods. Lower discharges may be used, such as the incipient road overtopping condition, if more severe scour is anticipated. Countermeasures are not recommended for piers at new bridges because the design should incorporate scour exposure. Abutment protection is the most common countermeasure at new bridges (abutment or guide bank riprap). The most severe scour event with a frequency up to the 100-year flood should be used as the design frequency for abutment protection at a new bridge. The design should be checked for the most severe scour event between the 100- and 500-year floods, but with a factor of safety of 1.0 (Richardson and Davis, 2001).

For countermeasures at existing bridges, flood frequency criteria established for new bridges may not be justified, because the remaining service life of the bridge may be insufficient to justify the cost of countermeasures designed for a 100-year event. For countermeasures at existing bridges with limited remaining service life, the concept of risk (Pearson et al., 2000) should be considered when selecting the type and level of protection. The risk analysis compares the cost of the countermeasure installation to the risk. The risk is computed as the estimated cost of a failure times the probability of failure during the remaining bridge service life. For existing bridges with significant remaining service life, a risk analysis may indicate that providing protection up to the 100-year or

even 500-year flood may be justified. For these bridges, the life cycle costs should be evaluated to determine which countermeasure option is most effective.

Spurs may have a design flood frequency much lower than the bridge design. Spurs may be subjected to the highest flow velocity before overtopping of the spur field (Brown, 1985a). Since the crest elevation of spurs is at or below the bank elevation, the design flow should be an in-channel flow.

3.6.2 Hydraulic Analysis

The level of hydraulic analysis should be sufficient to provide reasonable estimates of the hydraulic variables that are required to size the riprap, estimate scour, and to assess environmental and flooding impacts. Because velocity is much more important than depth, the hydraulic analysis should be performed using input parameters that yield reasonable, yet conservative estimates of velocity. The level of analysis should be commensurate with the importance of the structure and the cost of the countermeasure. If the countermeasure is located in a hydraulically complex area, 2-D or physical modeling may be required. Two-dimensional models may also be necessary to obtain accurate flow divisions between the main channel and relief (overflow) structures.

The results of any hydraulic analysis should be reviewed carefully before being used to size riprap. For pier riprap, the maximum channel velocity should be used if there is potential for thalweg shifting. For abutment riprap, 1-D models do not provide a good estimate of flow velocity at the toe of an abutment. The SBR method provides a reasonable estimate of this velocity (see Sections 2.4.3 and 3.4.1). Two-dimensional flow models may also provide good estimates of the flow velocity at an abutment, but only if the finite element network is very detailed in the vicinity of the abutment. If a 2-D model does not include sufficient detail for flow around an abutment to provide the design velocity directly, then the SBR method should be used to estimate the design velocity.

3.6.3 Velocity Multipliers

Each of the riprap sizing equations relies primarily on velocity for determining riprap size, and each of the equations requires specific adjustments to the velocity for use in the equation. For pier riprap, the flow velocity just upstream of the pier but outside the influence of the pier and including the constriction caused by the bridge should be used. If the channel average velocity is used, then it should be increased to account for velocity variation within the channel. Often the maximum velocity in the channel is used for design purposes to account for channel shifting. To account for flow acceleration around the pier, the added turbulence, and the horseshoe vortex that forms at the base of the pier, another velocity adjustment of 1.7 is used for square piers and 1.5 is used for circular piers.

Although there is not a specific adjustment factor for the velocity used for abutment riprap sizing, obtaining an accurate estimate of the flow velocity is not a trivial matter. The SBR method for estimating flow velocity at the abutment accounts for flow conditions upstream of the bridge and the proximity of the abutment to the channel bank. The computed velocity may be significantly higher than the velocity computed using HEC-RAS or another 1-D model. Two-dimensional modeling results indicate that guide bank riprap can be designed using 0.85 times the velocity computed for an abutment using the SBR method (Section 3.4.1).

The recommended revetment riprap sizing equation (from EM 1601) uses the average flow velocity that has been adjusted based on the ratio of the channel radius of curvature to channel width. For tight bends, this adjustment can be very significant (Section 3.2.1). A special case of revetment riprap is designing riprap for overtopping flow on steep slopes. Estimating the interstitial velocity of flow through the riprap is required for this application (Section 3.5.1).

A riprap spur will also produce a locally high velocity around the end of the spur. The EM 1601 equation can be used to size riprap for a spur. A C_v factor of 1.25 is recommended for spur applications (Equation 3.1).

Unlike the revetment equations (e.g., EM 1601), the abutment and pier equations do not include any explicit factor of safety. The pier equation does, however, include an unspecified amount of conservatism built in and the abutment equations probably do as well. Even the EM 1601 equation was developed to envelop most of the laboratory data, even without an additional factor of safety. However, if there is a great deal of uncertainty in the hydraulic analysis, the designer should consider incorporating this uncertainty when sizing riprap. Because velocity is the most important variable for determining riprap size, the most direct way of incorporating uncertainty is to compute the riprap size after increasing velocity by the level of uncertainty (e.g., 10%).

3.7 Riprap Failure Mechanisms

A fundamental premise of this study is that riprap is an integrated system and that successful performance of a riprap installation depends on the response of each component of the system to hydraulic and environmental stresses throughout its service life. A detailed examination of riprap failure mechanisms underscores the integrated nature of riprap armoring systems and supports development of inspection guidance (see Section 3.2.5). Selected case studies of failures are used to emphasize the need for post-flood/post-construction inspection of riprap installations.

3.7.1 Modes of Revetment Riprap Failure

In a preliminary evaluation of various riprap design techniques, Blodgett and McConaughy (1986) concluded that a

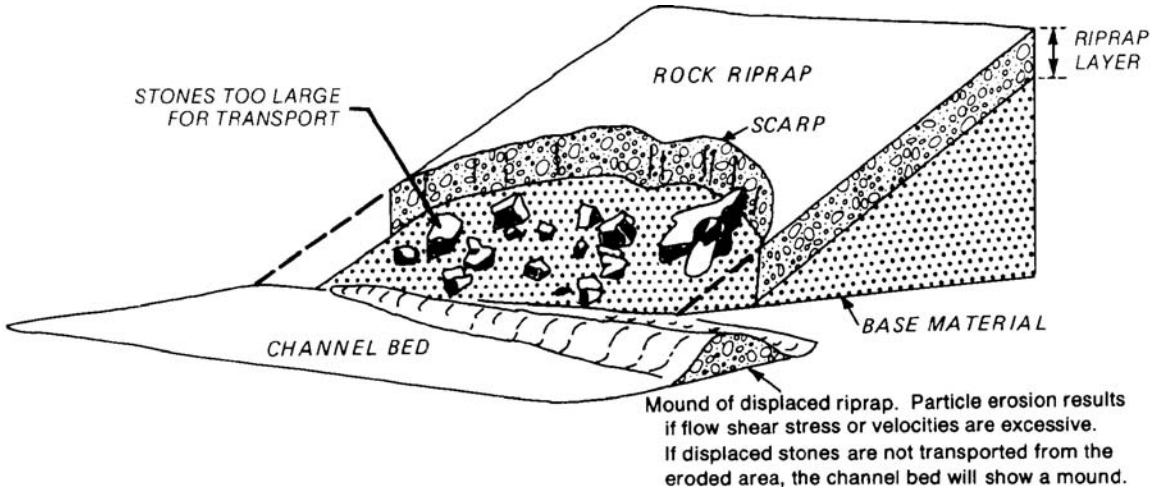
major shortcoming of all present design techniques is their assumption that failures of riprap revetment are due only to particle erosion. Procedures for the design of riprap protection need to consider all the various types of failures: (1) particle erosion, (2) translational slide, (3) modified slump, and (4) slump. These types of failure are illustrated in Figures 3.57 through 3.60.

Particle erosion is the most commonly considered erosion mechanism (Figure 3.57). Particle erosion occurs when individual particles are dislodged by the hydraulic forces generated by the flowing water. Particle erosion can be initiated by abrasion, impingement of flowing water, eddy action/reverse flow, local flow acceleration, freeze/thaw action, ice, or toe erosion. Probable causes of particle erosion include (1) stone size not large enough; (2) individual stones removed by impact or abrasion; (3) side slope of the bank so steep that the angle of repose of the riprap material is easily exceeded; and (4) gradation of riprap too uniform.

A translational slide is a failure of riprap caused by the downslope movement of a mass of stones, with the fault line on a horizontal plane (Figure 3.58). The initial phases of a translational slide are indicated by cracks in the upper part of the riprap bank that extend parallel to the channel. This type of riprap failure is usually initiated when the channel bed scours and undermines the toe of the riprap blanket. This failure could be caused by particle erosion of the toe material or by some other mechanism that causes displacement of toe material. Any other mechanism which would cause the shear resistance along the interface between the riprap blanket and base material to be reduced to less than the gravitational force could also cause a translational slide. It has been suggested that the presence of a filter blanket may provide a potential failure plane for translational slides. Probable causes of translational slides are (1) bank side slope too steep, (2) presence of excess hydrostatic (pore) pressure, and (3) loss of foundation support at the toe of the riprap blanket caused by erosion of the lower part of the riprap blanket.

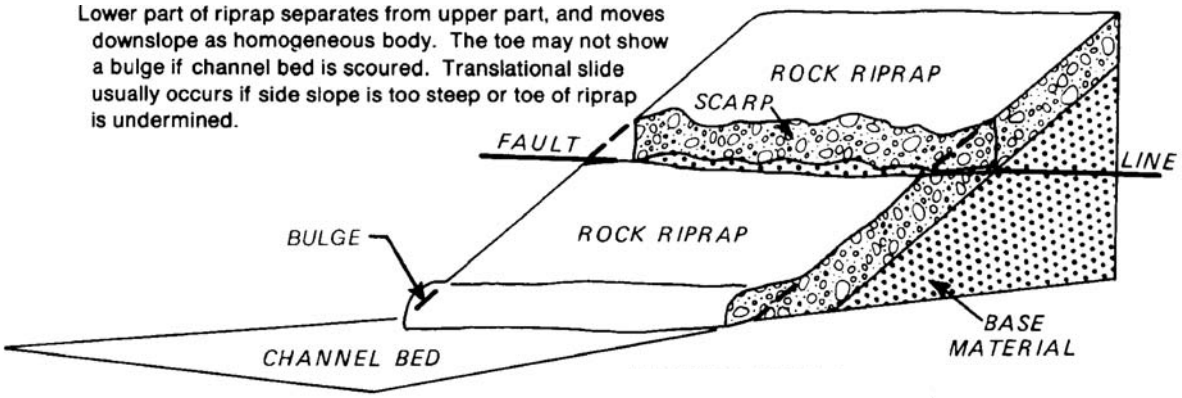
Modified slump failure of riprap (Figure 3.59) is the mass movement of material along an internal slip surface within the riprap blanket. The underlying material supporting the riprap does not fail. This type of failure is similar in many respects to the translational slide, but the geometry of the damaged riprap is similar in shape to initial stages of failure caused by particle erosion. Probable causes of modified slump are (1) bank side slope is so steep that the riprap is resting very near the angle of repose, and any imbalance or movement of individual stones creates a situation of instability for other stones in the blanket and (2) material critical to the support of upslope riprap is dislodged by settlement of the submerged riprap, impact, abrasion, particle erosion, or some other cause.

Slump failure is a rotational-gravitational movement of material along a surface of rupture that has a concave upward curve (Figure 3.60). The cause of slump failures is related to shear failure of the underlying base material that supports the riprap. The primary feature of a slump failure is the localized displacement of base material along a slip surface, which is usually caused by excess pore pressure that reduces friction along a fault line in the base material. Probable causes of slump failures are (1) nonhomogeneous base material with layers of impermeable material that act as a fault line when subject to excess pore pressure, (2) side slopes too steep and gravitational forces exceeding the



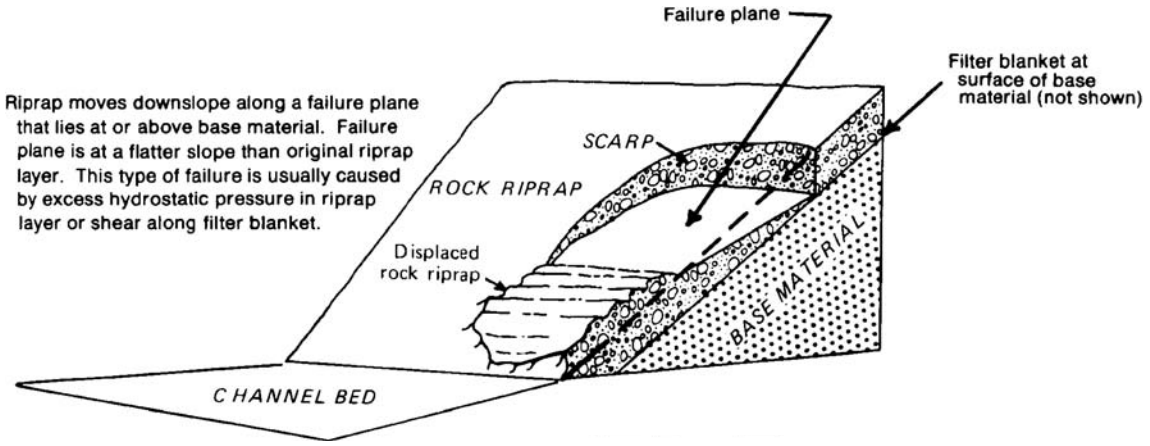
Source: Blodgett and McConaughy (1986)

Figure 3.57. Riprap failure due to particle erosion.



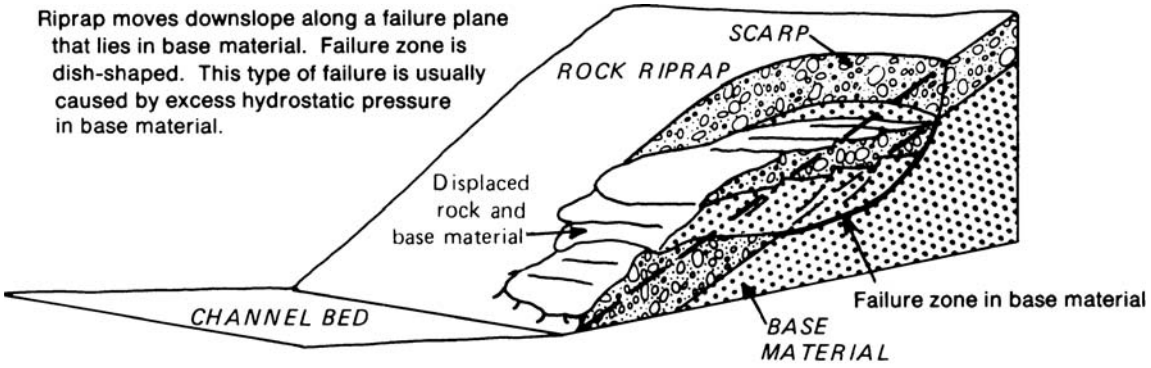
Source: Blodgett and McConaughy (1986)

Figure 3.58. Riprap failure due to translational slide.



Source: Blodgett and McConaughy (1986)

Figure 3.59. Riprap failure due to modified slump.



Source: Blodgett and McConaughy (1986)

Figure 3.60. Riprap failure due to slump.

inertia forces of the riprap and base material along a friction plane, and (3) too much overburden at the top of the slope (may be caused in part by the riprap).

Because of the general effectiveness of dumped riprap, a more detailed analysis of the relatively small number of cases in which it failed has been presented by Brice and Blodgett (1978a). The principal causes of failure and methods of mitigation are given in Table 3.16.

Removal of toe material through development of a scour hole is often cited as the most common mechanism for initiating streambank failure. Historically, bank slopes were generally provided with protection while nothing was done to protect the toe. The results were predictable: formation of a scour hole followed by sloughing-in of the armored bank (Brown, 1985b, 1985c; Galay et al., 1987).

Toe protection is most commonly provided by a "launching" apron, which involves use of a material that can readily conform to a scour hole while at the same time maintaining its integrity and ability to protect underlying bank material. Thus, in the case of aprons, there is a need for toe protection material to be flexible. The most frequent problems that arise in designing aprons are in assessing scour depth potential, arriving at armor material sizes, and using material that is not flexible (Galay et al., 1987). Generally, the approach is to provide a thickened toe or the extension of a single stone layer down to the expected maximum scour depth (see Figures 3.34 and 3.35).

Galay et al. (1987) notes that failure of a riprap blanket is probably from one of the following reasons:

- Inadequate stone sizes
- Poor end treatment (keys)
- Lack of an apron or an insufficient volume of apron stone
- Poor stone durability
- Lack of a filter
- Outflanking of the riprap blanket

According to Galay et al. (1987), the lack of detailed and comprehensive design and construction guidelines for this, the most popular, method of protecting streambanks is due in part to the belief that to require a contractor to conform to a set of specifications based on these guidelines would increase the cost of construction far beyond any benefits. Thus, construction procedures have typically been "unsophisticated" in comparison to those for other types of engineered structures.

The risk of failure can be considered when evaluating the performance of revetment riprap. As summarized in Section 2.3.3, a number of methods is available for assessing the causes and effects of a wide variety of factors in uncertain, complex systems and for making decisions in the light of uncertainty. One approach, failure modes and effects analysis, is a qualitative procedure to systematically identify potential component failure modes and assess the effects of associated failures on the operational status of the system (Johnson and Niezgoda, 2004).

Applying this type of analysis to a riprap revetment installation highlights the impact of various failure modes and emphasizes the integrated nature of the riprap system. Table 2.1 is repeated here as Table 3.17 for convenience and as a summary of revetment riprap failure mechanisms.

3.7.2 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 the stones' ability to withstand high approach velocities and buoyant forces. Parola (1995) noted that secondary currents induced by bridge piers cause high local boundary shear stresses, high local seepage gradients, and sediment diversion from the streambed surrounding the pier and that the addition of riprap also changes the boundary stresses.

Because of the sensitivity of riprap size to velocity, Parola (1995) recommended 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.

Table 3.16. Causes of riprap failure and solutions.

Cause	Solution
Inadequate size of riprap	Larger riprap
Impingement of current directly upon riprap rather than having flow parallel to riprap	Heavier stones, flatten riprap slopes, redirect flow
Channel degradation	Provide a volume of reserve riprap at the revetment toe
Internal slope failure (slump)	Reduce the riprap slope angle
Riprap with high percentage of fines causes washing out of the fines	Follow gradation specifications

Source: Brice and Blodgett (1978)

Table 3.17. Failure modes for riprap revetment.

Failure Modes	Effects on Other Components	Effects on Whole System	Detection Methods	Compensating Provisions
Translational slide or slump (slope failure)	Disruption of armor layer	Catastrophic failure	Mound of rock at bank toe; unprotected upper bank	Reduce bank slope; use more angular or smaller rock; use granular filter rather than geotextile fabric
Particle erosion (rock undersized)	Loss of armor layer, erosion of filter	Progressive failure	Rock moved downstream from original location, exposure of filter	Increase rock size; modify rock gradation
Piping or erosion beneath armor (improper filter)	Displacement of armor layer	Progressive failure	Scalloping of upper bank; bank cutting; voids beneath and between rocks	Use appropriate granular or geotextile filter
Loss of toe or key (under designed)	Displacement or disruption of armor layer	Catastrophic failure	Slumping of rock, unprotected upper bank	Increase size, thickness, depth, or extent of toe or key

- 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 that 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) found 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 remove the support of the edge stones and allow the edge stones to be entrained in the flow (Lim and Chiew, 1996). When the trough of a bed feature migrated past the riprap layer, stones would 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.

Chiew (1995) showed 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. Total disintegration occurs when the riprap stone size to sediment size ratio is small. 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 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

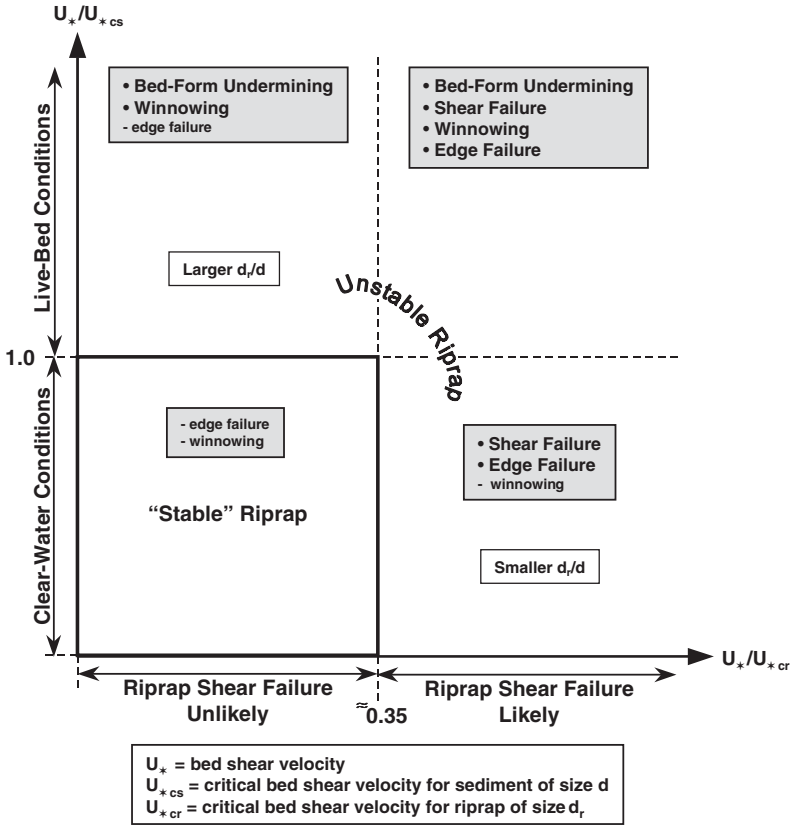
Falls Laboratory) under NCHRP Project 24-7 (Phase 1) (Parker et al., 1998), which verified the four modes of riprap failure (i.e., riprap shear, winnowing, edge failure, and embedment or settlement due to bed-form passage) defined by Lim and Chiew (1996, 1997). The experiments indicate that these processes, 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 Figure 3.61. 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).

3.7.3 Modes of Abutment Riprap Failure

Riprap failure mechanisms of concern at abutments and the approach roadway embankment are, in many respects, similar to the failure modes discussed for revetments in Section 3.7.1. However, because of the unique hydraulic characteristics and flow patterns experienced at abutments, there are several additional areas of concern.

As summarized in Section 2.4.3, FHWA conducted two research studies in a hydraulic flume to determine equations for sizing rock riprap for protecting abutments from scour (Pagán-Ortiz, 1991; Atayee, 1993). The first study investigated vertical wall and spill-through abutments that encroached 28% and 56% on the floodplain, respectively. The second study investigated spill-through abutments that encroached on a floodplain with an adjacent main channel. Encroachment varied from the largest encroachment used in the first study to a full encroachment to the edge of main channel bank. For spill-through abutments in both studies, the rock



Source: modified from Lauchlan (1999)

Figure 3.61. Summary of pier riprap failure conditions for clear-water and live-bed regimes.

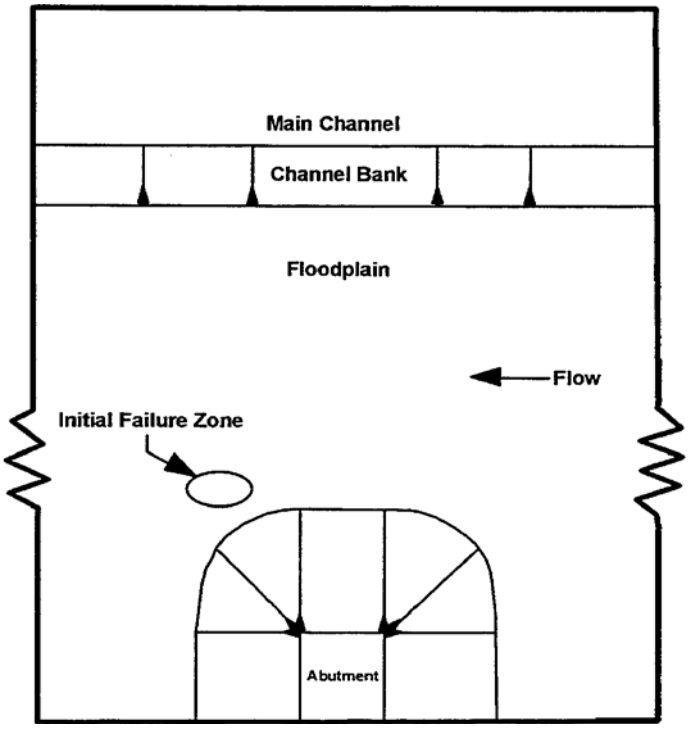


Figure 3.62. Plan view of the location of initial failure zone of rock riprap for spill-through abutment.

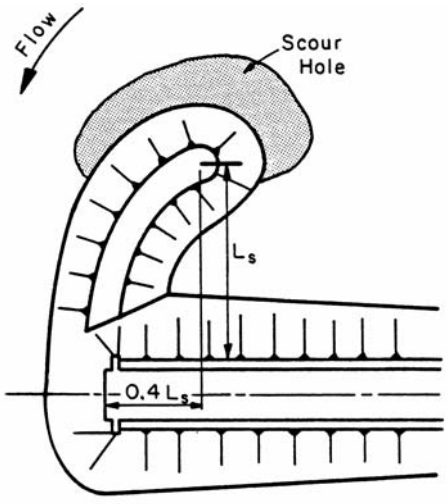
riprap consistently failed at the toe downstream of the abutment centerline (Figure 3.62). For vertical wall abutments, the first study consistently indicated failure of the rock riprap at the toe upstream of the centerline of the abutment.

Field observations and laboratory studies indicate that, with large overbank flow or large drawdown through a bridge opening, scour holes develop on the side slopes of spill-through abutments and the scour can be at the upstream corner of the abutment. In addition, flow separation can occur at the downstream side of a bridge (either with vertical wall or spill-through abutments). This flow separation causes vertical vortices that erode the approach embankment and the downstream corner of the abutment.

3.7.4 Modes of Countermeasure Riprap Failure

Guide banks, spurs, and other river-training countermeasures constructed of soil embankment material must be protected by riprap or other erosion-resistant material. While failure mechanisms can be similar to the failure modes discussed for revetment in Section 3.7.1, there are additional areas of concern for these and similar countermeasures.

Guide banks are placed at or near the ends of approach embankments to guide the stream through the bridge opening. Constructed properly, flow disturbances, such as eddies and cross-flow, will be minimized to make a more efficient waterway under the bridge. They are also used to protect the highway embankment and reduce or eliminate local scour at the embankment and adjacent piers. The effectiveness of guide banks is a function of river geometry, quantity of flow on the floodplain, and size of bridge opening. A typical guide bank at the end of a roadway approach embankment is shown in Figure 3.63. As overbank flows are directed from the floodplain around the end of the guide bank and through the bridge opening, scour at the



Source: Richardson et al. (2001)

Figure 3.63. Typical guide bank.

nose of the guide bank is of particular concern. Additional riprap should be placed around the upstream end of the guide bank to protect the embankment material from scour as this is the most likely failure zone for a guide bank.

Spurs are a linear river-training countermeasure projecting into the flow from an eroding bankline. The most common causes of spur failure are undermining at the toe and outflanking by the stream. These problems occur primarily in alluvial streams that experience wide fluctuations in the channel bed. Impermeable rock riprap spurs can be designed to counter erosion at the toe by providing excess material on the streambed. As scour occurs, excess material is launched into the scour hole, thus protecting the end of the spur. For a spur constructed of embankment material and protected by riprap, revetment riprap design procedures are generally used, and, as with a guide bank, the most likely failure zone is at the end of the spur.

Countermeasures such as guide banks or spurs are generally designed with some freeboard above the selected design flow. However, when embankments, guide banks, and spurs are subjected to flows in excess of the design flow, overtopping can occur. For embankments or countermeasures protected by revetment riprap, the mechanics of overflow erosion processes point to the most likely failure zones.

3.7.5 Embankment Overtopping Failure

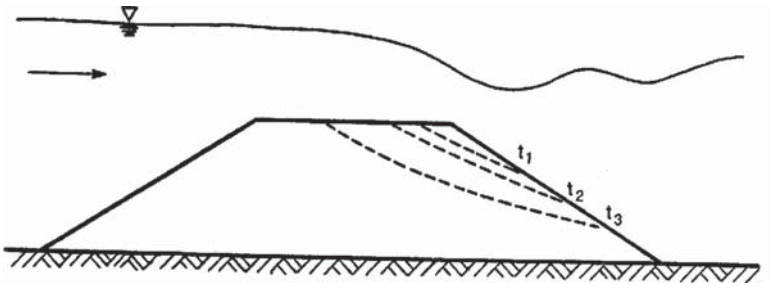
When flow overtops an embankment, spur, or guide bank, locally high velocities and shear stresses will create strong erosion forces, typically at the downstream shoulder and on the embankment slope, that are too great for the soil of the embankment to withstand. Two primary processes of erosion occur during an overtopping event.

When the overtopping flow is submerged, erosion typically begins at the downstream shoulder. This condition is often experienced by roadways and bridge approach embankments. Figure 3.64 (Chen and Anderson, 1987) shows the progression of this type of failure at times t_1, t_2, and t_3. As flow accelerates over the embankment, a surging hydraulic jump is formed that causes a nick point between the shoulder and downstream slope. This nick point will begin to migrate upstream because of the high velocities, and erosion will begin to move downstream. The downstream migration of the erosion is caused by the turbulence associated with the hydraulic jump. This condition would also apply to most river training countermeasures, such as spurs and guide banks, under overtopping conditions.

The second general erosion pattern results from the case of free flow. With low tailwater, the flow will accelerate down the slope with high velocity and shear stress associated with supercritical flow. Erosion typically initiates near the toe of the embankment, whether or not a hydraulic jump is present. Erosion progresses in the upslope and upstream direction through the embankment. Figure 3.65 (Chen and Anderson, 1987) illustrates this progression. This condition would typically apply to earth dams, spillways, or levees protected by revetment riprap.

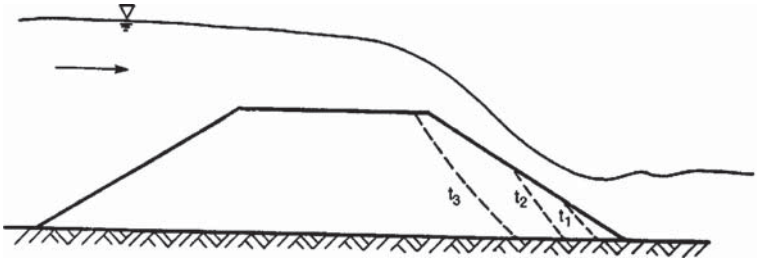
Near-prototype flume tests were conducted at CSU (Oswalt et al., 1994) with riprap placed on embankment slopes of 1%, 2%, 8%, 10%, and 20% and subjected to overtopping flows until failure. Failure was defined by exposure of the underlying sand and gravel bedding. Based on the results of five tests, rounded-shape riprap was found to fail at a unit discharge about 40% less than that of angular stones of the same median size, demonstrating the importance of stone shape on riprap layer stability. Angular stones tend to wedge or interlock and require fewer fines to fill voids, compared to similarly graded rounded stones. Rounded stones are much more likely to slide or roll, especially on the steeper slopes. Riprap specifications normally require angular-shaped stone.

Channelization was observed to occur between the threshold and collapsing stages of overtopping flow. Channels form in the riprap layer as the smaller stones are washed out, producing flow concentrations and increasing the localized unit discharge. The CSU studies suggest flow concentrations of three times the normal unit discharge are possible. The average point of incipient channel formation was identified at about 88% of the unit discharge at failure.



Source: Chen and Anderson (1987)

Figure 3.64. Typical embankment erosion pattern with submerged flow.



Source: Chen and Anderson (1987)

Figure 3.65. Typical embankment erosion pattern with free flow.

Wittler and Abt (1990) investigated the influence of material gradation on the stability of the riprap layer with overtopping flow. In general, uniformly graded riprap displays a greater stability for overtopping flows but fails suddenly, while well-graded riprap resists sudden failure as voids are filled with smaller material from upstream; this process is referred to as "healing."

Additional studies at CSU from 1994 to 1997 (see Section 3.5.1) provided more details on the failure mechanism (Mishra, 1998). Again, failure of the riprap slope was defined as removal or dislodgement of enough material to expose the bedding material. Failure of the riprap layer occurred with the measured water depth still within the thickness of the rock layer. A layer of highly aerated water was flowing over the surface of the riprap, but this surface flow was only a small portion of the total flow (Figures 3.66 and 3.67, see also Figure 3.54).

Before the riprap slope failed, many individual stones moved or readjusted locations throughout the test period. This movement, referred to as incipient motion, occurs when the displacing and overturning moments exceed the resisting moments. The force in the resisting moment is given by the component of the weight perpendicular to the embankment and interlocking between stones in the matrix. The overturning forces are the drag (or the jet impact on a stone), the lift, buoyancy, and, to a lesser degree, the component of the weight parallel to the embankment depending on the point(s) of contact with other stones. Even though buoyancy plays an important role in the removal of rocks, the hydrodynamic forces have the major role in producing failure of the protective layer. It was also concluded that on steep embankments, riprap failure on the slope is more critical than the failure at the toe.

Figure 3.66. Riprap failure in 1994 tests (d_{50} = 15.2 in [386 mm]).

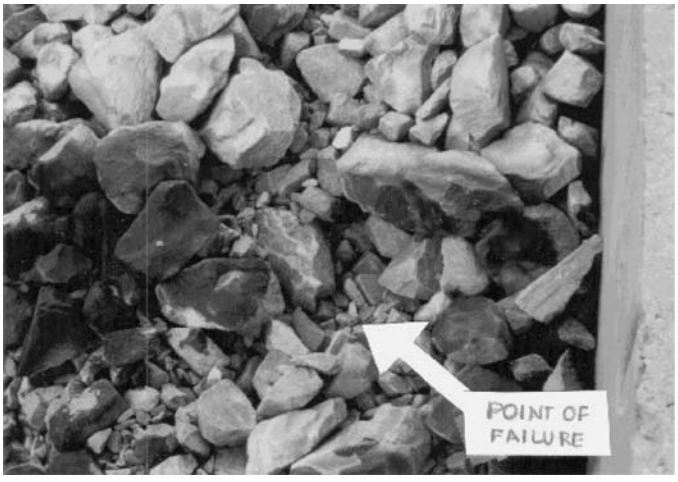


Figure 3.67. Riprap failure in 1997 tests (d_{50} = 10.7 in [271 mm]).

3.7.6 FHWA Case Studies

Brice and Blodgett (1978a, 1978b) developed guidelines to assist design, maintenance, and construction engineers in selecting measures that can be used to reduce bridge losses attributable to scour and bank erosion. The guidelines are based on case histories of 224 bridge sites in the United States and Canada, on interviews with bridge engineers in 34 states, and on a survey of published work on countermeasures. Each case history (Brice and Blodgett, 1978b) includes a photograph and/or a sketch of the site; data on bridge, geomorphic, and flow factors; a chronological account of relevant events at the site; and an evaluation of hydraulic problems and countermeasures. Brice and Blodgett documented the use of rock riprap at 110 sites (1978a, Table 2). They rated the performance at 58 sites and found satisfactory performance at 34 sites, partially satisfactory performance at 12 sites, and failure to perform satisfactorily at 12 sites.

A review of the causes of failure at these sites is instructive (1978a, Table 3). The absence of a filter blanket was clearly the cause of the failure at a site subject to tides and wave action. The riprap was placed on a fill of sand and fine gravel that eroded through the interstices of the riprap.

Internal slope failure was the cause of failure of riprap at the abutment of bridges at two sites. At one site, failure was attributed to saturation of a high fill by impounded water in a reservoir. Wave action also probably contributed to the failure. The other site is difficult to include as a riprap failure because the rock was not placed as riprap revetment. Thirty-three freight car loads of rock were dumped as an emergency measure to stop erosion at a bridge abutment during high-flow releases from a reservoir. The rock was displaced, and the high streambanks and highway fill were still susceptible to slumps. At both sites, riprap failed to prevent slumps in high fills.

Inadequate rock size and size gradation was given as the cause of failure at eight sites. All of these sites are complex, and failure is difficult to assign to one cause; but, rock size was definitely a factor.

Channel degradation accounted for failure at three sites in Mississippi. Channel degradation at these sites was due to channel straightening and clearing by the Soil Conservation Service (now NRCS) and USACE. Riprap installations on the streambanks, at bridge abutments, and in the streambed failed to stop lateral erosion. At one site, riprap placed on the banks and bed of the stream resulted in severe bed scour and bank erosion downstream of the riprap.

Failure of riprap at one site was attributed to the steep slope on which the riprap was placed. At this site, rock riprap failed to stop slumping of the steep banks downstream of a check dam in a degrading stream.

Successful rock riprap installations at bends were found at five sites. Bank erosion was controlled at these sites by rock riprap alone. Installations rated as failing were damaged at the toe and upstream end, indicating inadequate design and/or construction, and damage to an installation of rounded boulders, indicating inadequate attention to riprap specifications. Other successful rock riprap study sites were sites where bank revetment was used in conjunction with other countermeasures, such as spurs or retards. The success of these installations was attributed more to the spurs or retards, but the contribution of the bank revetment was not discounted.

3.7.7 U.S. Geological Survey (USGS) Case Studies

In analyzing causes of riprap failure, Blodgett and McConaughy (1986) provide a case history with photographs for several modes of revetment riprap failure identified in Section 3.7.1, including

- Particle erosion – Sacramento River near Chico, California;
- Translational slide – Cosumnes River near Sloughhouse, California; and
- Slump failure – Cosumnes River near Sloughhouse, California.

They also provide analysis of the hydraulics associated with riprap failure, including

- Particle erosion – Sacramento River near Chico, California, and Pinole Creek at Pinole, California;
- Particle erosion – Truckee River near Sparks, Nevada;
- Modified slump failure – Cosumnes River near Sloughhouse, California; and
- Modified slump failure – Hoh River near Forks, Washington.

The case studies for four of these sites provide excellent illustrations of riprap failure modes and are summarized here. In general, the design methods, filter, termination details, and quality control during construction are unknown for these case studies.

Sacramento River Near Chico, California

The floodplain at this site is low and subject to frequent and prolonged inundation. As a result, the entire riprap layer is subject to shear stress. Displacement of individual stones at the site has been documented, and the submerged weight of the largest rock moved was 14.6 lbs (6.63 kg); the intermediate axis was 0.60 ft (0.18 m).

Three localized areas of riprap failure caused by particle erosion (see Figure 3.57) were surveyed during the 1983 water year. A unique hydraulic condition at this site is the contraction of flow caused by a vertical rise in the channel bed rather than a reduction in channel width. The vertical constriction is caused by a delta built up in the riverbed by a tributary entering the Sacramento River downstream from the site. The channel bed slope at the site is –0.48%, in comparison with the water-surface slope of 0.056%. This site illustrates the problems in estimating the effective shear stress when slopes are estimated from topographic maps or from the water surface.

Failure of the riprap at this site was initiated by displacement of individual stones (particle erosion [Figure 3.68]). After repeated periods of high water, the riprap lining was eroded to the original base material; however, there was no evidence of base material failure at the site. The gradation of the riprap (ratio of d_{85}/d_{15} = 1.29) is close to the recommended ratio of 1.4 given in HEC-11 and HEC-15 and is within the range specified in EM 1601. Failure of the riprap is attributed to the rock size being too small, and side slope of the bank being too steep.

Pinole Creek at Pinole, California

Damage to the Pinole Creek riprap, which was designed by USACE (construction plans dated April 1965) using procedures given in EM 1601, resulted from particle erosion of the riprap from the lower part of the channel banks (Figure 3.69). A small zone of riprap near the top of the bank remained intact, indicating shear stresses were insufficient to remove the upper material. That the upper zone material remained in place even though vertical support had been removed indicates the side slope of the banks for this riprap was less than the angle of repose. Much of the eroded riprap was found on the channel bed and acted as a flow diverter that directed some of the flow towards the newly unprotected bank. Failure of the riprap at this site is attributed to a particle size d_{50} that is too small for the hydraulic stresses created by this size of flood.

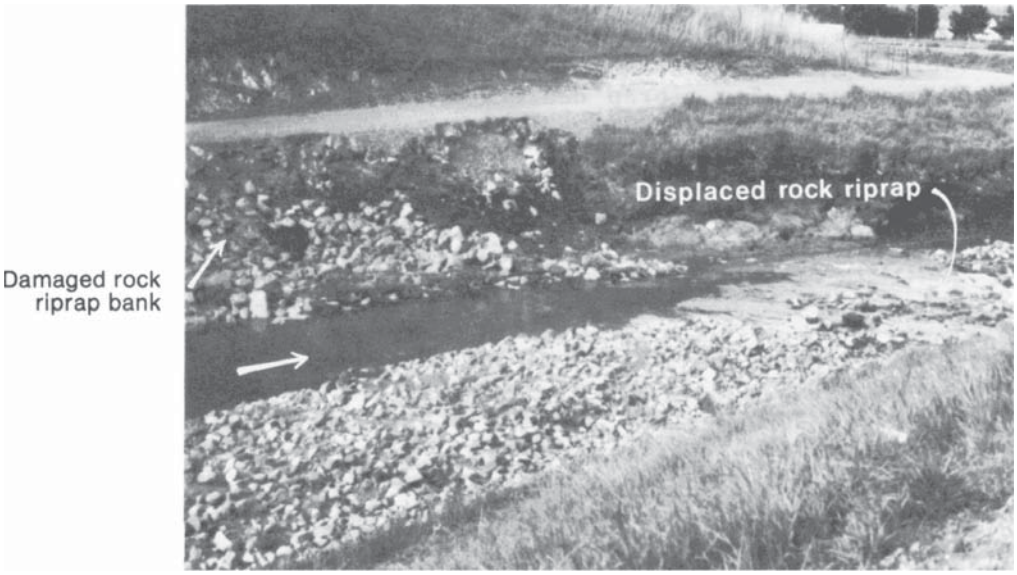
Cosumnes River Near Sloughhouse, California

The riprap at this site (Figure 3.70) was constructed to prevent lateral migration of the channel. The design procedure is not known. A modified slump failure (Figure 3.59) about 15 ft (4.6 m) wide was noted at Site 3 about 1 month after flooding and 6 months after construction of the riprap. The riprap is subject to impinging flows. Individual pieces of riprap in the slump area were displaced downslope, with the toe of the slump ending up 13 ft (4.0 m) below the top of the bank. The failure is attributed to failure of the interface between the base material and riprap and possible excess hydrostatic pressure in the base material. The location of the riprap failure, which is about 21 ft (6.4 m) above the channel bed, indicates that stresses near the top of the bank may be more critical than stresses defined for the channel bed.

A reference line shows location of stones in December 1981. Flow is from left to right. Note displaced stone near steel fencepost (photographed March 4, 1982)

Source: Blodgett and McConaughy (1986)

Figure 3.68. Riprap on left bank of Sacramento River near Chico, California.



Note deposition of displaced riprap from upstream locations in channel bed (photographed March 1982)

Source: Blodgett and McConaughy (1986)

Figure 3.69. Damaged riprap on left bank of Pinole Creek at Pinole, California, following flood of January 4, 1982.

Source: Blodgett and McConaughy (1986)

Figure 3.70. Riprap on Cosumnes River at Site 3 near Sloughhouse, California, looking downstream, showing modified slump failure (photographed May 31, 1983).

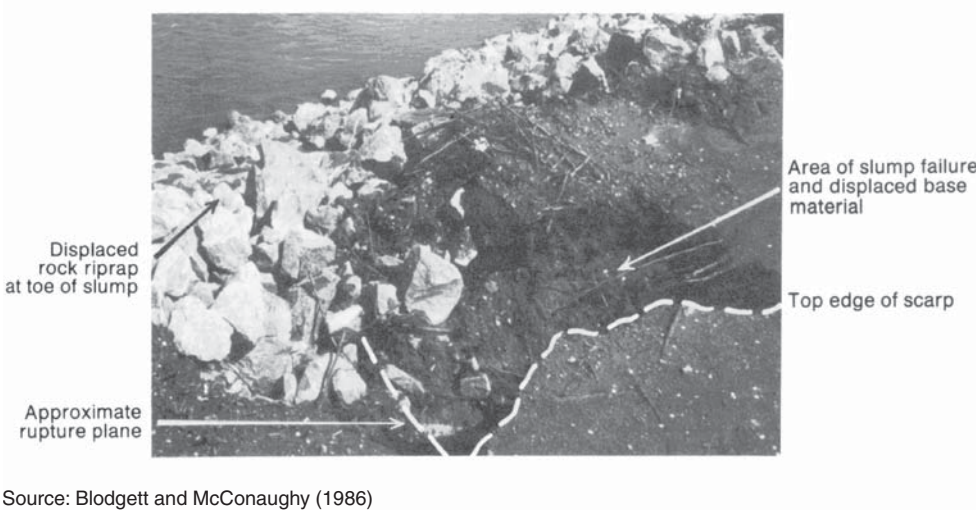
A translational slide failure (Figure 3.58) was also observed at Site 2 (Figure 3.71) and a slump failure occurred at Site 1 (Figure 3.72) at this same location.

Hoh River Near Forks, Washington

The procedure used for riprap design at this site is not known. Particle erosion occurred during the first several floods after the riprap was installed during summer 1982. Riprap damage occurred near the truck tires shown in Figure 3.73, during October 1982. The damage is attributed to (1) channel bed scour that undermined the toe of the riprap and caused modified slump, (2) poor size gradation of the riprap that allowed erosion of the supporting smaller material in the riprap, and (3) a steep side slope that reduced the amount of force required to displace individual stones.

Source: Blodgett and McConaughy (1986)

Figure 3.71. Riprap on Cosumnes River at Site 2 near Sloughhouse, California, looking down stream, showing translational slide failure (photographed May 31, 1983).



Source: Blodgett and McConaughy (1986)

Figure 3.72. Riprap on left bank of Cosumnes River at Site 1 near Sloughhouse, California, showing slump failure (photographed May 31, 1983).

Most of the larger stones still in position at the site were at a precarious state of balance.

Riprap damage on the left bank at an upstream location near the bulldozer shown in Figure 3.73 during the flood of December 3, 1982, is attributed to particle erosion (Figure 3.57). The damaged riprap shown in Figure 3.74 was overtopped about 3 ft (0.9 m) during the flood. Most of the damage occurred near the top of the bank next to the low elevation access road. Riprap erosion may have been caused by irregular patterns of overbank flow in the vicinity of the low bank access road. A schematic of the initiation of a typical partial erosion failure is shown in Figure 3.75.

USGS Summary

Blodgett and McConaughy (1986) conclude that certain hydraulic factors are associated with each of the four types of riprap failure (particle erosion, translational slide, modified slump, and slump [see Figures 3.57 through 3.60]). While the specific mechanism causing failure of the riprap is difficult to

Riprap was damaged by modified slump at a location near truck tires during flooding in autumn of 1982 (photographed before failure August 1982).

Source: Blodgett and McConaughy (1986)

Figure 3.73. New riprap placed on left bank (upstream view) of Hoh River at Site 1 near Forks, Washington.

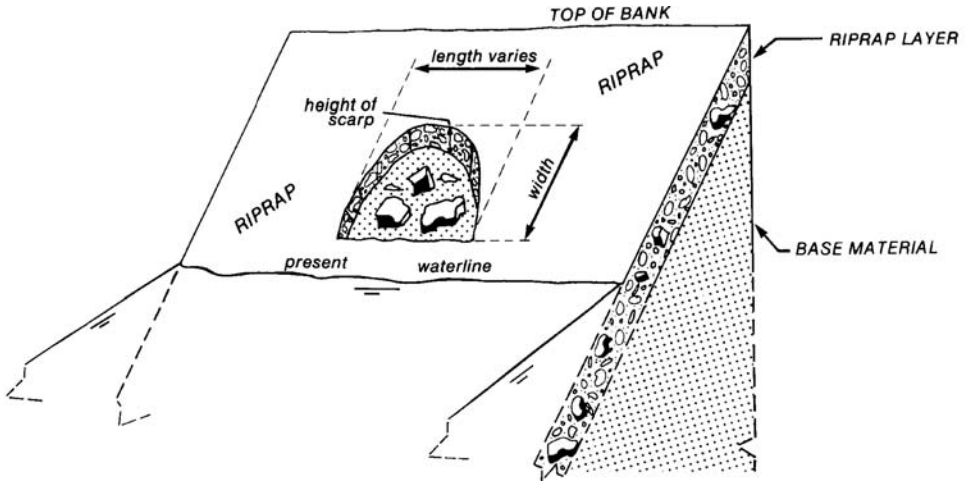
Damage is attributed to particle erosion by impinging flows that overtopped bank during flood of December 3, 1982 (photographed December 1982).

Source: Blodgett and McConaughy (1986)

Figure 3.74. Damaged riprap on left bank (downstream view) of Hoh River at Site 1 near Forks, Washington.

determine, and a number of factors, acting either individually or combined, may be involved, they identify the following reasons for riprap failures:

- Particle size was too small because
  - Shear stress was underestimated
  - 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
- Channel changes caused
  - Impinging flow
  - Flow to be directed at ends of protected reach
  - Decreased channel capacity or increased depth
  - Scour of toe of riprap
- Riprap material had improper gradation
- Material was placed improperly
- Side slopes were too steep



Source: Blodgett and McConaughy (1986)

Figure 3.75. Typical riprap failure area in the shape of a horseshoe, caused by particle erosion.

- No filter blanket was installed or blanket was inadequate or damaged
- Excess hydrostatic pressure caused failure of base material
- Differential settlement occurring during submergence or periods of excessive precipitation

Blodgett and McConaughy (1986) conclude that estimates of particle stability serve as the basis for most riprap design procedures. This approach seems sound because particle erosion is involved in most of the causes of failure described previously.

3.7.8 Caltrans Case Studies

In preparing CABS, Racin et al. (2000) developed a list of conditions that cause failure of riprap revetments, conducted field reviews of rock slope protection sites with local engineers in five states, and recorded site data and rated sites (as successful, or failed and repaired).

Specific causes of revetment riprap failure included the following:

- Channel is constricted (via debris, narrow gorge or bridge upstream, recent channel "repairs" by just adding extra thickness of rock to a previously failed section), causing local velocities to be greater than design velocity:
  - Higher magnitude velocity with impinging vectors (and turbulence) displaces and removes rocks or soil. With sustained impingement, rocks and/or soil are removed either gradually (several storms) or suddenly (same event).
  - Higher magnitude velocity with parallel vectors (and laminar or transitional flows) causes "suction force" that removes "lighter" or smaller particles that
    - Are loosely stacked on outer surface
    - Are not held firmly by outer matrix
    - "Worm out" from below surface layer of rock through voids in outer matrix, because of lack of a filter/separator.
- Toe is undermined (mining, steep gradient, incised bed, headcutting, transverse or skewed inflows).
- Rocks are too small to withstand design or smaller flows.
- Rock revetment is not thick enough.
- Rounded rocks roll out of matrix.
- Slope is too steep.
- Rock quality is poor.

A tabular summary of each of 65 sites includes the following information:

- Site location
- Method of riprap design
- Site, riprap, or failure description
- Construction date, status, and date evaluated

Of the 65 sites evaluated, 21 had experienced a riprap failure or the riprap had needed repair. Appendix C of CABS (Racin et al., 2000) provides photographs and a description of the conditions at each site and a discussion of the failure, where applicable. Site 14 on Salmon Creek, Oregon, and Site 60 on Grizzly Creek, California, provide well-documented photographs of riprap failure.

Salmon Creek, Oregon

Site 14 (Figure 3.76) is on Salmon Creek on the Willamette Highway (Route 58) in Lane County, Oregon. The riprap was designed by the USACE, Portland District, using EM 1110 (pre-1957). The failure was attributed to impinging flow apparently caused by debris and shifting gravel bars that undermined the toe of the riprap revetment on a river levee. Parallel flow conditions were assumed for the design. The causes of failure include rounded rock, steep slope (wV:2H), and no filter layer. In addition, channel capacity in this reach had increased and 6 ft (1.8 m) of degradation had occurred since the riprap was installed in 1959.

Grizzly Creek, California

Site 60 (Figure 3.77) is on Grizzly Creek on Route 30 in Lake County, California. The riprap was designed using BSP (State of California DPW, 1960). It was concluded that the riprap on the streambanks and bottom was undersized (1/2-ton RSP Class using Method B). Contributing to the failure were the steep channel grade, upper slope runoff, and impinging transverse down drains, which eroded behind the

View upstream of west bank just upstream of Route 58 bridge (photographed May 1992)

Source: Racin et al. (2000)

Figure 3.76. Site 14 Salmon Creek, Oregon, failed riprap on levee bank.

View is looking upstream. Woven-tape geotextile (slit-film) marked with arrows is inappropriate as RSP-fabric on banks (photographed May 1995)

Source: Racin et al. (2000)

Figure 3.77. Site 60 Grizzly Creek, California, failed bank and channel invert protection.

filter fabric and above the riprap. The 1/2-ton rock was gap graded with poor interlocking and no backing material. The riprap on the channel side slopes (1V:2H) had the wrong RSP fabric (low permittivity woven tape, slit film geotextile).

3.7.9 Case Study – Bridge Pier Riprap Failure

The case studies cited, previously, document cases of revetment riprap failure. 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 NTSB. The peak flow was 64,900 cfs (1,838 m^3/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 stratified glacial drift, which was considered nonerodible by the designers (Figure 3.78). 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-to-50–scale, 3-D model study established a flow velocity of 10.8 ft (3.3 m/s) at the pier that failed. Also, the 1-to-50–scale and a 1-to-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 3.79).

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."

The NYSTA inspected the bridge annually or biennially with the last inspection on April 1, 1986. A 1979 inspection by a consultant hired by NYSDOT indicated that most of the riprap around the piers was missing (Figures 3.80 and 3.81); however, the 1986 inspection failed to detect any problems with the condition of the riprap at the piers.

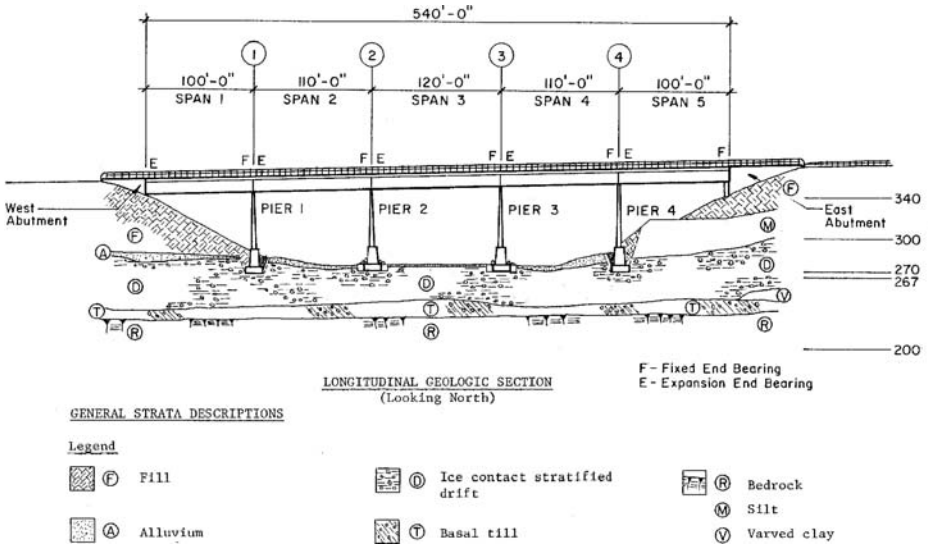


Figure 3.78. South elevation of Schoharie Creek Bridge showing key structural features and a schematic geological section.

Pier 2 in the foreground with Pier 3 in the background.

Figure 3.79. Pier scour holes at Schoharie Creek bridge in 1987.

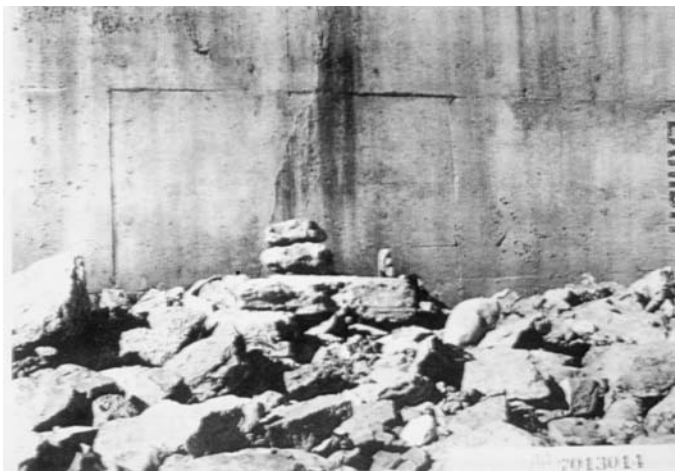
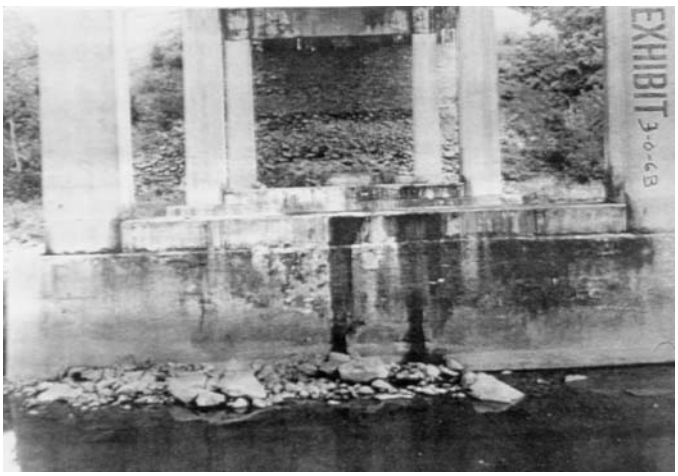


Figure 3.80. Photograph of riprap at Pier 2, October 1956.



Flow is from right to left.

Figure 3.81. Photograph of riprap at Pier 2, August 1977.

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 (see Section 3.2.5).

3.7.10 Ice and Debris

Ice and debris can create additional stresses on riprap by impact and flow concentration. In addition, ice attachment to riprap particles can cause displacement. A study by USACE's Cold Regions Research Engineering Laboratory (CRREL) (Sodhi et al., 1996) suggests that the predominant mode of ice damage to riprap on slopes takes place during pileup events. As the incoming ice sheet is forced against the slope, it is driven between the riprap and the previously piled up ice. In doing so, the ice sheet forces rocks from the bottom to the surface of the ice pile. To counteract this effect, the CRREL recommends that the d_{100} of the riprap be at least twice the ice thickness for mild slopes (shallower than 3H:1V) and about three times the ice thickness for steeper slopes.

Riprap deterioration on slopes caused by vegetative debris loading primarily involves particle dislodgement by direct impact. EM 1601 (USACE, 1991) recommends that riprapped slopes on streams with heavy debris loads should be no steeper than 2.5H:1V. However, the potential for riprap displacement due to flow redirection or concentration caused by debris accumulations must not be overlooked. In particular, debris accumulations at bridges constrict the waterway and may redirect high velocity flow toward riprap at piers or abutments during floods.

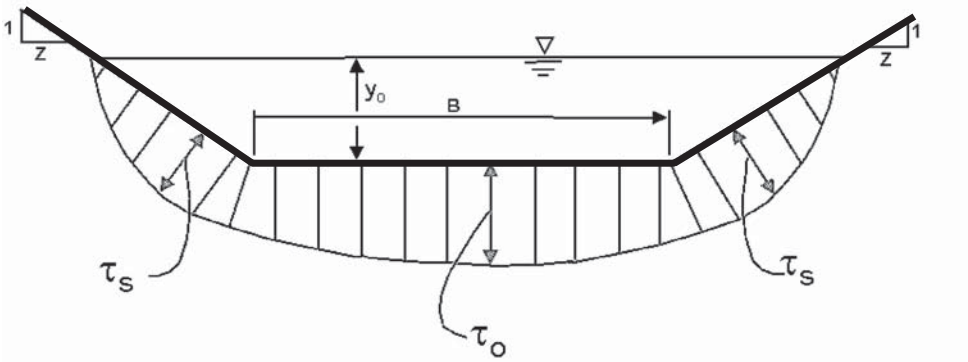
3.8 Bioengineering/Hybrid Design

3.8.1 Introduction

In the context of this study, hybrid designs are bank stabilization treatments that are conceptualized as a standard riprap section at the toe and lower bank areas, transitioning to a less heavy-duty treatment on the mid- and upper bank slopes. The lighter treatment provides protection for areas that experience less severe hydraulic forces and are inundated less frequently compared to the lower bank. Use of lighter materials is intended to result in a more economically efficient installation and can also provide secondary benefits associated with habitat enhancement and aesthetic value.

The following materials are often considered for mid- and upper bank stability:

- Rock riprap with smaller d_{50} and decreased layer thickness
- Grass vegetation reinforced with synthetic erosion control blanket ("turf reinforcement")



Source: modified from Chen and Cotton (1988)

Figure 3.82. Shear stress distribution in a trapezoidal channel.

- Willow fascines, brush layering, or live staking (alone or in combination with riprap)

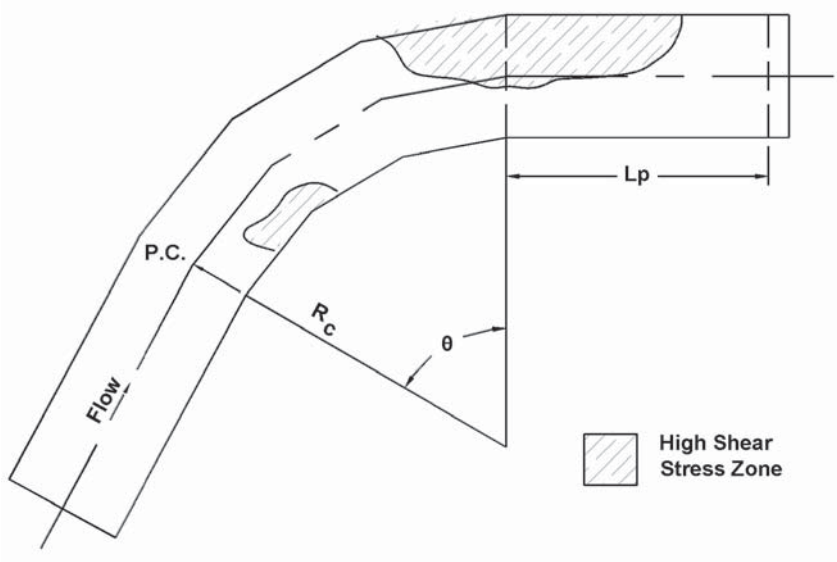
3.8.2 Hydraulic Considerations

Resistance of vegetation to shear stress depends on plant density, plant stem height, uniformity of plant cover, plant rooting habits, and soil erodibility. Shear stress is a preferred measure of vegetation resistance because it considers several variables including depth, wetted perimeter, and flow velocities (Hoitsma and Payson, 1998). Additionally, failure criteria for a particular lining can be approximated by a single shear stress value, applicable over a given range of channel slopes and shapes (Gray and Sotir, 1996).

In a channel, shear stress varies along the wetted perimeter. Typically, the zone of highest shear occurs at the centerline of the bed. On the side slopes, the highest shear occurs on the lower third of the bank. Figure 3.82 (Chen and Cotton, 1988) presents a schematic diagram showing a typical shear stress distribution along the wetted perimeter of a trapezoidal channel in a straight reach.

At channel bendways, secondary currents exist that impose higher shear stresses on the channel sides on the outside of the bend due to impinging flow, as illustrated in Figure 3.83 (Chen and Cotton, 1988). Figure 3.83 also shows a smaller area of shear stress concentration caused by flow separation at the beginning of the bend on the opposite side of the channel.

Rigid vegetation, like large trees or woody debris, should be analyzed differently from flexible types like grasses, and emergent vegetation that protrudes through the water surface has a different effect on flow than fully submerged vegetation. Flow resistance values for regions covered with rigid vegetation or woody debris depend upon the size and spacing of the rigid objects (i.e., trees and whether the trees are submerged or protrude through the free surface).



Source: Chen and Cotton (1988)

Figure 3.83. Shear stress distribution in a bend.

Table 3.18. Critical boundary shear stress for various live materials.

Boundary Material	Critical Boundary Shear Stress (lb/ft^2)	Critical Boundary Shear Stress (N/m^2)	Reference
Long, Native Grasses	1.2-1.7	57.5-81.4	Fischenich (2001)
Hardwood Trees	0.45-2.5	21.5-119.7	Fischenich (2001)
Bermuda grass, non reinforced	7.1	342	WCHL (1979)
Bermuda grass, nylon mesh reinforced	8.7	415	WCHL (1979)
Class A Vegetation*	3.7	177.2	Chen and Cotton (1988)
Class B Vegetation*	2.1	100.6	Chen and Cotton (1988)
Class C Vegetation*	1	47.9	Chen and Cotton (1988)
Class D Vegetation*	0.6	28.7	Chen and Cotton (1988)
Class E Vegetation*	0.35	16.8	Chen and Cotton (1988)

*See Table 3.19

In general, flow resistance caused by flexible vegetation declines with increasing discharge as stems are flattened by the flow. For example, Oplatka (1998) reported that field tests conducted on 3- to 6-year-old willows grown from cuttings showed that the area of the plants perpendicular to flow decreased by a factor of 4 to 5 at a flow velocity of 3.3 ft/s (1 m/s) and by a factor of 20 to 40 at a flow velocity of 13 ft/s (4 m/s). Since resistance due to flexible vegetation is a function of the shear stress applied to the vegetation, iterative solutions are required to determine hydraulic conditions.

Vegetal resistance to shear stress can range from 0.35 to 8.50 lb/ft^2 (16.8 N/m^2 to 407 N/m^2) (Hoitsma and Payson, 1998). Table 3.18 presents a comparison of critical boundary shear stress from various sources. Table 3.19 provides examples of common vegetative species composing the vegetal resistance classes A through E.

Table 3.19. Classification of vegetal covers as to degree of resistance.

Retardance Class	Cover	Condition
A	Weeping lovegrass	Excellent stand, tall (average 760 mm)
	Yellow bluestem ischaemum	Excellent stand, tall (average 910 mm)
B	Kudzu	Very dense growth, uncut
	Bermuda grass	Good stand, tall (average 300 mm)
	Native grass mixture (little bluestem, bluestem, blue gamma, and other long and short Midwest grasses)	Good stand, unmowed
	Weeping lovegrass	Good stand, tall (average 610 mm)
	Lespedeza sericea	Good stand, not woody, tall (average 480 mm)
	Alfalfa	Good stand, uncut (average 280 mm)
	Weeping lovegrass	Good stand, unmowed (average 330 mm)
	Kudzu	Dense growth, uncut
	Blue gamma	Good stand, uncut (average 280 mm)
C	Crabgrass	Fair stand, uncut 250 to 1200 mm
	Bermuda grass	Good stand, mowed (average 150 mm)
	Common lespedeza	Good stand, uncut (average 280 mm)
	Grass-legume mixture (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (150 to 200 mm)
	Centipede grass	Very dense cover (average 150 mm)
	Kentucky bluegrass	Good stand, headed (150 to 300 mm)
D	Bermuda grass	Good stand, cut to 60 mm height
	Common lespedeza	Excellent stand, uncut (average 110 mm)
	Buffalo grass	Good stand, uncut (80 to 150 mm)
	Grass-legume mixture (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (100 to 130 mm)
	Lespedeza sericea	After cutting to 50-mm height. Very good stand before cutting.
E	Bermuda grass	Good stand, cut to height of 40 mm
	Bermuda grass	Burned stubble

Source: Chen and Cotton (1988)

In addition to withstanding higher shear stress, vegetation must also survive frequent inundation. Pezeshki and Shields (2006) provide results from field studies on the survivability of Black Willow cuttings used for stream bank stabilization in the Southeastern United States, as well as a good literature review of this topic. They conclude that, while Black Willow is a flood tolerant species, frequent inundation significantly decreases root growth and survivability. In their study the best conditions for Black Willow were limited to 0.5 to 1.0 m (1.6 to 3.3 feet) above stream base flow water surface elevation and included the portion of the bank with ample soil moisture and adequate drainage. These observations support the concept of combining a riprap toe with a vegetated bank.

3.8.3 Results from NCHRP Project 24-19

The following discussion draws primarily from concepts presented in the final report for NCHRP Project 24-19, *NCHRP Report 544: Environmentally Sensitive Channel- and Bank-Protection Measures* (McCullah and Gray, 2005).

General Concepts

Continuous and resistive bank protection measures, such as riprap and longitudinal rock toes, are primarily used to armor outer bends or areas with impinging flows. These continuous and concentrated high velocity areas will generally result in reduced aquatic habitat. It has been widely documented that resistive techniques, in general, and riprap, in particular, provide minimal aquatic habitat benefits (Shields et al., 1995). Recently the concerns over the poor aquatic-habitat value of riprap, both locally and cumulatively, have made the use of riprap alone controversial (Washington Department of Fish and Wildlife, 2003).

Because streambank protection designs that consist of riprap, concrete, or other inert structures alone are often unacceptable for lack of environmental and aesthetic benefits, there is greater interest in designs that combine vegetation with inert materials into living systems that can reduce erosion while providing environmental and aesthetic benefits (Sotir and Nunnally, 1995).

The negative environmental consequences of riprap can be reduced by minimizing the height of the rock revetment up the bank and/or including biotechnical methods, such as vegetated riprap with brush layering and pole planting; vegetated riprap with soil, grass, and ground cover; vegetated riprap with willow (*Salix spp.*) bundles; and vegetated riprap with bent poles.

Combining riprap with deep vegetative planting (e.g., brush layering and pole planting) is also appropriate for banks with geotechnical problems, because additional tensile strength is often contributed by roots, stems, and branches. In contrast, trees and riparian vegetation planted only on top of the bank can sometimes have a negative impact (Simon and Collison, 2002).

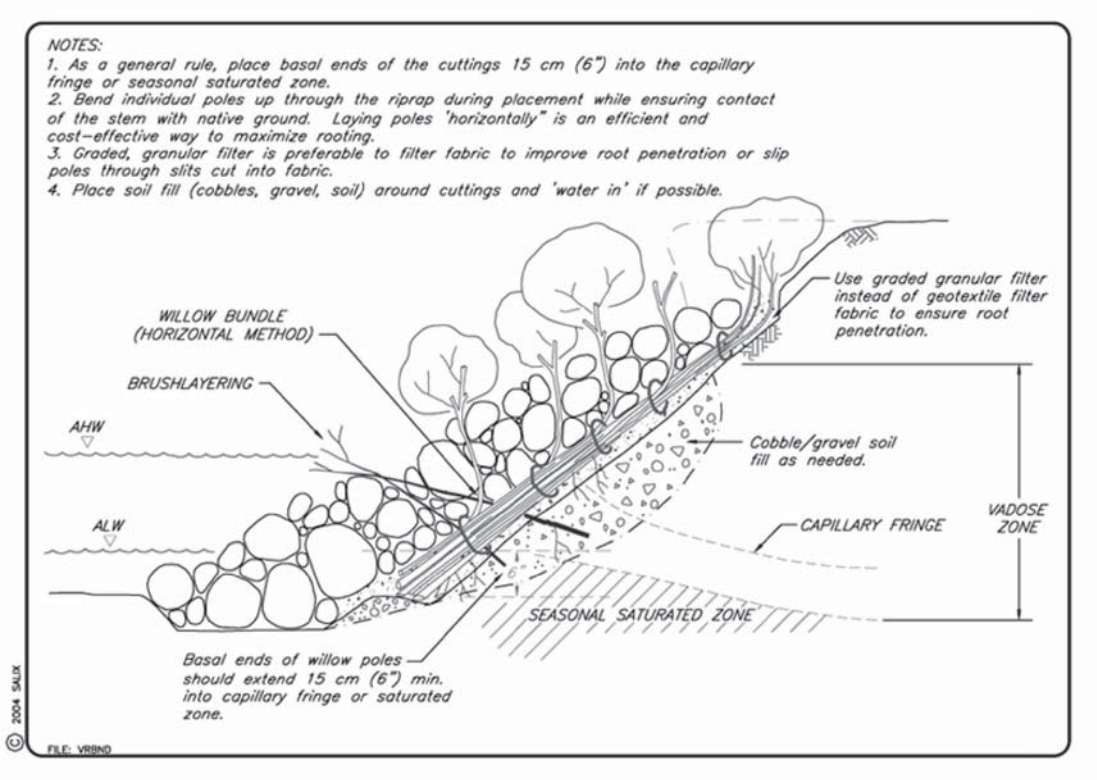
Correctly designed and installed, vegetated riprap offers an opportunity for the designer to attain the immediate and long-term protection afforded by riprap with the habitat benefits inherent with the establishment of a healthy riparian buffer. The riprap will resist the hydraulic forces, while roots and branches increase geotechnical stability, prevent soil loss (or piping) from behind the structures, and increase pullout resistance (McCullah, 2004).

Above ground components of the plants will create habitat for both aquatic and terrestrial wildlife, provide shade (reducing thermal pollution), and improve aesthetic and recreational opportunities. The roots, stems, and shoots will help anchor the rocks and resist 'plucking' and gouging by ice and debris.

Commonly Used Vegetative Methods

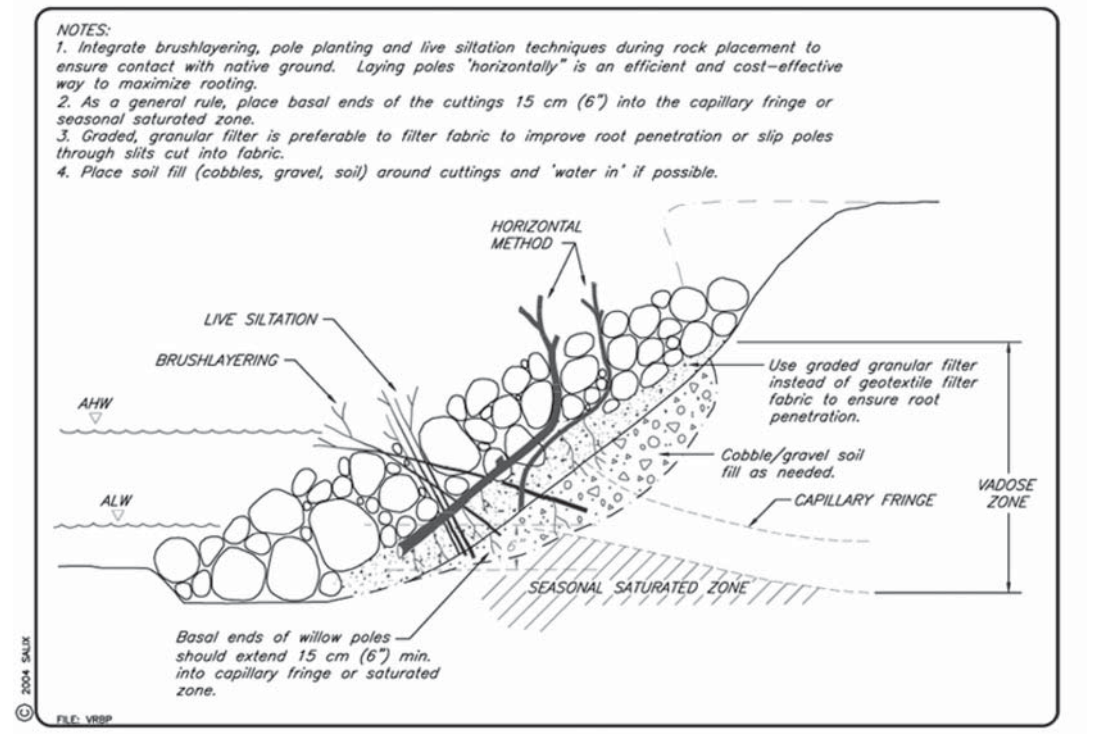
Four methods for constructing vegetated riprap have demonstrated effectiveness (see Figures 3.84 through 3.87 for design concept sketches):

- **Vegetated riprap with willow bundles:** Vegetated riprap with willow bundles is the simplest to install, but it has a few drawbacks. This technique typically requires very long 10- to 23-ft (3- to 7-m) poles and branches, as the cuttings should reach from 6 in (15 cm) below the low water table to 1 ft (30 cm) above the top of the rocks. In addition, only those cuttings that are in contact with the soil will take root, and, therefore, the geotechnical benefits of the roots from those cuttings on the top of the bundle may not be realized.
- **Vegetated riprap with bent poles:** Vegetated riprap with bent poles is slightly more complex to install, and is the only method that can be installed with filter fabric. Additionally, a variety of different lengths of willow cuttings can be used because they will protrude from the rock at different elevations.
- **Vegetated riprap with brush layering and pole planting:** Vegetated riprap with brush layering and pole planting is the most complex type of riprap to install but also provides the most immediate habitat benefits. This method can be installed by two techniques: one technique is used when building a bank back up, while the other is for a well-established bank. If immediate aquatic-habitat benefits are desired, this method should be used. However, vegetated riprap with brush layering and pole planting may not provide the greatest amount of root reinforcement, as the stem-contact with soil does not extend up the entire slope. A combination of this method with pole- or bundle-planted



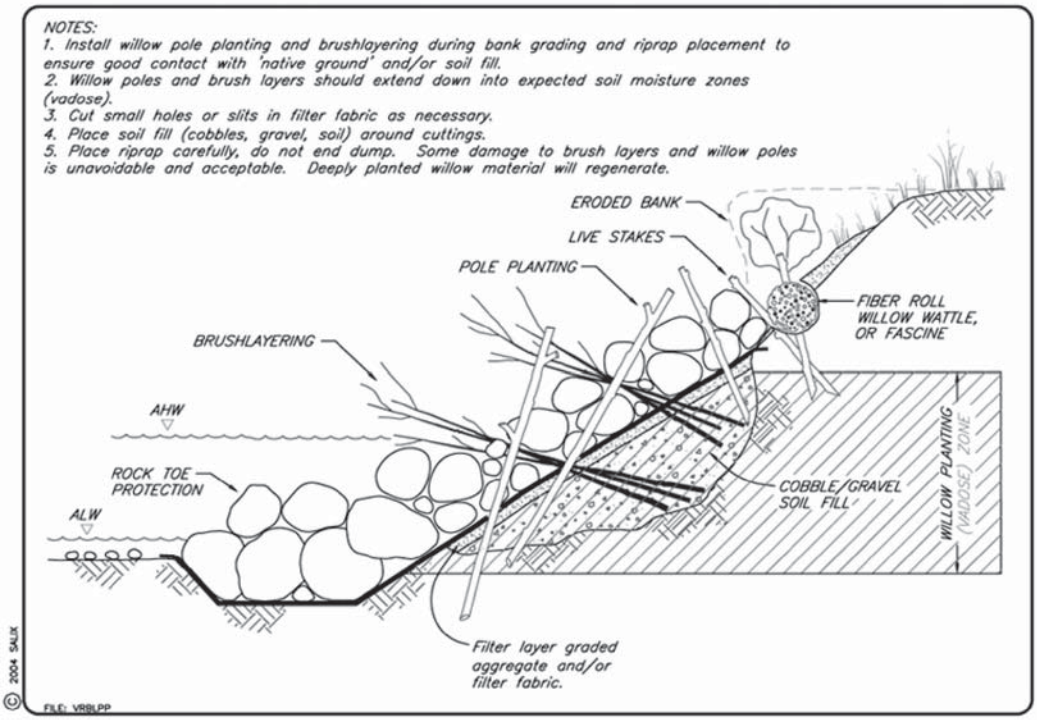
Source: McCullah and Gray (2005)

Figure 3.84. Vegetated riprap with willow bundles.



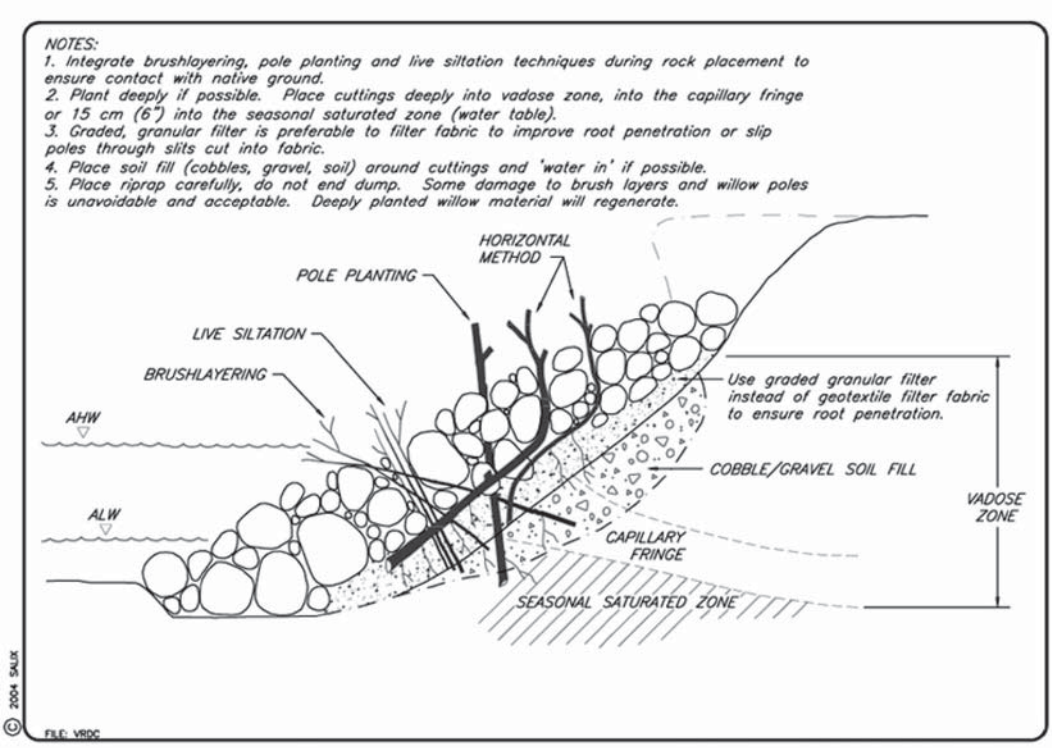
Source: McCullah and Gray (2005)

Figure 3.85. Vegetated riprap with bent poles.



Source: McCullah and Gray (2005)

Figure 3.86. Vegetated riprap with brush layering and pole planting.



Source: McCullah and Gray (2005)

Figure 3.87. Vegetated riprap with brush layering and pole planting – construction techniques.

riprap will perform well, as the latter methods typically have higher rooting success.
- **Vegetated riprap with soil cover, grass, and ground cover**: This method is also known as "buried riprap" and consists of infilling and covering a standard rock riprap installation with soil and subsequently establishing grass vegetation. Some stripping of the soil and grass may be expected during severe events.

Joint or live-stake planted riprap (Figure 3.88) is revegetated riprap, as opposed to the other techniques, which are true vegetated riprap methods. This method should be used only when attempting to get vegetative growth on previously installed riprap.

Environmental Considerations and Benefits

Many environmental benefits are offered by vegetated riprap; most are derived from the planting of willows or other woody species in the installation. Willow provides canopy cover to the stream, which gives fish and other aquatic fauna cool places to hide. The vegetation also supplies the river with carbon-based debris, which is integral to many aquatic food webs, and birds that catch fish or aquatic insects will be attracted by the increased perching space next to the stream (Gray and Sotir, 1996). The exclusive placement of predator-perching–type habitat may not be appropriate where fish-rearing habitat is desired. In that situation, large rocks and logs located above the average high water line (AHW) might be replaced with shrubby-type protective vegetation. An additional environmental benefit is derived from the use of rock, as the surface area of the rocks is substrate that is available for colonization by invertebrates (Freeman and Fischenich, 2000). The small spaces between the rocks also provide benthic habitat and hiding places for small fish and fry.

Limitations

Vegetated riprap may be inappropriate if flow capacity is an issue, as bank vegetation can reduce flow capacity, especially when in full leaf along a narrow channel. The critical threats to the successful performance of biotechnical engineering projects are (1) improper site assessment, design, or installation and (2) lack of monitoring and maintenance (especially following floods and during droughts) (Lagasse et al., 2001). Some of the specific limitations to the use of vegetation for streambank erosion control include the following:

- Lack of design criteria and knowledge about properties of vegetative materials
- Lack of long-term quantitative monitoring and performance assessment
- Difficulty in obtaining consistent performance from countermeasures relying on live materials
- Possible failure to grow and susceptibility to drought conditions
- Depredation by wildlife or livestock
- Possible need for significant maintenance

More important, the type of plants that can survive at various submersions during the normal cycle of low, medium, and high stream flows is critical to the design, implementation, and success of biotechnical engineering techniques.

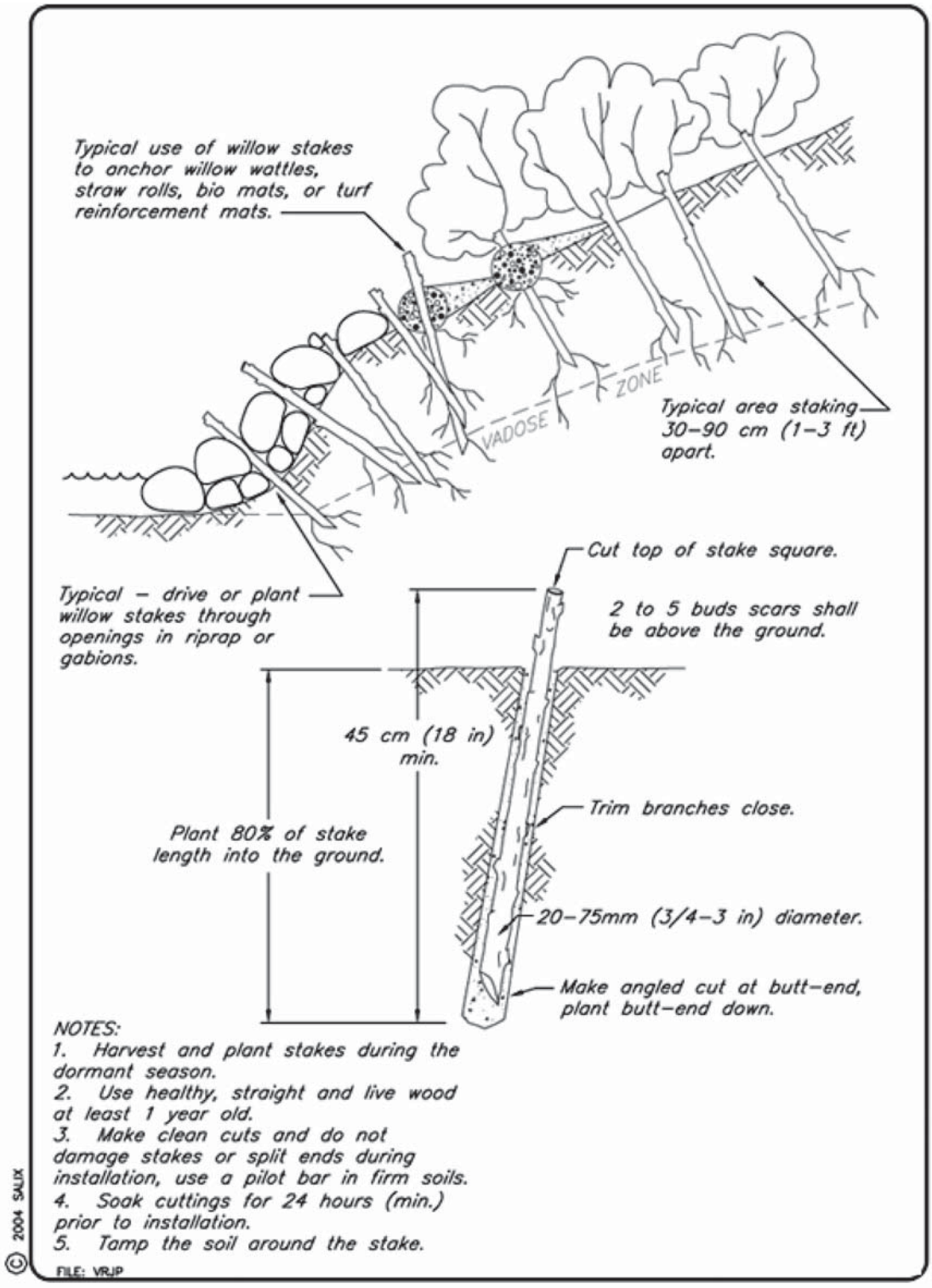
Common Reasons/Circumstances for Failure

Flanking, overtopping, or undermining of the revetment due to improperly installed or insufficient keyways is one of the biggest reasons for failure of riprap. Improperly designed or installed filter material also can cause undermining and failure of the installation. Undersized stones can be carried away by strong currents, and sections of the revetment may settle because of poorly consolidated substrate. Vegetation may require irrigation if planted in a nondormant state or in extremely dry soils. In addition, vegetation may be limited by excess soil moisture (Pezeshki et al., 1998). At a bridge, any revegetation effort should be directed away from the "hydraulic opening" of the bridge; that is, abutment fills under the bridge and through the waterway areal limits should not be planted (Racin et al., 2000).

As summarized in HEC-23 (Lagasse et al., 2001), biotechnical engineering can be a useful and cost-effective tool in controlling bank erosion or providing bank stability at highway bridges, while increasing the aesthetics and habitat diversity of the site. However, where failure of the countermeasure could lead to failure of the bridge or highway structure and danger to the user, the only acceptable solution may be traditional, "hard" engineering approaches. Biotechnical engineering needs to be applied in a prudent manner, in conjunction with channel planform and bed stability analysis, and rigorous engineering design. Designs must account for a multitude of factors associated with the geotechnical characteristics of the site, the local and watershed geomorphology, local soils, plant biology, hydrology, and site hydraulics. Finally, programs for monitoring and maintenance, which are essential to the success and effectiveness of any biotechnical engineering project, must be included in the project and strictly adhered to.

Design Concept Sketches

Typical design concept sketches of the five methods described previously are provided as Figures 3.84 through



Source: McCullah and Gray (2005)

Figure 3.88. Vegetated riprap with joint planting.

3.88. These sketches are reproduced from *NCHRP Report 544* (McCullah and Gray, 2005). Note that the key hydraulic design variable "design high water" is not indicated in these sketches.

3.9 Riprap Design Software

Because riprap sizing equations are easy to apply and generally can be accomplished with hand calculations or a spreadsheet, riprap design software has only marginal utility. In addition, unless design software is maintained and revised, it can become dated and not reflect the current state of practice. The critical input parameters for riprap design are hydraulic and flow variables, which the software does not provide. The synthesis of Section 2.4.5 provides basic information on riprap design software. This section provides an annotated description of the software listed in Table 2.7 and a reference data set.

3.9.1 Riprap Software Synopses

West Consultants' Riprap Design System (Version 2.0) computes riprap sizing for channel bed and bank revetment. The software includes seven design equations: (1) EM 1601 (USACE, 1991), HEC-11 (Brown and Clyde, 1989), Engineering Monograph No. 25 (Peterka, 1978), USGS Water Resources Investigations Report 86-4127 (Blodgett, 1986), CABS (Racin et al., 2000), Isbash (from EM 1601), and ASCE Manual 54 (ASCE, 1975). This software treats the CABS equation as if it were providing the d_{50} size of a single-layer system rather than the uniform rock size of the outer layer of a multiple-layer system.

The HYDRAIN/HYCHL software developed by GKY and Associates for roadside drainage analysis includes riprap sizing calculations based on HEC-15 (Chen and Cotton, 1988) and HEC-11 (Brown and Clyde, 1989) procedures.

The CHANLPRO software computes riprap size using the EM 1601 procedure. The software also provides guidance on gabion mattress design and on estimating scour in erodible channels. CHANLPRO software and documentation is available for download from the USACE, Coastal and Hydraulics Laboratory website.

SAM is a hydraulic design package developed by USACE for computing normal-depth hydraulics, sediment transport capacity, and sediment yield. SAM includes riprap sizing calculations based on the EM 1601 procedure. SAMwin was developed by Ayres Associates and is a Windows interface to the SAM software. SAMwin is available to USACE users through the Coastal Hydraulics Laboratory, to FHWA and state DOTs through FHWA, and to all other users through Ayres Associates.

STONE3 computes a factor of safety (stability factor) using the HDS 6 procedure. The input data include riprap size, riprap angle of repose, bank angle, water depth, and water velocity. The software may be available through NYSDOT.

PB_Riprap analysis and design software includes riprap sizing for revetment, pier, abutment, and propeller wash applications. There are three revetment equations, including the HEC-11 approach. None of the three procedures include any correction for bend curvature. There are also three pier equations including the HEC-23 equation. The abutment equation includes the HEC-23 method but relies on the user to input the characteristic velocity computed using the SBR method.

3.9.2 Reference Data Sets

Because riprap size calculations are relatively simple, the use of specialized software is probably not warranted unless a comparison of a large number of equations is desired. For the three recommended equations from this study (EM 1601 for revetment, HEC-23 for pier, and HEC-23 for abutment), reference data sets (Tables 3.20, 3.21, and 3.22) are provided as test data sets for hand calculations or spreadsheet applications. Two data sets are provided for each riprap application. The gravitational constant (g) is assumed to be 32.2 ft/s^2 (9.81 m/s^2). Design examples using these reference data sets are provided in Appendix C.

3.10 Implementation

3.10.1 The Product

As described in more detail in the preceding sections, the products of this research include guidelines and specifications for design, materials, test methods, construction inspection, and quality control for the use of riprap for a range of erosion control and bridge scour applications.

3.10.2 The Market

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

Table 3.20. Revetment riprap reference data sets.

Variables	English Units			SI Units		
	Units	Data Set 1	Data Set 2	Units	Data Set 1	Data Set 2
Given Data						
Average channel velocity	ft/s	7.2	7.6	m/s	2.19	2.32
Flow depth at bank toe	ft	11.4	10.2	m	3.47	3.11
Bank side slope (xH:1V)		2.0	1.8		2.0	1.8
Bank slope	Deg.	26.6	29.1	Deg.	26.6	29.1
Channel centerline radius of curvature	ft	500	175	m	152.4	53.3
Channel width	ft	100	50	m	30.5	15.2
S_g		2.54	2.65		2.54	2.65
Safety factor (S_f)		1.2	1.1		1.2	1.1
Riprap angularity		Rounded	Angular		Rounded	Angular
Computed						
K_1 (side slope correction factor)		0.87	0.82		0.87	0.82
Rc/W		5.0	3.5		5.0	3.5
Cs		0.36	0.30		0.36	0.30
Cv		1.14	1.17		1.14	1.17
Vss	ft/s	9.9	11.1	m/s	3.01	3.38
d_{30}	ft	0.75	0.79	m	0.23	0.24
d_{30}	in	9.0	9.5			
$d_{50} = 1.2d_{30}$	in	10.8	11.4	m	0.28	0.29

Table 3.21. Pier riprap reference data sets.

Variables	English Units			SI Units		
	Units	Data Set 1	Data Set 2	Units	Data Set 1	Data Set 2
Given Data						
Velocity at pier	ft/s	6.6	7.5	m/s	2.01	2.29
Pier shape		Square	Round		Square	Round
S_g		2.5	2.65		2.5	2.65
Computed						
K (shape factor)		1.7	1.5		1.7	1.5
d_{50}	ft	0.90	0.82	m	0.27	0.25
d_{50}	in	10.8	9.9			

- National Park Service
- Forest Service
- Bureau of Indian Affairs
- Any other governmental agency with bridges under their jurisdiction
- Consultants to the agencies above

3.10.3 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 impediment can be countered by a well-planned technology transfer program.

Because of the complexity and geographic scope of riprap applications, 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 guidelines and specifications 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.

3.10.4 Leadership in Application

Through the NHI and its training courses, **FHWA** can reach a diverse and decentralized target audience.

Table 3.22. Abutment riprap reference data sets.

Variables	English Units			SI Units		
	Units	Data Set 1	Data Set 2	Units	Data Set 1	Data Set 2
Given Data						
Main channel average flow depth	ft	8.3	9.2	m	2.53	2.80
Flow depth at toe of abutment	ft	2.8	2.6	m	0.85	0.79
Abutment toe setback from channel bank	ft	20	80	m	6.1	24.4
Total discharge	cfs	4000	6000	m^3/s	113.3	169.9
Overbank discharge	cfs	400	1200	m^3/s	11.3	34.0
Total bridge area	ft^2	520	830	m^2	48.3	77.1
Setback area	ft^2	56	208	m^2	5.2	19.3
Abutment shape		Spill Through	Vertical Wall		Spill Through	Vertical Wall
S_g		2.65	2.5		2.65	2.5
Bridge average velocity	ft/s	7.7	7.2	m/s	2.35	2.20
Maximum channel velocity	ft/s	9.1	8.4	m/s	2.77	2.56
Computed						
Set-back ratio		2.4	8.7		2.4	8.7
Riprap design velocity	ft/s	7.7	5.8	m/s	2.35	1.76
Local Froude number		0.81	0.63		0.81	0.63
HEC-23 equation		8.3	8.2		8.3	8.2
K (abutment shape factor)		0.61	1.02		0.61	1.02
Riprap d_{50}	ft	0.98	0.70	m	0.30	0.22
Riprap d_{50}	in	11.8	8.4			

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 committee 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. Obviously, material quality standards for revetment materials 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. The ASCE Task Committee on Bridge Scour could 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 riprap design, installation, and inspection and their acceptance of the results of this research will be key to implementation by bridge owners.

3.10.5 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 adapted and/or 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.

3.10.6 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 gauged 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, practical, and reliable methods for designing, installing, and inspecting riprap for a range of erosion control and bridge scour applications. The ultimate result will be a reduction in the number of bridge failures and reduction in damage to highway facilities attributable to scour and erosion.

CHAPTER 4

Conclusions and Suggested Research

4.1 Applicability of Results to Highway Practice

Approximately 83% of the 583,000 bridges in the National Bridge Inventory (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 erosion and scour account for most of the bridge failures in this country. A 1973 study for 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 erosion and 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 the inability to design and install riprap 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 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 unified design guidelines and specifications for other potentially effective riprap applications has resulted in an unacceptable level of uncertainty when riprap is used as a countermeasure on riverbanks, abutments, guide banks, spurs, and other locations requiring scour countermeasures. The guidelines, specifications, and recommendations from this research will provide more definitive and unified guidance and specifications for design, installation, and inspection of riprap. The end result will be a more efficient use of highway resources and a reduction in costs associated with the impacts of erosion and scour on highway facilities.

4.2 Conclusions and Recommendations

4.2.1 Overview

This research accomplished its basic objectives of developing design guidelines; recommended material specifications and test methods; recommended construction specifications; and construction, inspection and quality control guidelines for riprap for a range of applications, including revetment on streams and riverbanks, bridge piers and abutments, and bridge scour countermeasures such as guide banks and spurs. A fundamental premise of this study is that riprap is an integrated system and as such, successful performance of a riprap installation depends on the response of each component of

the system to hydraulic and environmental stresses throughout its service life.

Many different techniques are currently used to determine the size and extent of a riprap installation, and existing techniques and procedures for design of riprap protection can be confusing and difficult to apply. Depending on the technique used to size riprap, the required size of stone can vary widely. Most states have their own specifications for classifying riprap size and gradation and there is not a consistent classification system or set of specifications that can be used when preparing plans or assembling a specification package for a project. In addition, various construction practices are employed for installing riprap; many of them are not effective and projects requiring the use of riprap historically have suffered from poor construction practices and poor quality control. **The intent of this study was to develop a unified set of guidelines, recommended specifications, and procedures that can be accepted by the state DOTs.** As a result of a similar effort, the European Union recently adopted a unified standard for riprap that transcends geographic and institutional boundaries (CEN, 2002).

Conclusions and recommendations for each of the functional areas investigated for the riprap applications of interest to this study are summarized in the following sections.

4.2.2 Riprap Design Equations

Design equations for sizing riprap were evaluated with sensitivity analyses using laboratory and/or field data, where available, for the applications of interest to this study. Based on the sensitivity analyses, the following design equations or design approaches are recommended for each application.

- For revetment riprap, the USACE EM 1601 equation is recommended as the most comprehensive approach for sizing riprap considering the ability of the basic equation to discriminate between stable and failed riprap, bank and bend correction factors, and the reasonableness of safety/stability factors (Section 3.2.1).
- For pier riprap, the HEC-23 equation is recommended as the most reliable design equation for sizing riprap. The velocity multiplication factors for round- and square-nose piers were confirmed using available laboratory data (Section 3.3.1).
- For abutment riprap, the FHWA SBR method as presented in HEC-23 was confirmed, using 2-D modeling, as an accurate approach for estimating flow velocity and sizing riprap at an abutment. It is recommended, however, that the computed characteristic average velocity not exceed the maximum velocity in the channel (Section 3.4.1).
- For guide bank riprap, the abutment riprap design equations can be used. The recommended velocity for computing riprap size at a guide bank is 0.85 times the velocity estimated using the SBR method for an abutment (Section 3.5.1).
- No definitive guidance for sizing riprap could be derived from 2-D modeling of the flow field around flow control structures such as spurs; however, USACE (EM 1601) provides some guidance. Engineering judgment and conservatism is recommended for sizing riprap for zones of high stress such as the nose of a spur (Section 3.5.1).
- Designing riprap for overtopping flow conditions on roadway embankments and flow control structures such as guide banks and spurs is also of concern. An equation derived from laboratory experiments for the Bureau of Reclamation is recommended (Section 3.5.1).

4.2.3 Filter Requirements

Filter design criteria are the most overlooked aspect of riprap design. More 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. Filter processes and existing methods for design and placement were thoroughly investigated and discussed. Design and placement guidance for both granular and geotextile filters is provided.

- Historically in the United States, the Terzaghi criteria have been used for design of granular filters. An alternative approach, widely used in Europe, that follows the Cistin–Ziems methodology is recommended for consideration as a practical alternative for filter design (Section 3.2.2).
- For many applications, placing a geotextile filter under water is a challenge. For low-velocity applications, a blanket-like product, SandMat™, is used in Germany. The SandMat™ is essentially a blanket of two non-woven geotextiles (or a woven and a non-woven) with a layer of sand in between. The composite blanket has a high specific gravity so it sinks readily. For higher velocity or deep water applications, German practice calls for 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 stable against erosive forces under a range of conditions, including wave-attack environments. There are many applications where adoption of these approaches to filter placement in U.S. practice would be highly beneficial (Section 3.2.2).
- The laboratory testing phases of NCHRP Projects 24-07(1) and 24-07(2) included evaluation of riprap as a pier scour countermeasure. For this application, it was found that granular filters performed poorly in the case where bedforms are present. Specifically, when dune troughs that are deeper than the riprap armor move past the pier, the

underlying finer particles of a granular filter are rapidly swept away. The result is that the entire installation becomes progressively destabilized beginning at the periphery and working in toward the pier. It is strongly recommended that only geotextile filters be used at bridge piers in riverine systems where dune-type bed forms may be present during high flows. These laboratory studies also resulted in the finding that geotextile filters at piers should not be extended to the periphery of the riprap, but instead should terminate at two-thirds the riprap extent. With these two exceptions, the remainder of the guidance provided for filters for revetment riprap is appropriate for riprap installations at bridge piers (Section 3.3.2).

- The guidance provided for filters for revetment riprap is generally appropriate for riprap installations at bridge abutments located on floodplains and set back from the main channel. In the case where the abutment is integral with the bank of the main channel, the same concern regarding the use of granular filters exists as for pier riprap. That is, if dune troughs passing the abutment are deeper than the riprap apron thickness, the underlying finer particles of a granular layer can be swept away rapidly. The result is that the entire riprap installation becomes progressively destabilized beginning at the periphery and working in toward the abutment. For this reason, it is strongly recommended that only geotextile filters be used at bridge abutments in riverine systems where dune-type bed forms may be present during high flows, and where the abutment and/or abutment riprap apron extend into the main channel. In addition, where the abutment and/or abutment riprap apron extend into the main channel, the geotextile filter should not be extended to the periphery of the riprap, but instead should terminate at two-thirds the riprap extent (Section 3.4.2).
- The guidance provided for filters for revetment riprap is generally appropriate for countermeasures constructed of or armored by riprap, such as guide banks or spurs (Section 3.5.2). Scour at the nose of the guide bank or spur is of particular concern. Additional riprap should be placed around the upstream end of the guide bank or spur to protect the embankment material from scour as this is the most likely failure zone for these countermeasures (Section 3.7.4).

4.2.4 Material and Testing Specifications

Currently, material and testing specifications for riprap available in the United States (e.g., AASHTO, ASTM) are generally adequate for determining riprap quality. However, there is little consistency in specifications for riprap gradation properties. For example, many gradation specifications can be interpreted to result in an essentially uniform rock size where a more widely graded mixture was intended by the designer. In addition, the wide variety of size designations (classes) among agencies results in confusion and, potentially, increased project cost. A standardized methodology was developed and is recommended for U.S. practice. The method considers both the rock size and slope of the riprap particle distribution curve, as well as typical rock production methods.

- Riprap gradations from six methods most often used in the United States and Europe were examined and compared. A gradation classification system that meets the needs of the designer, producer, and contractor was developed. A classification system consisting of 10 standard classes is proposed (Section 3.2.3).
- A standardized method for converting stone size as a dimension to an equivalent weight is proposed based on the work of Galay et al. (1987). The method is based on the intermediate or B axis of the particle and the rock's specific gravity and assumes a volume equal to 85% of a cube (Section 3.2.3)
- Material properties and testing requirements for both the field and laboratory from ASTM, OSM, AASHTO, CUR, and CEN were investigated and specific recommendations adapted to the revetment riprap application are provided in Section 3.2.3.
- The requirements for the quality and characteristics of riprap materials, and the associated tests to support those requirements, are presented for revetment riprap installations in Section 3.2.3. These requirements are suitable for use with riprap used to protect bridge piers and abutments and to construct or armor scour countermeasures.
- It was apparent from the survey of current practice that very little field testing during construction or inspection is done on a programmatic basis. A simple methodology developed by OSM is recommended to facilitate a decision to accept or reject a rock product at the quarry or on site. In addition, a pebble-count approach for verifying size distribution of riprap at the quarry or construction site is suggested for U.S. practice (Sections 3.2.3 and 3.2.5).

4.2.5 Construction/Installation Guidelines

A generalized overview of riprap construction methods and placement techniques was developed for installations both in the dry and under water (Section 3.2.4). The following topics were considered:

- Quarry operations
- Equipment overview
- Loading and transportation of riprap
- Placing riprap and the filter

- Terminations and transitions
- Site considerations
- Measurement and payment

Construction and installation guidance for the applications of interest to this study were developed and are included in Appendix D.

4.2.6 Inspection and Quality Control

According to a survey of current practice in the United States, very little guidance is being promulgated by the DOTs for riprap inspection and quality control either during construction or for long-term monitoring. A field test procedure described by Galay et al. (1987) is presented as an example of a simple, practical approach to ensuring that an appropriate riprap size distribution is achieved during construction and that the stone does not deteriorate over the long term (Section 3.2.5). Other field tests suitable for inspection and quality control are discussed in Section 3.2.3.

A suggested riprap inspection code was developed. This code parallels the format of Item 113, "Scour Critical Bridges," of NBIS and would be applicable to all riprap installations including revetments and riprap at bridge piers, abutments, and countermeasures (see Appendix D). The code provides a numeric ranking scheme based on both the observed condition of the entire riprap installation as well as the condition of the riprap particles themselves. The code is intended to serve for underwater inspections as well as for installations that can be observed in the dry. Action items associated with the coding guidance are also provided with the inspection code.

4.2.7 Other Topics Considered

Several additional topics were considered relevant to the objectives of this study. While these topics did not directly support the development of design guidelines for riprap applications, they do contribute to the comprehensive overview of riprap technology undertaken in this study.

- The results obtained from any riprap design (sizing) equation are dependent on the quality of the hydraulic variables used in the computations. The level of hydraulic analysis should be commensurate with the importance and/or cost of the riprap installation. To improve the quality of the riprap design process, guidance was developed on the use of 1-D and 2-D modeling to obtain appropriate input design variables (Section 3.6.2).
- As with any complex engineering design process, much can be learned from experience and, where available, well documented field performance and failure studies. Riprap failure mechanisms were identified as a basis for developing inspection guidance, and selected case studies of failures are used to emphasize the need for post-flood/post-construction inspection and performance evaluation (Section 3.7).
- For many applications, the use of a hybrid design consisting of standard riprap protection at the toe and transitioning to less heavy duty or vegetative treatment on the mid- to upper bank slopes can present an attractive alternative to traditional techniques of full riprap armor. Detailed design guidelines were not developed for a hybrid design, but the concepts are discussed. Although these biotechnical engineering approaches can be a useful and cost-effective tool in controlling bank erosion or providing bank stability and increasing the aesthetics and habitat diversity of the site, where failure of the countermeasure could lead to failure of the bridge or highway structure and danger to the user, the only acceptable solution may be traditional, "hard" engineering approaches (Section 3.8).
- There is a limited availability of riprap design software but several alternatives were evaluated. Because riprap size calculations are relatively simple, the use of specialized software is probably not warranted unless a comparison of a large number of equations is desired. The critical input parameters for riprap design are hydraulic and flow variables, which the software does not provide. In addition, unless design software is maintained and revised, it can become dated and not reflect the current state of practice. For the three equations recommended in this study (EM 1601 for revetment, HEC-23 for pier, and HEC-23 for abutment), reference data sets are provided as test data sets for hand calculations or spreadsheet applications (Section 3.9.2). These data sets are also applied in a series of design (sizing) examples in Appendix C.

4.2.8 Design Guidelines

To guide the practitioner in developing appropriate riprap designs and ensuring successful installation and performance of riprap armoring systems for bankline revetment, at bridge piers, and at bridge abutments and guide banks, the findings of Chapter 2 and the recommendations of Chapter 3 are combined to provide detailed guidelines in a set of supplemental materials:

- Appendix C, Guidelines for the Design and Specification of Rock Riprap Installations
- Appendix D, Guidelines for the Construction, Inspection, and Maintenance of Rock Riprap Installations
- Drawings of Typical Details (AutoCAD®, MicroStation®, and Adobe® Acrobat® formats) available from the TRB website (http://www4.trb.org/trb/crp.nsf/All+Projects/NCHRP+24-23)

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

This study did not involve any original laboratory experiments, but some analytical work (specifically, 1-D and 2-D computer modeling) was necessary to address issues related to input hydraulic variables for design. The findings of Chapter 2 and the appraisal of Chapter 3, including the sensitivity analyses of design equations using laboratory and field data, are the basis for recommended design methods, material tests, specifications, and the guidelines for each riprap application presented in Appendixes C and D. In developing the design guidelines, additional information or data would have supported more detailed guidance or specificity in several areas. The following suggestions for research would permit extending the recommendations of this study in these areas:

- Detailed 2-D modeling of flow at bridge abutments and guide banks verified the SBR approach and provided recommended adjustments for obtaining a characteristic average velocity for sizing riprap at abutments and on guide banks. Two-dimensional modeling did not yield definitive results that could be used to design riprap for flow control structures such as spurs (or bendway weirs). Additional computer modeling or, preferably, physical modeling in a hydraulics laboratory could provide valuable data for enhancing the design guidelines for spurs. Such modeling could also address similar design issues for bendway weirs.
- The guidelines for abutment riprap are based, primarily, on available guidance in FHWA's HEC-23. The results of NCHRP Project 24-18A, "Countermeasures to Protect Bridge Abutments from Scour," were not available to be evaluated or included in this study. When available, these results could support refining the guidelines for riprap as an abutment scour countermeasure. When available, the results of NCHRP Project 24-20, "Prediction of Scour at Bridge Abutments," should also be reviewed and evaluated with reference to the recommendations of this study.
- The scope of work in the Research Work Plan for this study was predicated on the assumption that the literature search would produce several laboratory and field data sets in each of the application areas as a basis for sensitivity analyses and developing design guidelines. For revetment riprap, only the field data set compiled by Blodgett and McConaughy (1986) and the laboratory data set from the studies of Maynord (1987, 1990) and Maynord et al. (1989) met the needs of this study. Although laboratory data sets were available for the pier and abutment riprap applications, each had limitations; no field data sets were found. Neither laboratory nor field data were available for the countermeasure applications. Additional laboratory studies for these applications and studies to gather field and performance data would be extremely valuable in extending the results of this study.
- State DOTs and other bridge owners should invest in post-project monitoring and maintenance reporting on significant riprap projects or those with innovative designs (e.g., use of geotextile bags as a filter for pier scour riprap). Funding should be allocated to this activity to support development of a performance database. Diligent design and construction inspection documentation is essential. The site initial history and recommendations on post-flood inspections provide vital information to support a "forensic" analysis at both successful and unsuccessful installations.
- Inadequate or improperly designed and installed transitions or toe downs for riprap in all applications were found to be one of the most frequently cited reasons for failure of riprap armoring systems. Installation of a properly designed filter at transitions or toe downs is equally critical. In particular, the size and volume of riprap for a launched toe, or toe key application is based largely on experience and engineering judgment. Again, laboratory and field studies could provide valuable guidance for this component of riprap design.
- Geotextiles are increasingly being used as the filter material of choice for riprap installations. It is typically assumed that if the geotextile survives the loads and stresses during initial construction, it will be fine for the remainder of its service life. However, some concern still exists regarding the long-term durability of geotextiles, environmental conditions that could lead to deterioration or loss of functional properties, and potential for gradual fouling or clogging by physical or biological processes. Funding should be allocated to exhume and test geotextile specimens taken from a variety of riprap installations and over a range of environments. Both woven and non-woven fabrics should be included in the study.

CHAPTER 5

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APPENDIX A

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APPENDIX B

Survey of Current Practice

1 Overview

In the summer of 2003, a survey questionnaire was distributed to approximately 80 individuals representing state DOTs and other government agencies that have experience with design, construction, maintenance, and inspection of riprap structures. The purpose of this survey was to gather information on the current state of practice with respect to a variety of topics related to riprap installations.

A total of 33 completed questionnaires were received. Twenty-four states are represented by the completed questionnaires with eleven states located west of the Mississippi River and thirteen located east of the Mississippi. Most of the individuals responding were from state DOTs. Several federal agencies were represented and one consulting firm responded. Of the 37 individuals who participated in completing the surveys (some surveys were completed by several people), seven identified themselves as licensed engineers. The following sections provide a general summary of the responses given that pertain to the subject listed in the section heading.

2 Design Guidelines

The first two parts of the questionnaire addressed design guidelines. Table B.1 was developed from Part 1, Question 1, and shows the number of "Yes" answers for each design method used for various riprap applications. Where respondents indicated "Other" as shown in the table, responses were varied with no consistency among the answers.

For computing hydraulic conditions, the U.S. Army Corps of Engineers' HEC-RAS computer model was cited most often (27 times) as the means of determining hydraulic variables for use with a design method. Thirteen respondents provided comments in relation to question 1 and, in general, these comments were varied and did provide some useful and instructive information.

Every respondent provided answers to Questions 2 through 5 in Part 1. A summary discussion of these responses is provided as follows:

- Most respondents use DOT standard specifications for the allowable gradation of the stones composing a riprap blanket. Some respondents actually listed what those gradations/criteria were.
- Many attachments (23) were included that listed the standard rock size categories available. If a respondent did not include an attachment, the categories were typically listed in his/her reply.
- Most answers referred to some multiple of d_{50} and/or d_{100} for specifying the total thickness of the riprap blanket.
- Twenty-one respondents addressed toe design. Responses were split between qualitative and quantitative methods. Several mentioned that toe design was dependent upon anticipated (calculated) scour.
- The 100-year storm was the event most cited when designing riprap facilities. Some use a 50-year event and others said the design event is dependent upon the roadway classification and project specifics.
- Most respondents indicated there was no requirement for minimum service life.

Less than half of the respondents described how site conditions outside the intended limits of the design equation are addressed. If there was a response, it most often listed a criterion associated with a design limit such as maximum allowable side slope of 1.5H:1V, or that riprap can be used on designs only when the Froude number is less than 0.8. Overall, there was not much detail in the responses relating to extreme or challenging site conditions.

Many comments were elicited concerning the respondents' experience with the general performance of riprap installations. Typically riprap performance history was characterized as satisfactory to good, based on the respondents' experience.

Table B.1. Responses to questions regarding design methodologies most often used in practice.

Design Method	Riprap Application						
	Pier Scour	Abutment Scour	Bank Revetment	Spurs and Groins	Guide Banks	Grade Control*	Other (specify)
HEC-23 (piers)	21	3	3	2	2	3	2
HEC-23 (abutments)	2	22	7	3	3	4	1
HEC-11	5	5	17	7	6	7	2
Colo. State Univ.	3	1	0	0	0	1	1
Corps of Engrs. (specify)	0	0	2	2	1	2	5
California B&SP	0	1	5	1	1	0	1
Other (specify)	5	6	9	5	3	6	7

*Check dams, weirs, rock trenches, etc.

Gradation was often mentioned as being a critical factor in performance. ***Problems associated with the toe were cited as the number one cause of failure***.

Selection and installation of the filter was repeatedly mentioned as critical to riprap performance. Failure of the filter was mentioned as a fairly common mode of riprap installation failure in general. Construction and the experience of the contractor were listed as important when considering performance of a riprap installation. Several respondents noted failures where installation occurred on steep grades.

When asked, "Do you consider hybrid designs, such as the use of larger rock at the toe, transitioning to smaller rock (or alternate materials, including biotechnical stabilization) on the upper banks?" seven respondents replied "No" while most others mentioned the use of larger rock transitioning to smaller rock sizes dependent upon location with respect to the toe. There were several responses that made some mention of bio-stabilization although this does not seem to be a widespread practice.

Part 2 of the questionnaire includes questions relating to filter design. Most (23) respondents indicated that geotextile filters were required with riprap installations, and 11 indicated that granular filters were required where the designer felt it was necessary. ***Comments generally indicate that geotextiles are the preferred filter material***.

Other comments address the use of granular filters underwater and a variety of instances when a filter is not used. HEC-11 was identified as the method most often used to design and specify a filter for riprap. However, a wide variety of other methods was cited as well. Twelve respondents provided detailed comments on their filter design method. Over half of the respondents do not make any distinction between designing and placing a filter under water versus in the dry. Approaches for those who do make a distinction varied with no one consistent approach.

Eight respondents indicated they do allow cutting the geotextile where vegetation is proposed while 12 do not allow cutting or do not use geotextiles at all in conjunction with vegetation.

3 Material and Testing Specifications

Part 3 of the questionnaire addresses rock quality. Most respondents stated that standard specifications developed by their state DOT are the criteria used to ensure the rock is competent and durable for use as riprap. Several respondents included their specifications as an attachment to the completed questionnaire. Several respondents mentioned specific tests such as LA Rattler abrasion testing and AASHTO-T104.

Most respondents rely only on laboratory testing for the determination of rock quality. If a field procedure was indicated, visual inspection was the most common response. ***There was very limited evidence that actual field testing is performed to determine rock quality, either during construction or at the time of periodic inspections***.

Allowing concrete rubble to be substituted for rock elicited a split response. Fifteen respondents indicated that substitution is acceptable. Most of those "Yes" replies included some type of criterion for substitution to be allowed. Twelve respondents stated that concrete rubble absolutely cannot be substituted, or else had very restrictive limitations for such a substitution.

Of the 21 persons providing an answer to the question, "Do you account for variability in specific gravity in your design procedure?" 16 said "No" and five responded "Yes." When mentioned, 2.5–2.65 was the range given for minimum allowable specific gravity.

Most respondents referred to their applicable state standard specifications in response to the questions, "Do you have

Table B.2. Responses to questions regarding placement techniques most often used in practice.

Application	Number of Responses	Notes
River bank	10	Mechanical or hand placement
River bed	7	
Bridge piers	9	Placement preferred over dumping
Bridge abutments	8	
Spurs or groin fields	7	Not much experience with these structures
Guide banks	8	
Other	20	Several references to attached specs; dump in place a common method

any requirement or criteria for rock angularity? Do your design and QA/QC procedures address the 'shape' of the rock particles?" Six respondents replied that they do not have any guidance for rock shape or angularity.

4 Construction and Installation Guidelines

Riprap placement is addressed in Part 4 of the questionnaire. Descriptions of the typical placement techniques used for rock riprap in various applications were limited, but the answers we did receive were wide ranging. Table B.2 shows the number of responses pertaining to placement for each riprap application and any notes on the types of answers received.

Only five respondents indicated a distinction between techniques for placement under water versus in the dry. Fifteen said they make no distinction between wet versus dry construction. Some noted that for wet installations, dewatering is performed before riprap installation through the use of sandbags, cofferdams, diversions, or other method.

A wide variety of answers was given for the series of questions: "Please describe the considerations given to edge and end termination treatments of riprap installations. How do you transition the riprap back to the native soil? What are your criteria for the lateral extent of riprap away from structures such as piers, abutments, toe of slope?" Some respondents provided standard specifications and details with their response. Several mentioned a "keyed" approach at the toe of an application or for use as an end treatment. For the transition of riprap back to the native soil, a few respondents said the riprap was "blended" to meet existing ground elevations. As for lateral extent away from piers, two times the pier width was mentioned more than once.

5 Inspection and Quality Control

Very few of the respondents had ever performed a case study of riprap performance at a specific site. Of the four who had, one respondent attached three examples of case studies.

Defining "failure" of riprap installations elicited a large number of responses with specific information. The following responses were received regarding how riprap failure was identified:

- Undercutting or soil erosion
- Outflanking
- Piping
- Substantial riprap movement, sloughing, sinking, or removal (washed away)
- Exposure of the underlying soil

Overall, the number one reason for failure cited was some form of displacement of the riprap particles (sloughing, washing away, etc.).

The question, "Please describe your riprap inspection program (frequency, method/protocol, differences between dry vs. underwater installations, etc.). If you have a standard field evaluation form, please attach," produced the most consistent results in the survey. Bi-annual inspection was the most common answer given for frequency of inspections. Many respondents indicated that riprap inspections are conducted in conjunction with a bridge inspection program or after a flood event. ***No inspection forms were provided by any of the respondents***.

6 Applications

The last part of the questionnaire asks respondents to provide samples of riprap specifications from actual projects and examples of standard forms, design templates, typical details, etc. that are used for riprap design and/or installation. Twenty-eight of the returned questionnaires included at least one attachment. Table B.3 shows the number and type of attachments received in conjunction with the survey. Several of the attachments, because of their content, were counted in more than one category.

Table B.3. Supplemental information received with the returned questionnaires.

Attachment Type	Number
Actual project examples	11
Riprap specifications; stone requirements; gradation requirements	23
DOT riprap for slope and erosion protection specifications (including construction specifications)	18
Standard details/drawings	7
Design examples and calculation templates	2
Test data	1
Design methods	4
Geotextile specifications	3
Failure investigations	3
Miscellaneous references for erosion protection, related studies, etc.	5

APPENDIX C

Guidelines for the Design and Specification of Rock Riprap Installations

1 Introduction, C-2
2 Revetment Riprap, C-2
3 Riprap at Bridge Piers, C-7
4 Riprap at Bridge Abutments and Guide Banks, C-9
5 Riprap for Spurs, C-19
6 Riprap for Overtopping Flow, C-21
7 Filter Requirements, C-25
8 Materials, C-31
9 References, C-35

1 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 guideline provides recommended approaches for designing and specifying riprap for the following applications:

- Section 2: Revetment Riprap (slope protection)
- Section 3: Riprap at Bridge Piers
- Section 4: Riprap at Bridge Abutments and Guide Banks
- Section 5: Riprap for Spurs
- Section 6: Riprap for Overtopping Flow

Design of a riprap installation requires knowledge of river bed and foundation material; flow conditions including velocity, depth, and orientation; riprap characteristics of size, density, durability, and availability; and the type of interface material between the riprap and underlying foundation. At bridges, the size, shape, and skew angle of piers with respect to the flow direction must be known, and the location and type of abutments (spill-through or vertical wall) must be determined. 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. Filter design is addressed in Section 7.

Section 8 provides guidance on riprap size, shape, and gradation. Ten standardized gradation classes are proposed. Recommended specifications for physical properties of rock for riprap, and of geotextiles for filters used in conjunction with riprap, are provided along with the recommended test procedures for determining these properties.

Reference documents that provide the basis for the guidance in this document are cited in Section 9.

The guidance provided in this document has been developed primarily from the results of NCHRP Project 24-23 (Lagasse et al., 2006a) and the Federal Highway Administration (FHWA) Hydraulic Engineering Circular No. 23 (HEC-23) (Lagasse et al., 2001). The guidelines should be closely examined and modified, as appropriate, for local design practices, specification tests, specification values, and procedures for materials testing, construction inspection, and periodic maintenance inspection.

2 Revetment Riprap

2.1 Sizing the Riprap

To determine the required size of stone for revetment riprap, NCHRP Project 24-23 recommends using the method developed by Maynord et al. (1989) and Maynord (1990) and published by the U.S. Army Corps of Engineers (USACE) as Engineering Manual No. 1110-2-1601 (EM 1601) (USACE, 1991). The values of coefficients used in the following equation are provided in equations with the variable definitions (below) and given graphically in Appendix B of EM 1601 (USACE, 1991). It is recommended that anyone applying this equation refer to EM 1601 (downloaded from USACE websites) for additional guidance. The EM 1601 equation is

$$d_{30} = y(S_f C_S C_V C_T)\left[\frac{(V_{des})}{\sqrt{k_1(S_g - 1)gy}}\right]^{2.5} \quad (C2.1)$$

where

d_{30} = Particle size for which 30% is finer by weight, ft (m)
y = Local depth of flow above particle, ft (m)
S_f = Safety factor (must be > 1.0)
C_S = Stability coefficient (for blanket thickness = d_{100} or $1.5d_{50}$, whichever is greater, and uniformity ratio d_{85}/d_{15} = 1.7 to 5.2)
= 0.30 for angular rock
= 0.375 for rounded rock
C_V = Velocity distribution coefficient
= 1.0 for straight channels or the inside of bends
= 1.283 − 0.2log(R_c/W) for the outside of bends (1 for R_c/W > 26)
= 1.25 downstream from concrete channels
= 1.25 at the end of dikes
C_T = Blanket thickness coefficient given as a function of the uniformity ratio d_{85}/d_{15}. C_T = 1.0 is recommended because it is based on very limited data.
V_{des} = Characteristic velocity for design, defined as the depth-averaged velocity at a point 20% upslope from the toe of the revetment, ft/s (m/s)
For natural channels, $V_{des} = V_{avg}(1.74 - 0.52\log(R_c/W))$
For trapezoidal channels $V_{des} = V_{avg}(1.71 - 0.78\log(R_c/W))$
V_{avg} = Channel cross-sectional average velocity, ft/s (m/s)
K_1 = Side slope correction factor

$$K_1 = \sqrt{1 - \left(\frac{(\sin\theta - 14^\circ)}{\sin(32^\circ)}\right)^{1.6}}$$

where: θ is the bank angle in degrees

R_c = Centerline radius of curvature of channel bend, ft (m)
W = Width of water surface at upstream end of channel bend, ft (m)
S_g = Specific gravity of riprap (usually taken as 2.65)
g = Acceleration due to gravity, 32.2 ft/s^2 (9.81 m/s^2)

Using the findings of NCHRP Project 24-23, the d_{30} size of the riprap determined by Equation C2.1 is related to the recommended median (d_{50}) size by

$$d_{50} = 1.20d_{30} \quad (C2.2)$$

The flow depth used in Equation C2.1 is defined as the local flow depth above the particle. The flow depth at the toe of slope can be used or the average channel depth. The smaller value produces a slightly larger computed d_{30} size since riprap size is inversely proportional to $y^{0.25}$.

The blanket thickness coefficient (C_T) is 1.0 for standard riprap applications where the thickness is equal to $1.5d_{50}$ or d_{100}, whichever is greater. Because only limited data are available for selecting lower values of C_T when greater thicknesses of riprap are used, a value of 1.0 is reasonable for all applications.

The standard safety factor is 1.1. Greater values should be considered where there is significant potential for ice or impact from large debris, freeze-thaw that would significantly decrease particle size, or large uncertainty in the design variables, especially velocity.

A limitation to Equation C2.1 is that the longitudinal slope of the channel should not be steeper than 2%. For steeper channels the riprap sizing approach for overtopping flows should be considered and the results compared with Equation C2.1 (see Section 6).

Once a design size 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 larger size class, thereby creating a slightly over-designed structure, but economically a less expensive one.

2.2 Layout

Based on information derived primarily from HEC-23 (Lagasse et al., 2001) for revetment riprap layout, the following guidelines were developed.

Revetment riprap on channel banks should be installed to a thickness of the largest allowable stone size d_{100}, or 1.5 times the d_{50} stone size, whichever is greater. When placement must occur under water, the thickness should be increased by 50%.

A filter layer is typically required for revetment riprap. It should be extended fully beneath the entire area to be riprapped. 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 inches, whichever is greater. As with riprap, the filter layer thickness should be increased by 50% when it is being placed under water.

Revetment riprap should be toed down below the toe of the bank slope to a depth at least as great as the depth of anticipated long-term bed degradation plus toe scour. Installations in the vicinity of bridges must also consider the potential for contraction scour. In river systems where dune bed forms are present during flood flows, the depth of the trough below the ambient bed elevation should be estimated using the methods of van Rijn (1984) and Karim (1999). 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 suggests that the maximum depth of the bed form trough below ambient bed elevation will not exceed 0.2 times the depth of flow.

Recommended freeboard allowance calls for the riprap to be placed on the bank to an elevation at least 2.0 feet greater than the design high water level. Upstream and downstream termi-

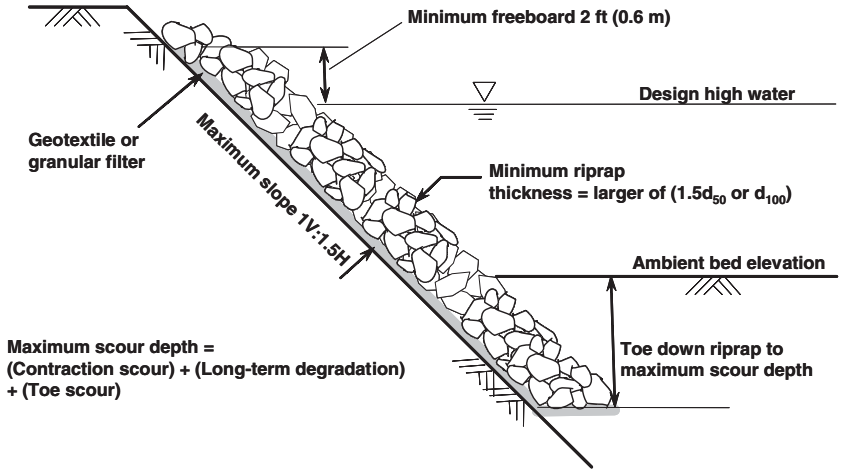


Figure C2.1. Revetment riprap with buried toe.

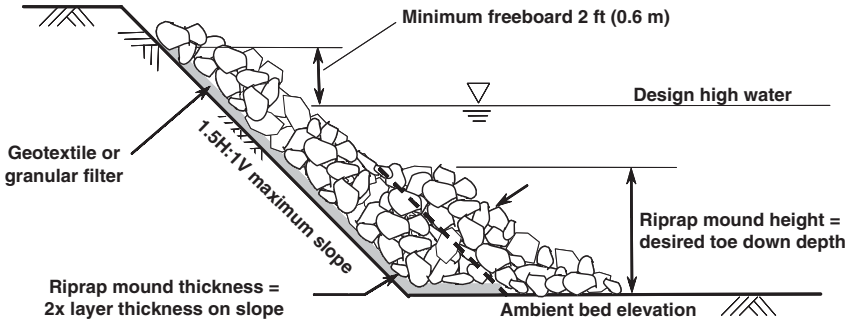


Figure C2.2. Revetment riprap with mounded toe.

nations should utilize a key trench that is dimensioned in relation to the d_{50} size of the riprap. Where the design water level is near or above the top of bank, the riprap should be carried to the top of the bank. Figures C2.1, C2.2 and C2.3 are schematic diagrams that summarize these recommendations.

If toe down cannot be placed below the anticipated contraction scour and degradation depth (Figure C2.1), a mounded toe approach (Figure C2.2) is suggested. Typical details (Figure C2.3)

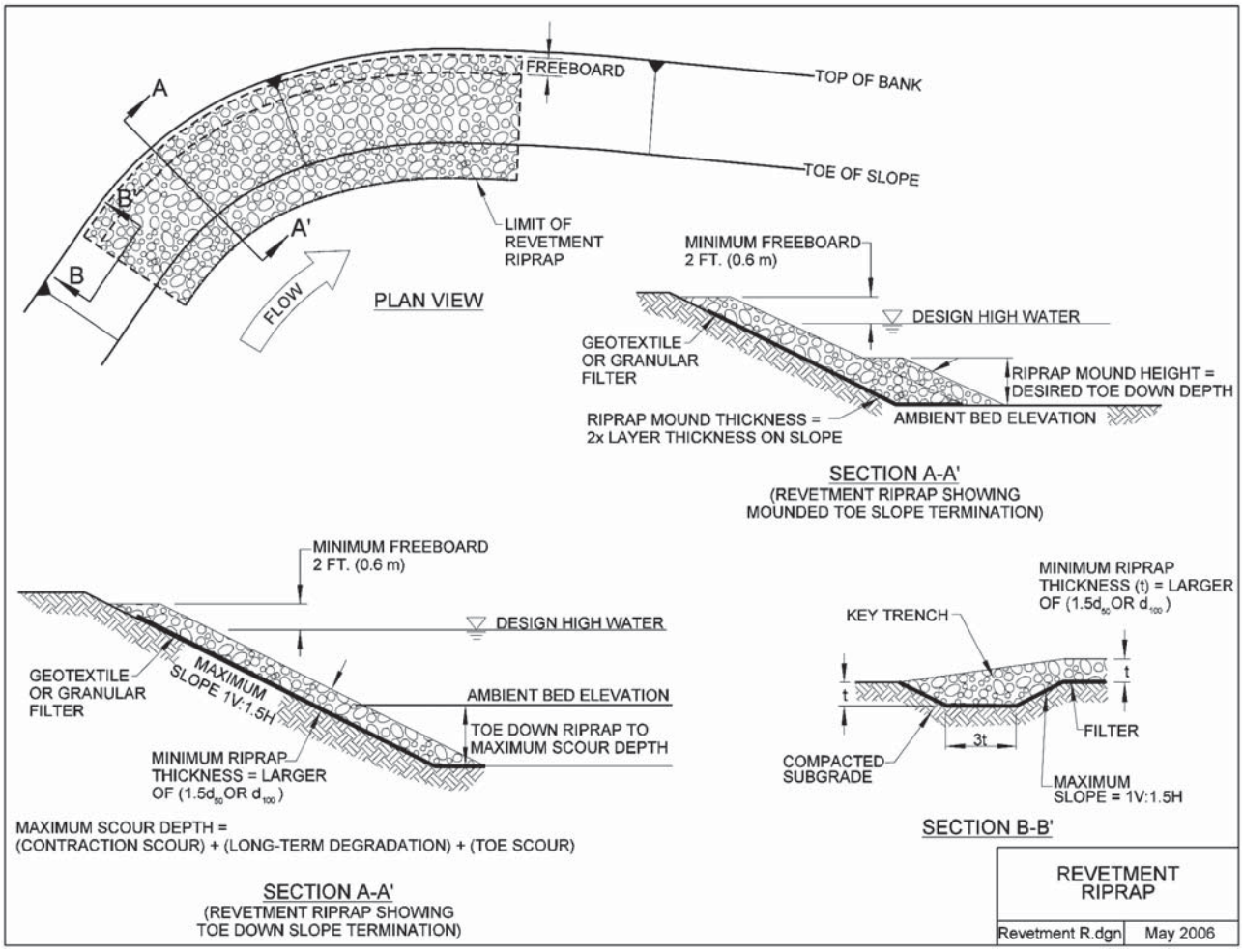


Figure C2.3. Revetment riprap details.

are available in computer-aided design (CAD) formats from the NCHRP Project 24-23 description on the TRB website (http://www4.trb.org/trb/crp.nsf/All+Projects/NCHRP+24-23).

2.3 Example Application

Riprap is to be designed for a 100-ft (30.5-m) wide natural channel on a bend that has a centerline radius (R_c) of 500 ft (152.4 m). The radius of curvature divided by width (R_c/W) is 5.0. The revetment will have a 2H:1V side slope (26.6°) and the rounded riprap has a specific gravity of 2.54. A factor of safety (S_f) of 1.2 is desired. Toe scour on the outside of the bend has been determined to be 2.5 ft during the design event.

The data in Table C2.1 were obtained from hydraulic modeling of the design event.

Table C2.1. Data for example application.

Variable	English Units		SI Units	
	Units	Value	Units	Value
Average channel velocity	ft/s	7.2	m/s	2.19
Flow depth at bank toe	ft	11.4	m	3.47

Step 1: Compute the side slope correction factor (or select from graph on Plate B-39 of EM 1601):

$$K_1 = \sqrt{1-\left(\frac{\sin(\theta-14^\circ)}{\sin(32^\circ)}\right)^{1.6}} = \sqrt{1-\left(\frac{\sin(26.6^\circ-14^\circ)}{\sin(32^\circ)}\right)^{1.6}} = 0.87$$

Step 2: Select the appropriate stability coefficient for rounded riprap: $C_S = 0.36$

Step 3: Compute the vertical velocity factor (C_V) for $R_c/W = 5.0$:

$$C_V = 1.283 - 0.2\log(R_c/W) = 1.283 - 0.2\log(5.0) = 1.14$$

Step 4: Compute local velocity on the side slope (V_{ss}) for a natural channel with Rc/W = 5.0:

$$V_{ss} = V_{avg}[1.74 - 0.52\log(R_c/W)] = 7.2[1.74 - 0.52\log(5.0)]$$
$$= 9.9 \text{ ft/s } (3.01\text{m/s})$$

Step 5: Compute the d_{30} size using Equation C2.1:

$$d_{30} = S_f C_S C_V y\left[\frac{V_{ss}}{\sqrt{(Sg-1)K_1 gy}}\right]^{2.5}$$
$$= 1.2(0.36)(1.14)(11.4)\left[\frac{9.9}{\sqrt{(2.54-1)(0.87)(32.2)(11.4)}}\right]^{2.5} = 0.75 \text{ ft } (0.23\text{m})$$

Step 6: Compute the d_{50} size for a target gradation of $d_{85}/d_{15} = 2.0$:

$d_{50} = 1.2d_{30} = 1.2(0.75) = 0.90$ ft = 10.8 inches (0.28 m)

Step 7: Select Class III riprap from Table C8.1: d_{50} = 12 in (0.3 m)

Step 8: Determine the depth of riprap embedment below the streambed at the toe of the bank slope:

Since toe scour is expected to be 2.5 ft, the 2H:1V slope should be extended below the ambient bed level 5 ft horizontally out from the toe to accommodate this scour. Alternatively, a mounded riprap toe 2.5 ft high could be established at the base of the slope and allowed to self-launch when toe scour occurs.

3 Riprap at Bridge Piers

3.1 Sizing the Riprap

To determine the required size of stone for riprap at bridge piers, NCHRP Project 24-23 recommends using the rearranged Isbash equation from FHWA's HEC-23 (Lagasse et al., 2001) to solve for the median stone diameter:

$$d_{50} = \frac{0.692\left(V_{des}\right)^2}{\left(S_g - 1\right)2g} \qquad (C3.1)$$

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/s^2 (9.81 m/s^2)

It is important that the velocity used in Equation C3.1 is representative of conditions in the immediate vicinity of the bridge pier including the constriction caused by the bridge. As recommended in HEC-23, if the cross-section or channel average velocity, V_{avg}, is used, then it must be multiplied by factors that are a function of the shape of the pier and its location in the channel:

$$V_{des} = K_1 K_2 V_{avg} \qquad (C3.2)$$

If a local velocity is available from stream tube or flow distribution output of a one-dimensional (1-D) model or directly from a two-dimensional (2-D) model, then only the pier shape coefficient should be used. The maximum velocity is often used since the channel could shift and the highest velocity could impact any pier.

$$V_{des} = K_1 V_{local} \qquad (C3.3)$$

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 or 1.7 for square-faced piers
K_2 = Velocity adjustment factor for location in the channel (ranges from 0.9 for a pier near the bank in a straight reach to 1.7 for a pier located in the main current of flow around a sharp bend)
V_{avg} = Channel average velocity at the bridge, ft/s (m/s)
V_{local} = Local velocity in the vicinity of a pier, ft/s (m/s)

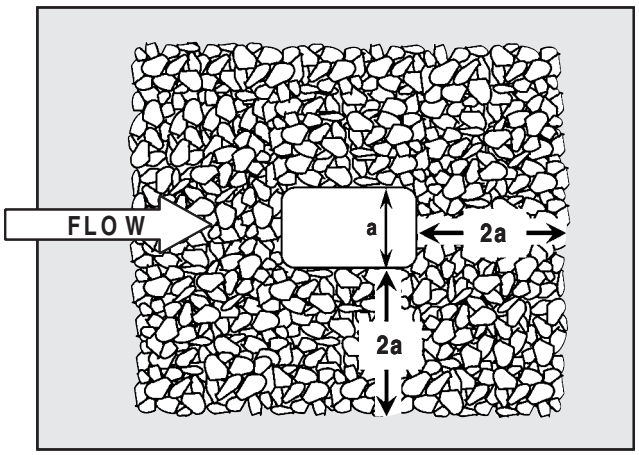
Once a design size is established, a standard gradation class can be selected, if design criteria and economic considerations permit. Using standard sizes the appropriate gradation can be achieved by selecting the next larger size class, thereby creating a slightly over-designed structure, but economically a less expensive one.

3.2 Layout

Based on information derived primarily from NCHRP Project 24-07(2) (Lagasse et al., 2006b) 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.

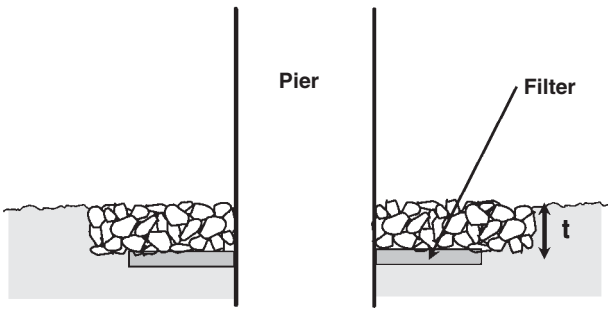
Riprap should be placed in a pre-excavated hole around the pier so that the top of the riprap layer is level with the ambient channel bed elevation. Placing the top of the riprap flush with the bed is ideal for inspection purposes, and does not create any added obstruction to the flow. Mounding riprap 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 bed forms are present during flood flows, the depth of the trough below the ambient bed elevation should be estimated using the methods of van Rijn (1984) and Karim (1999). 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 suggests that the maximum depth of the bed form



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 C3.1. Riprap layout diagram for pier scour protection.

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 C3.1. Typical details are available in CAD formats from the NCHRP Project 24-23 description on the TRB website (http://www4.trb.org/trb/crp.nsf/All+Projects/NCHRP+24-23).

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 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, 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 C3.2.

NOTE: In cases where dune-type bed forms may be present, it is strongly recommended that only geotextiles be considered for use as a filter material.

3.3 Example Application

Riprap is to be sized for an existing 2-ft (0.61-m) square pier. The maximum velocity in the channel is 6.6 ft/s (2.01 m/s) and as a result of channel shift this velocity could occur at the pier. The riprap specific gravity is 2.5. The computed contraction scour is 4.5 ft (1.37 m).

Step 1: Select the appropriate shape coefficient (K) = 1.7.

Step 2: Determine d_{50} from equation C3.1:

$$d_{50} = \frac{0.692(KV)^2}{(S_s - 1)2g} = \frac{0.692(1.7 \times 6.6)^2}{(2.5 - 1)2 \times 32.2} = 0.90\ \text{ft} \times 12\frac{\text{in}}{\text{ft}} = 10.8\ \text{in}\ (0.27\text{m})$$

Step 3: Select Class III riprap from Table C8.1: d_{50} = 12 in (0.3 m)

Step 4: Determine the depth of riprap below the streambed at the pier:

The depth of riprap is the greater of $3d_{50}$ or the contraction scour depth. Therefore, the burial depth must be increased to 4.5 ft (1.37 m).

Step 5: Determine the riprap extent:

The recommended extent is at least 2 times the pier width. Therefore, the minimum riprap extent is 4 ft (1.22 m) from each face of the pier. Given the deep contraction scour, 4 ft is not a sufficient extent to keep the riprap from launching away from the pier. An extent of 9 ft (1.8 m) would provide adequate extent for this depth of contraction scour assuming the riprap launches at a 2H:1V slope under water.

4 Riprap at Bridge Abutments and Guide Banks

4.1 Sizing the Riprap

In HEC-23 (Lagasse et al., 2001) FHWA recommends an abutment riprap design approach based on the FHWA studies by Pagán-Ortiz (1990) and Atayee (1993). **The riprap sizing procedure**

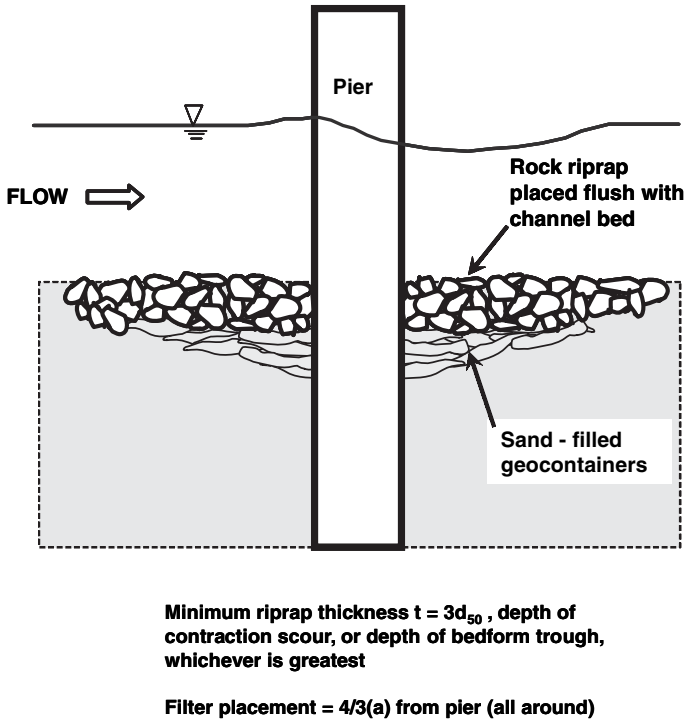


Figure C3.2. Schematic diagram of sand-filled geocontainers beneath pier riprap.

requires the calculation of the flow velocity at the abutment or guide bank (characteristic average velocity) using the procedure discussed later in this section.

For Froude numbers $(V/(gy)^{1/2})$ less than or equal to 0.80, the recommended design equation for sizing rock riprap for spill-through and vertical wall abutments is in the form of the Isbash relationship:

$$\frac{d_{50}}{y} = \frac{K}{(S_g - 1)}\left[\frac{V^2}{gy}\right] \qquad (C4.1)$$

where d_{50} = Median stone diameter, ft (m)
V = Characteristic average velocity in the contracted section, ft/s (m/s) (explained below)
S_g = Specific gravity of rock riprap (usually taken as 2.65)
g = Acceleration due to gravity 32.2 ft/sec^2 (9.81 m/sec^2)
y = Depth of flow in the contracted bridge opening, ft (m)
K = 0.89 for a spill-through abutment
= 1.02 for a vertical wall abutment

For Froude numbers greater than 0.80, the recommended equation is

$$\frac{d_{50}}{y} = \frac{K}{(S_g - 1)}\left[\frac{V^2}{gy}\right]^{0.14} \qquad (C4.2)$$

where K = 0.61 for spill-through abutments
= 0.69 for vertical wall abutments

In both equations, the coefficient K is a velocity multiplier to account for the apparent local acceleration of flow at the point of rock riprap failure. Both of these equations are envelope relationships that were developed to overpredict 90% of the laboratory data.

The recommended procedure for selecting the characteristic average velocity is as follows:

- Determine the set-back ratio (SBR) of each abutment. SBR is the ratio of the set-back length to channel flow depth. The set-back length is the distance from the near edge of the main channel to the toe of abutment.

  SBR equals set-back length/average channel flow depth. **In each of the calculations of the characteristic average velocity the continuity equation (V = Q/A) is used. The discharge (Q) is always taken from the upstream, unencroached (approach) cross section and the area (A) is always taken at the bridge.**
  - If SBR is less than 5 for both abutments (Figure C4.1), compute a characteristic average velocity, Q/A, based on the entire contracted area through the bridge opening. This includes the total upstream flow, exclusive of that which overtops the roadway.
  - If SBR is greater than 5 for an abutment (Figure C4.2), compute a characteristic average velocity, Q/A, for the respective overbank flow only. Assume that the entire respective overbank flow stays in the overbank section through the bridge opening.

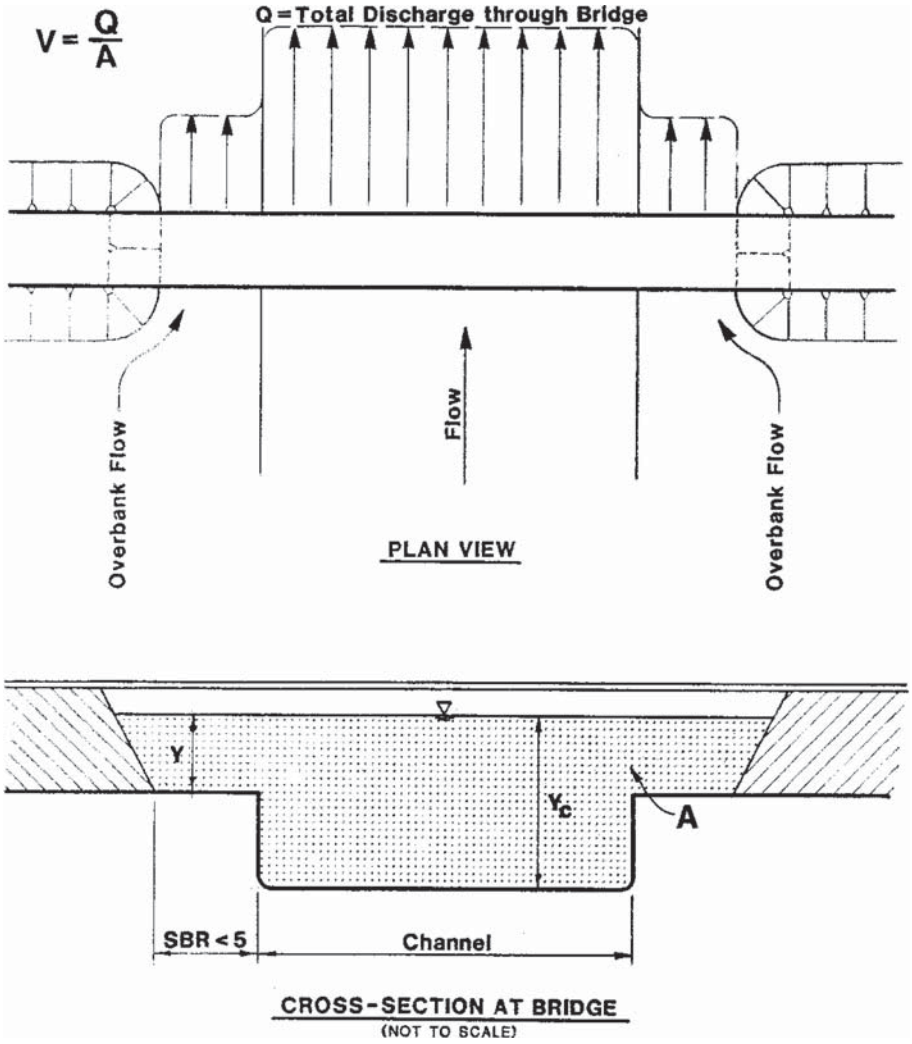


Figure C4.1. Characteristic average velocity for SBR < 5.

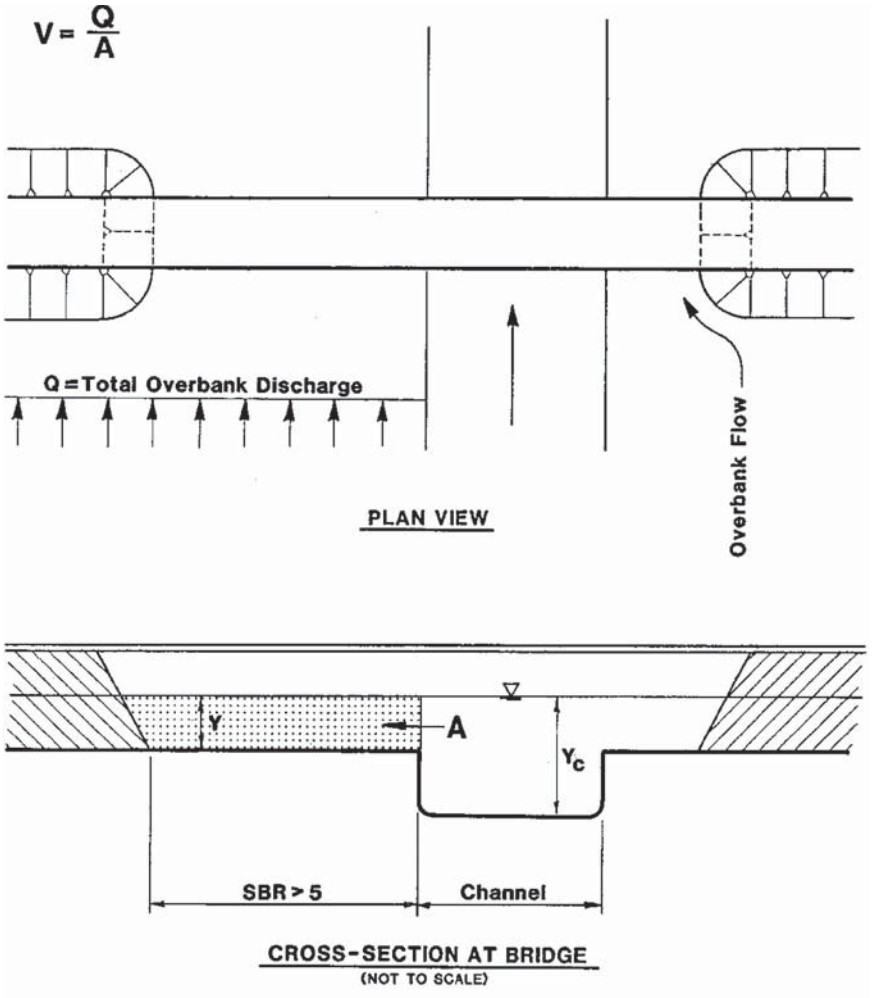


Figure C4.2. Characteristic average velocity for SBR > 5.

  - If SBR for an abutment is less than 5 and SBR for the other abutment at the same site is more than 5 (Figure C4.3), a combination of the two methods must be used. For the abutment with SBR greater than 5, use the method described above. The characteristic average velocity for the abutment with SBR less than 5 should be based on the remaining flow area limited by the boundary of that abutment and an imaginary wall located on the opposite channel bank. The remaining discharge is bounded by this area.
  - When the characteristic average velocity is calculated for SBR greater than 5, the result should be compared with the **maximum** channel velocity in the bridge opening and the lesser of the two velocities should be used.
- Compute the rock riprap size from Equation C4.1 or C4.2, based on the Froude number limitation for these equations.
- For sizing **guide bank riprap**, compute the characteristic average velocity as described above for an abutment, but use 85% of this velocity in either Equation C4.1 or C4.2 depending on the Froude number. Use the K factor for spill-through abutments.

Once a design size 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 larger size class, thereby creating a slightly over-designed structure, but economically a less expensive one.

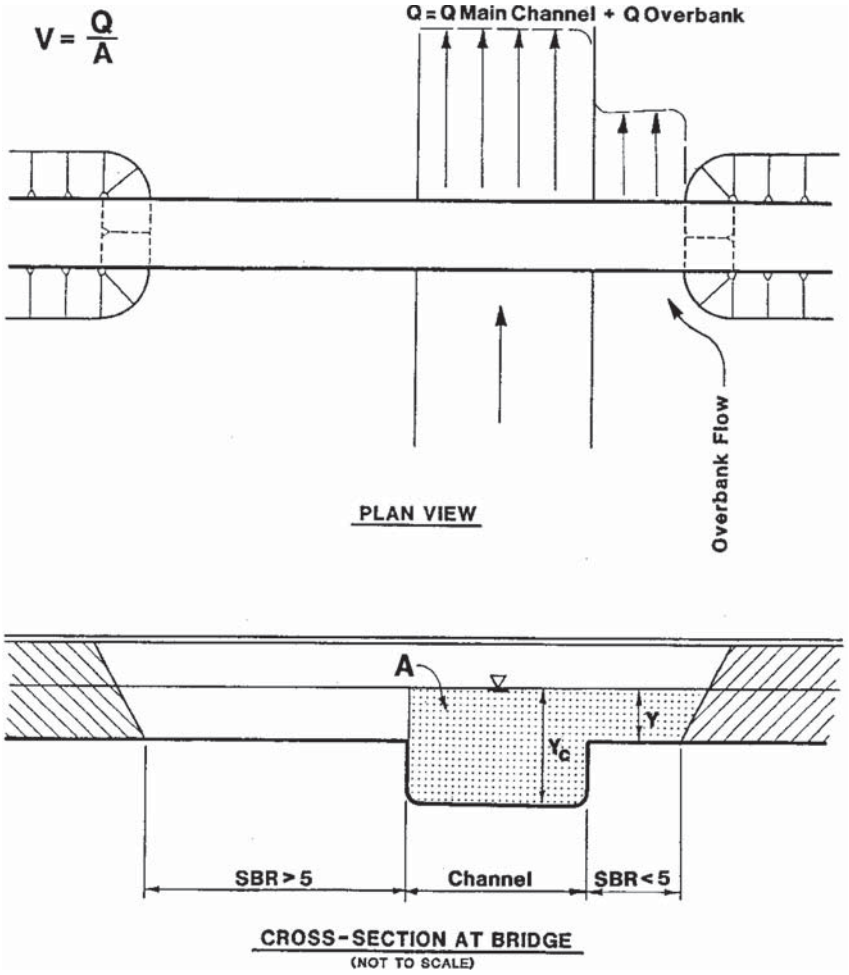


Figure C4.3. Characteristic average velocity for SBR > 5 and SBR < 5.

4.2 Layout

Abutment Riprap

FHWA HEC-23 (Lagasse et al., 2001) gives the extent of rock riprap and construction/installation guidelines at abutments as follows.

The apron should extend from the toe of the abutment into the bridge waterway a distance equal to twice the flow depth in the overbank area near the embankment, but need not exceed 25 ft (7.5 m). There may be cases where an apron extent of twice the flow depth is not adequate (Melville et al., 2006). Melville's findings are based on data collected for NCHRP 24-18A (final report in process). Therefore, the engineer should consider the need for a greater apron extent. The downstream coverage should extend back from the abutment 2 flow depths or 25 ft (7.5 m), whichever is larger, to protect the approach embankment (Figure C4.4).

Spill-through abutment slopes should be protected with the rock riprap size computed from Equation C4.1 or C4.2 to an elevation 2 ft (0.6 m) above expected high water elevation for the design flood. Rock riprap thickness should not be less than the larger of either 1.5 times d_{50} or d_{100}. Figure C4.5 illustrates the recommendation that the top surface of the apron should be flush with the existing grade of the floodplain. This is recommended because the layer thick-

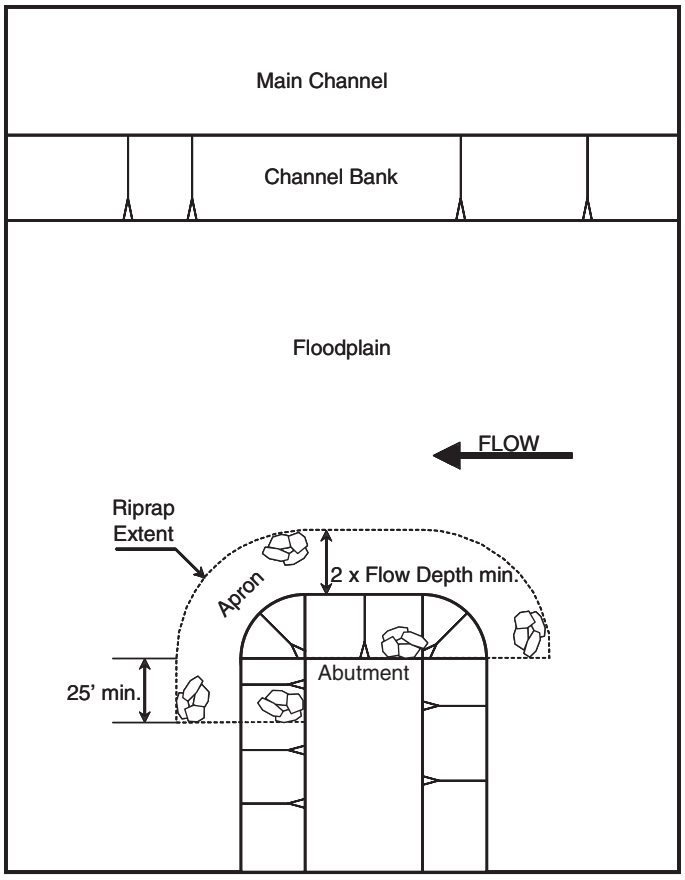


Figure C4.4. Plan view of the extent of rock riprap apron.

ness of the riprap ($1.5d_{50}$ or d_{100}) could block a significant portion of the floodplain flow depth (reducing bridge conveyance) and could generate significant scour around the apron. The rock riprap thickness should be increased by 50% when it is placed under water to provide for the uncertainties associated with this type of placement. The apron thickness may also be increased to protect the edge of the apron from contraction scour, long-term degradation and/or channel migration. Figure C4.6 illustrates a riprap apron at a vertical wall abutment. Typical details are available in CAD formats from the NCHRP Project 24-23 description on the TRB website (http://www4.trb.org/trb/crp.nsf/All+Projects/NCHRP+24-23).

It is not desirable to construct an abutment that encroaches into the main channel. If abutment protection is required at a new or existing bridge that encroaches into the main channel, then riprap toe down or a riprap key should be considered. 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 van Rijn (1984) and Karim (1999). 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 suggests that the maximum depth of the bed form trough below ambient bed elevation will not exceed 0.2 times the depth of flow.

NOTE: In cases where the abutment extends into the main channel and dune-type bed forms may be present, it is strongly recommended that only a geotextile filter be considered for the riprap protection.

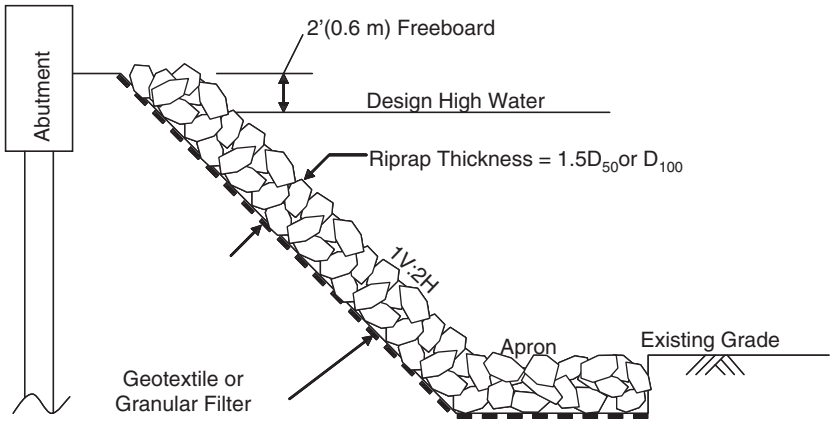


Figure C4.5. Typical cross section for abutment riprap.

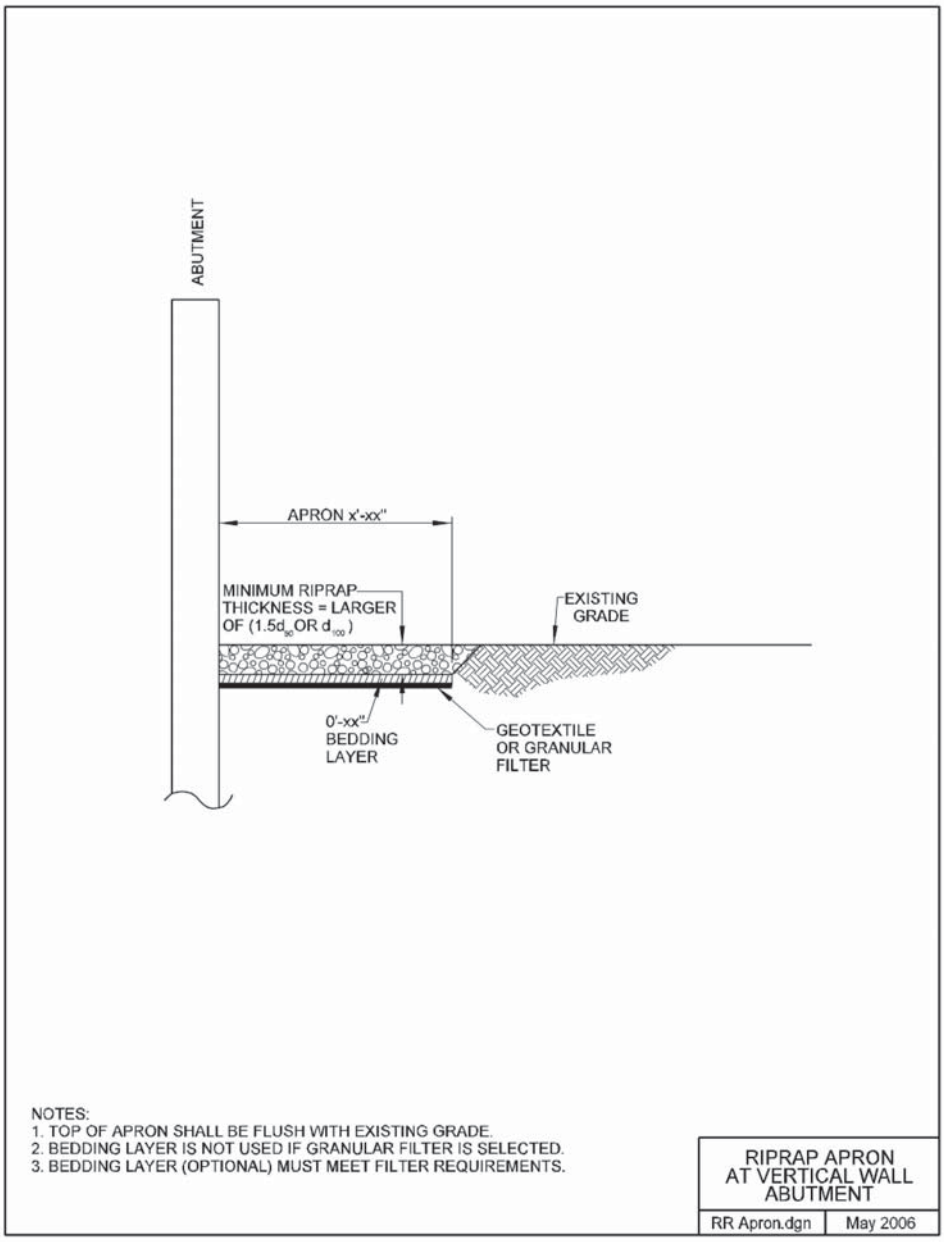


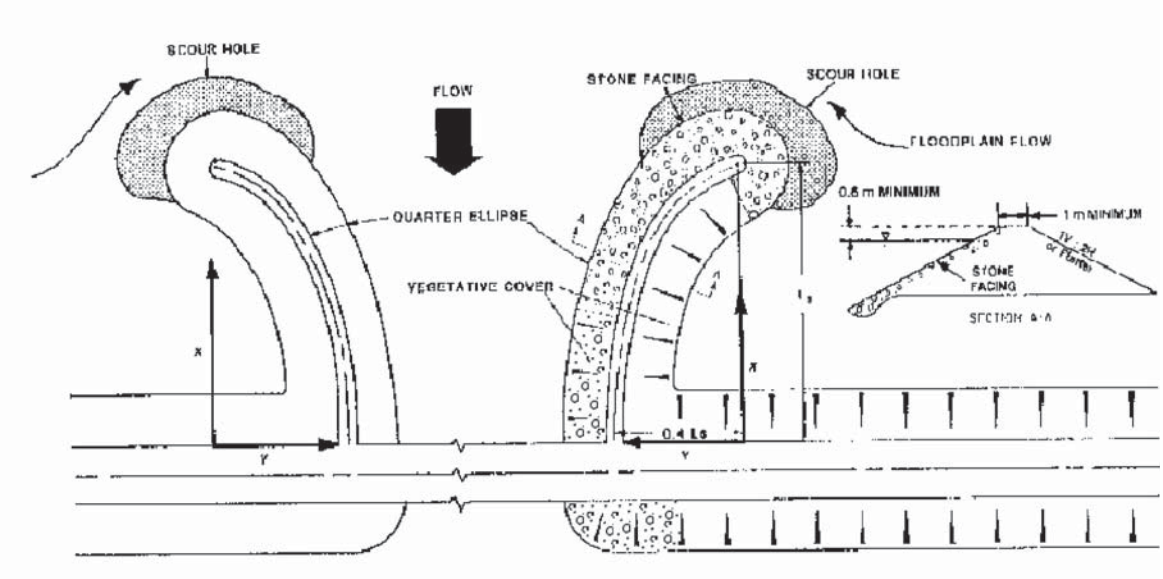
Figure C4.6. Riprap apron at vertical wall abutment.

Guide Banks

FHWA HEC-23 (Lagasse et al., 2001) provides information on determining guide bank length, shape and orientation. Figure C4.7 shows a typical guide bank layout.

Riprap should extend from the end of the short radius curve on the back side of the upstream end of the guide bank all the way around the entire front face of the guide bank through the bridge opening and around the downstream embankment at least 25 ft (7.5 m). If the downstream expansion of flow is too abrupt and erodes the embankment, a shorter guide bank (also called a heel) that is usually 50 ft (15 m) or shorter can be used.

The riprap should extend below the bed elevation to the maximum scour depth (contraction scour plus long-term degradation) and up the face to 2 ft (0.6 m) above the design high water (Figure C4.8). Additional riprap should be placed around the upstream end of the guide bank to protect against the scour that is likely to occur there. Based on the designer's judgment, a riprap key (see Figure C4.9), similar to the mounded riprap toe used for revetments



Source: modified from Bradley (1978)

Figure C4.7. Typical guide bank.

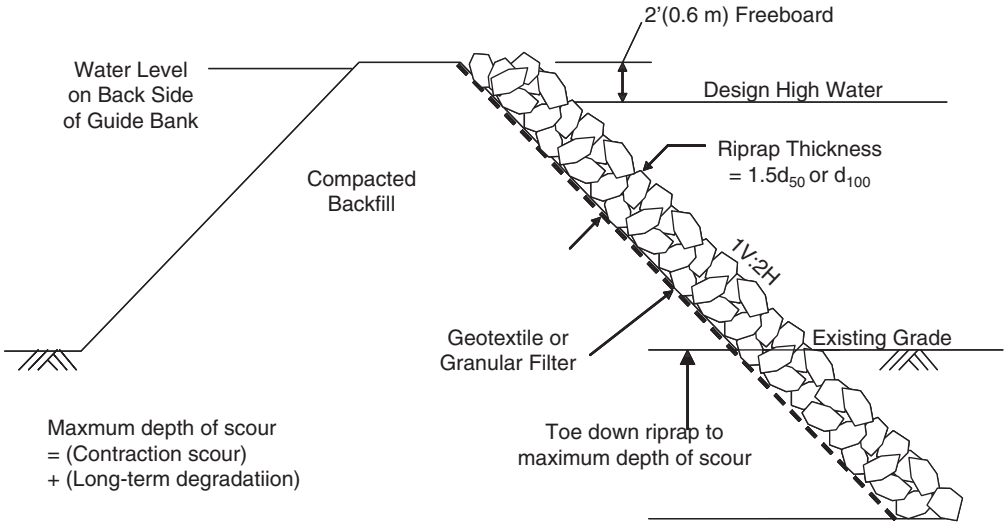


Figure C4.8. Typical cross section through guide bank.

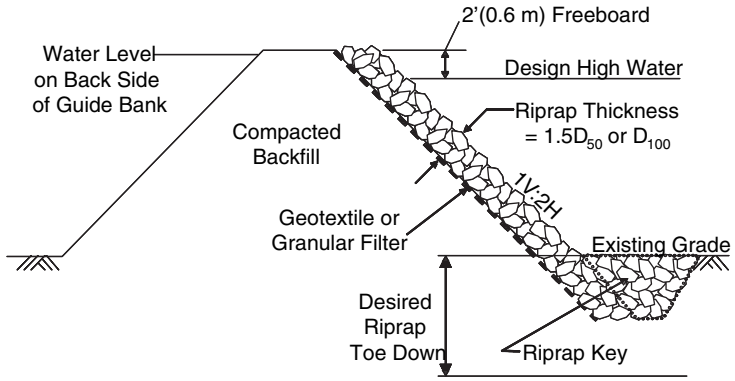


Figure C4.9. Riprap key alternative to toe down.

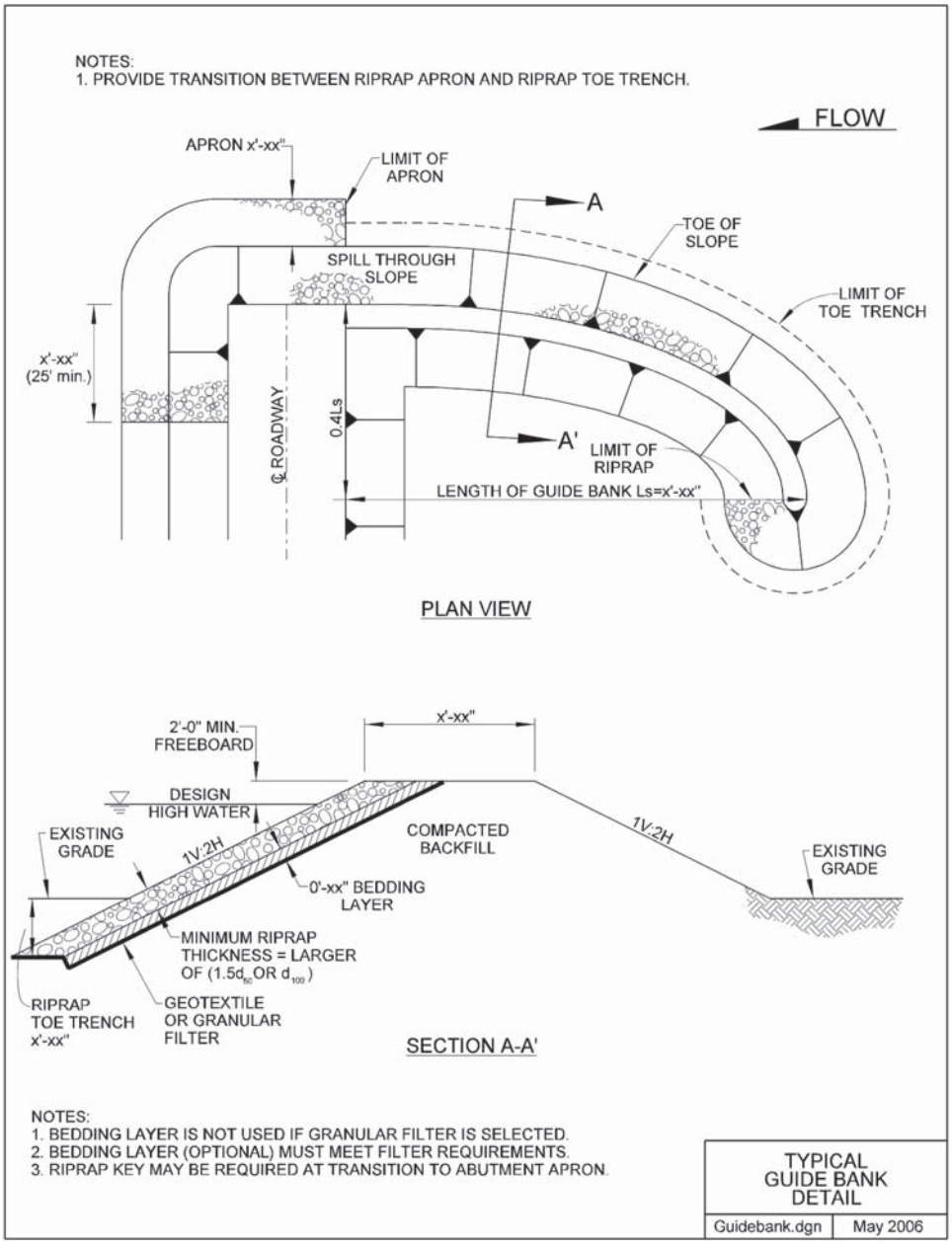


Figure C4.10. Riprap details at guide bank.

shown in Section 2, can be used in lieu of a riprap toe down in order to avoid excessive excavation. The top of the riprap key should be at or below existing grade and the volume should be 1.5 times the volume required to launch down the 2H:1V slope to the desired toe down level at the required layer thickness. The designer should provide a transition from the guide bank riprap toe to the abutment riprap apron. The riprap thickness should not be less than the larger of either 1.5 times d_{50} or d_{100}. The riprap thickness should be increased by 50% when it is placed under water to provide for the uncertainties associated with this type of placement.

The guide bank should also be checked for overtopping from the water ponded on the back side. The water surface on the back side can be several feet higher than the water surface in the bridge opening and can be estimated as the energy grade elevation at the upstream end of the guide bank. Figure C4.10 provides additional details for riprap at guide banks. Typical details are available in CAD formats from the NCHRP Project 24-23 description on the TRB website (http://www4.trb.org/trb/crp.nsf/All+Projects/NCHRP+24-23).

4.3 Example Application

Riprap is to be sized for a spill-through abutment located on the floodplain of a new bridge. The abutment toe is set back from the channel 20 ft (6.1 m). The riprap specific gravity is 2.65. The data in Table C4.1 were obtained from hydraulic modeling of the design event. Also the riprap size should be computed if a guide bank is designed for this abutment.

Table C4.1. Data for example abutment and guide bank applications.

Variable	English Units		SI Units	
	Units	Value	Units	Value
Main channel average flow depth	ft	8.3	m	2.53
Flow depth at toe of abutment	ft	2.8	m	0.85
Total discharge	ft^3/s	4000	m^3/s	113.3
Overbank discharge	ft^3/s	400	m^3/s	11.3
Total bridge area	ft^2	520	m^2	48.3
Set-back area	ft^2	56	m^2	5.2
Bridge average velocity	ft/s	7.7	m/s	2.35
Maximum channel velocity	ft/s	9.1	m/s	2.77

Abutment Riprap Size Computation

Step 1: Determine the SBR (set-back distance divided by the average channel flow depth):

SBR = 20/8.3 = 2.4

Step 2: Obtain characteristic velocity: Since the SBR is less than 5 the velocity at the abutment is estimated as the average flow velocity in the bridge opening.

V = 4000/520 = 7.7 ft/s (2.35 m/s)

(Note: If the SBR was greater than 5, the velocity would be estimated at the abutment toe by dividing the upstream overbank discharge by the set-back area.)

Step 3: Compute the Froude number at the abutment toe:

$$\mathrm{Fr} = \frac{V}{\sqrt{gy}} = \frac{7.7}{\sqrt{32.2(2.8)}} = 0.81$$

Step 4: Calculate the required riprap size:

Select the appropriate equation and abutment shape coefficient from Section 4.1 of this design guide. For Froude numbers greater than 0.80 use Equation C4.2 and K = 0.61 for spill-through abutments.

$$d_{50} = \frac{K}{(S_g - 1)}\left[\frac{V^2}{gy}\right]^{0.14} = \frac{0.61(2.8)}{(2.65-1)}\left[\frac{(7.7)^2}{32.2(2.8)}\right]^{0.14}$$

Step 5: Select Class III riprap from Table C8.1. Recognizing that the Class III gradation could potentially allow stone with a d_{50} as small as 11.5 in, engineering judgment suggests that Class III riprap will perform satisfactorily with a target d_{50} of 12 in (0.30 m).

Step 6: Follow the guidelines presented in Section 4.2 for riprap placement and layout dimensioning.

Guide Bank Riprap Size Computation

Step 1: Estimate velocity at guide bank as 0.85 times the velocity computed for the abutment:

V = 0.85(7.7) = 6.5 ft/s (2.0 m/s)

Step 2: Compute the Froude number at the guide bank toe:

$$Fr = \frac{V}{\sqrt{gy}} = \frac{6.5}{\sqrt{32.2(2.8)}} = 0.68$$

Step 3: Calculate the required riprap size:

Select the appropriate equation and abutment shape coefficient from Section 4.1 of this design guide. For Froude numbers less than 0.80, use Equation C4.1 and K = 0.89 for spill-through abutments.

$$d_{50} = \frac{Ky}{(Sg-1)}\left[\frac{V^2}{gy}\right] = \frac{0.89(2.8)}{(2.65-1)}\left[\frac{(6.5)^2}{32.2(2.8)}\right]$$
$$= 0.71 \text{ ft} = 8.5 \text{ in } (0.22 \text{ m})$$

Step 4: Select Class II riprap from Table C8.1, d_{50} = 9 in.

5 Riprap for Spurs

5.1 Sizing the Riprap

Spurs are used to protect an eroding bank line or control the migration of bends. Spurs can be constructed entirely of riprap, or they can be an earthen core overlain by a layer of riprap armor. Because the crest elevation of spurs is at or below the bank elevation, the design flow should be an in-channel flow. The details involved in the hydraulic design and spacing of a spur field are beyond the scope of this document; guidance for this is provided in HEC-23 (Lagasse et al., 2001). However, sizing riprap for this type of application is provided here.

A riprap spur will produce a locally high velocity around the end of the spur. The EM 1601 equation for revetment riprap can be used to size the armor stone, with a C_V factor of 1.25 recommended for spur applications. The following example illustrates the method.

5.2 Example Application

Spurs are to be designed as an alternative to bank revetment (see revetment riprap design example, Section 2.3). The natural channel is 100 ft (30.5 m) wide on a bend that has a centerline radius (R_c) of 500 ft (152.4 m). The radius of curvature divided by width (R_c/W) is 5.0. The spurs will have 2H:1V side slopes (26.6°) and the rounded riprap has a specific gravity of 2.54. A factor of safety (S_f) of 1.2 is desired. The data in Table C5.1 were obtained from hydraulic modeling of the design event. The hydraulic modeling indicates that the average channel velocity will increase and the depth at the end of the spur will be slightly greater than the depth at the toe of the channel bank.

Table C5.1. Data for example spur application.

Variable	English Units		SI Units	
	Units	Value	Units	Value
Average channel velocity without spurs	ft/s	7.2	m/s	2.19
Flow depth at toe of bank	ft	11.4	m	3.47
Average channel velocity with spurs	ft/s	8.3	m/s	2.53
Flow depth at end of spurs	ft	12.1	m	3.69

Using the revetment riprap equations presented in Section 2.1, the following steps are required to size the riprap for spurs:

Step 1: Compute the side slope correction factor:

$$K_1 = \sqrt{1-\left(\frac{\sin(\theta-14^\circ)}{\sin(32^\circ)}\right)^{1.6}} = \sqrt{1-\left(\frac{\sin(26.6^\circ-14^\circ)}{\sin(32^\circ)}\right)^{1.6}} = 0.87$$

Step 2: Select the appropriate stability coefficient for rounded riprap: $C_S = 0.36$.

Step 3: Select the vertical velocity factor (C_V) for dikes: $C_V = 1.25$.

Step 4: Compute the local velocity on the side slope (V_{ss}) for a natural channel with R_c/W = 5.0:

$$\begin{aligned} V_{SS} &= V_{avg}\left[1.74-0.52\log(R_c/W)\right] \\ &= 8.3\left[1.74-0.52\log(5.0)\right] \\ &= 11.4 \text{ ft/s } (3.48 \text{ m/s}) \end{aligned}$$

Step 5: Compute the d_{30} size using Equation 3.3 in EM 1601 (See Equation C2.1).

$$\begin{aligned} d_{30} &= S_f C_S C_V y\left[\frac{V_{ss}}{\sqrt{(Sg-1)K_1 gy}}\right]^{2.5} \\ &= 1.2(0.36)(1.25)(12.1)\left[\frac{11.4}{\sqrt{(2.54-1)(0.87)(32.2)(12.1)}}\right]^{2.5} = 1.15 \text{ ft } (0.35 \text{ m}) \end{aligned}$$

Step 6: Compute the d_{50} size for a target gradation of $d_{85}/d_{15} = 2.0$.

$d_{50} = 1.2d_{30} = 1.2(1.15) = 1.38$ ft = 16.6 inches (0.42 m)

Step 7: Select Class V riprap from Table C8.1: $d_{50} = 18$ in (0.46 m)

6 Riprap for Overtopping Flow

6.1 Sizing the Riprap

When flow overtops an embankment, spur, or guide bank, locally high velocities occur at the downstream shoulder of the structure. When tailwater is low relative to the crest of the structure, the flow will continue to accelerate along the downstream slope. Guidance for riprap stability under these conditions is provided by Mishra (1998). For slopes steeper than 4H:1V, the method requires that all the flow is contained within the thickness of the riprap layer (interstitial flow). For milder slopes, a portion of the total discharge can be carried over the top of the riprap layer. The three equations necessary to assess the stability of rock riprap in overtopping flow are

$$V_i = 2.48\sqrt{gd_{50}}\left(\frac{S^{0.58}}{C_u^{2.22}}\right) \qquad (C6.1)$$

where V_i = Interstitial velocity, ft/s (m/s)
g = Acceleration due to gravity, 32.2 ft/s^2 (9.81 m/s^2)
d_{50} = Particle size for which 50% is finer by weight, ft (m)
S = Slope of the embankment, ft/ft (m/m)
C_u = Coefficient of uniformity of the riprap, d_{60}/d_{10}

$$d_{50} = \frac{K_u q_f^{0.52}}{C_u^{0.25} S^{0.75}}\left(\frac{\sin\alpha}{(S_g\cos\alpha - 1)(\cos\alpha\tan\phi - \sin\alpha)}\right)^{1.11} \qquad (C6.2)$$

where d_{50} = Particle size for which 50% is finer by weight, ft (m)
K_u = 0.525 for English units
0.55 for SI units
q_f = Unit discharge at failure, ft^3/s/ft (m^3/s/m)
C_u = Coefficient of uniformity of the riprap, d_{60}/d_{10}
S = Slope of the embankment, ft/ft (m/m)
S_g = Specific gravity of the riprap
α = Slope of the embankment, degrees
ϕ = Angle of repose of the riprap, degrees

When the embankment slope is less than 4H:1V (25%), the allowable depth of flow (h) over the riprap is given by

$$h = \frac{0.06(S_g - 1)d_{50}\tan\phi}{0.97(S)} \qquad (C6.3)$$

6.2 Example Application for Slopes Less Than 4H:1V (25%)

Riprap is to be designed to protect a 5H:1V slope from overtopping. The riprap has a specific gravity (S_g) of 2.65, uniformity coefficient (C_u) of 2.1, porosity (η) of 0.45 and an angle of repose (ϕ) of 42°. The data in Table C6.1 are provided for the design.

Table C6.1. Data for example application for slopes less than 4H:1V.

Variable	English Units		SI Units	
	Units	Value	Units	Value
Total discharge (Q)	cfs	2000	m^3/s	56.63
Embankment overtopping length (L)	ft	1000	m	304.8
Unit discharge (q_f)	cfs/ft	2.0	m^2/s	0.186
Weir flow coefficient (C)	$ft^{0.5}/s$	2.84	$m^{0.5}/s$	1.57
Riprap sizing equation coefficient (K_u)	$s^{0.52}/ft^{0.04}$	0.525	$s^{0.52}/m^{0.04}$	0.55
Manning-Strickler coefficient		0.034		0.0414
Slope (S)	ft/ft	0.2	m/m	0.2
Slope angle (α)	degrees	11.3	degrees	11.3

Step 1: Determine the overtopping depth using the broad-crested weir equation:

$Q = CLH^{1.5}$

$H = (Q/CL)^{2/3} = [2000/(2.84 \times 1000)]^{2/3} = 0.79$ ft (0.24 m)

Step 2: Compute the smallest possible median rock size (d_{50}) using Equation C6.2:

$$d_{50} = \frac{k_u q_f^{0.52}}{C_u^{0.25} S^{0.75}} \left(\frac{\sin\alpha}{(S_g \cos\alpha - 1)(\cos\alpha \tan\phi - \sin\alpha)} \right)^{1.11}$$

$$= \frac{0.525(2.0)^{0.52}}{(2.1)^{0.25}(0.2)^{0.75}} \times \left(\frac{\sin(11.3^\circ)}{[2.65\cos(11.3^\circ) - 1][\cos(11.3^\circ)\tan(42^\circ) - \sin(11.3^\circ)]} \right)^{1.11}$$

$$= 0.31 \text{ ft} = 3.7 \text{ inches } (0.094 \text{ m})$$

Step 3: Select Class I riprap from Table C8.1: $d_{50} = 6$ in (0.15 m)

Step 4: Compute the interstitial velocity and the average velocity using Equation C6.1:

$$V_i = 2.48\sqrt{g d_{50}} \frac{S^{0.58}}{C_u^{2.22}} = 2.48\sqrt{32.2(0.5)} \frac{(0.2)^{0.58}}{(2.1)^{2.22}}$$

$$= 0.75 \text{ ft/s } (0.228 \text{ m/s})$$

$$V_{avg} = \eta V_i = 0.45(0.75) = 5.9 \text{ ft/s } (0.103 \text{ m/s})$$

Step 5: Compute the thickness, t, of the riprap layer as if all the flow were through the riprap:

$t = q_f/V_{avg} = 2.0/0.34 = 5.9$ ft (1.81 m)

NOTE: If the average depth is less than $2d_{50}$ then the design is complete with a riprap thickness of $2d_{50}$. If the depth is greater than $2d_{50}$ and the slope is greater than 0.25, go to Step 11. Otherwise, go to Step 6.

5.9 ft > $2d_{50}$ (1.0 ft) and S (0.2) < 0.25, so go to step 6.

Step 6: Find the allowable flow depth over the riprap using Equation C6.3:

$$h = \frac{0.06(S_g - 1)d_{50}\tan\phi}{0.97(S)} = \frac{0.06(2.65 - 1)(0.5)\tan 42^\circ}{0.97(0.2)}$$

$$= 0.23 \text{ ft } (0.069 \text{ m})$$

Step 7: Calculate the Manning roughness coefficient, n

$$n = 0.034(d_{50})^{1/6} = 0.034(0.5)^{1/6} = 0.030$$

Step 8: Calculate the unit discharge, q_1, which can flow over the riprap using Manning's equation:

$$q_1 = \frac{1.489}{n} y^{5/3} S^{1/2} = \frac{1.486}{0.03}(0.23)^{5/3}(0.2)^{1/2}$$
$$= 1.91 \text{ ft}^3/\text{s/ft} = 0.173 \text{ m}^3/\text{s/m}$$

Step 9: Calculate the required interstitial flow, q_2, through the riprap and the flow provided by a riprap thicknesses of $2d_{50}$.

$q_2 = q_f - q_1 = 2.0 - 1.91 = 0.09$ ft³/s/ft (0.013 m³/s/m)

$q = 2d_{50}(V_{avg}) = 2(0.5)(0.34) = 0.34$ ft³/s/ft (0.031 m³/s/m)

Note: If the flow (q) provided by a $2d_{50}$ thickness is greater than or equal to the required flow (q_2), the design is complete with a thickness of $2d_{50}$. If the flow provided by $2d_{50}$ is less than the required flow, proceed to Step 10.

q (0.34 ft³/s/ft) > q_2 (0.09 ft³/s/ft)

Therefore, the design is complete using a thickness of $2d_{50}$ and a riprap d_{50} of 6 in.

Step 10: (not needed for this example). Calculate the flow provided by a $4d_{50}$ thickness of riprap. If the flow provided is greater than the required flow, the design is complete with a thickness of $4d_{50}$ (or an appropriate intermediate thickness). If the flow provided by a $4d_{50}$ thickness is less than the required flow, proceed to Step 11.

Step 11: (not needed for this example). Increase the riprap size to the next gradation class and return to Step 4.

6.3 Example Application for Slopes Greater Than 4H:1V (25%)

Using the same data (provided in Table C6.2 for easy reference) as the previous example, design riprap for a 2H:1V slope (50%). Because the slope is steeper than 4H:1V, the riprap is designed such that all the flow is through the riprap (interstitial flow).

Table C6.2. Data for example application for slopes greater than 4H:1V.

Variable	English Units		SI Units	
	Units	Value	Units	Value
Total discharge (Q)	cfs	2000	m^3/s	56.63
Embankment overtopping length (L)	ft	1000	m	304.8
Unit discharge (q_f)	cfs/ft	2.0	m^2/s	0.186
Weir flow coefficient (C)	$ft^{0.5}/s$	2.84	$m^{0.5}/s$	1.57
Riprap sizing equation coefficient (K_u)	$s^{0.52}/ft^{0.04}$	0.525	$s^{0.52}/m^{0.04}$	0.55
Manning-Strickler coefficient		0.034		0.0414
Slope (S)	ft/ft	0.5	m/m	0.5
Slope angle (α)	degrees	26.6	degrees	26.6

Step 1: Check the overtopping depth using the weir equation:

$Q = CLH^{1.5}$

$H = (Q/CL)^{2/3} = [2000/(2.84 \times 1000)]^{2/3} = 0.79$ ft (0.24 m)

Step 2: Compute the smallest possible median rock size (d_{50}):

$$d_{50} = \frac{k_u q_f^{0.52}}{C_u^{0.25} S^{0.75}} \left(\frac{\sin\alpha}{(S_g \cos\alpha - 1)(\cos\alpha \tan\phi - \sin\alpha)} \right)^{1.11}$$

$$= \frac{0.525(2.0)^{0.52}}{(2.1)^{0.25}(0.5)^{0.75}} \left(\frac{\sin(26.6^\circ)}{[2.65\cos(26.6^\circ) - 1][\cos(26.6^\circ)\tan(42^\circ) - \sin(26.6^\circ)]} \right)^{1.11}$$

$$= 0.96 \text{ ft} = 11.5 \text{ inches } (0.29 \text{ m})$$

Step 3: Select Class III riprap from Table C8.1: d_{50} = 12 in (0.15 m).

Step 4: Compute the interstitial velocity and the average velocity:

$$V_i = 2.48\sqrt{g d_{50}} \frac{S^{0.58}}{C_u^{2.22}} = 2.48\sqrt{32.2(1.0)} \frac{(0.5)^{0.58}}{(2.1)^{2.22}}$$

$$= 1.81 \text{ ft/s } (0.548 \text{ m/s})$$

$$V_{avg} = \eta V_i = 0.45(1.81) = 0.81 \text{ ft/s } (0.247 \text{ m/s})$$

Step 5: Compute the thickness as if all the flow were through the riprap:

$t = q_f/V_{avg} = 2.0/0.81 = 2.5$ ft (0.75 m)

Note: If the average depth is less than $2d_{50}$ then the design is complete with a riprap thickness of $2d_{50}$. If the depth is greater than $2d_{50}$ and the slope is greater than 0.25, go to step 11. Otherwise, go to Step 6 of the previous example.

2.5 ft > $2d_{50}$ (2.0 ft) and S (0.5) > 0.25, so go to Step 11.

Step 11: Increase the riprap size to the next gradation class.

Step 12: Select Class IV riprap with d_{50} of 15 in from Table C8.1 and return to Step 4.

Step 4 (trial 2): Compute the interstitial velocity and the average velocity:

$$V_i = 2.48\sqrt{g d_{50}} \frac{S^{0.58}}{C_u^{2.22}} = 2.48\sqrt{32.2(1.25)} \frac{(0.5)^{0.58}}{(2.1)^{2.22}}$$

$$= 2.03 \text{ ft/s } (0.617 \text{ m/s})$$

$$V_{avg} = \eta V_i = 0.45(2.03) = 0.91 \text{ ft/s } (0.278 \text{ m/s})$$

Step 5 (trial 2): Compute the thickness as if all the flow were through the riprap:

$t = q_f/V_{avg} = 2.0/0.91 = 2.2$ ft (0.67 m)

Note: If the average depth is less than $2d_{50}$ then the design is complete with a riprap thickness of $2d_{50}$. If the depth is greater than $2d_{50}$ and the slope is greater than 0.25, go to Step 11. Otherwise, go to Step 6 of the previous example.

2.2 ft < $2d_{50}$ (2.5 ft), so design is complete with d_{50} = 15 in and a riprap thickness of 2.5 ft. This check ensures that all the flow is contained within the thickness of the riprap layer (interstitial flow).

7 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 the base soil is composed primarily of relatively large particles (coarse sands and gravels), a filter layer may not be necessary.

7.1 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/yd^2 (400 g/m^2). ***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, or AOS), and porosity (for non-woven fabrics) or percent open area (for woven fabrics). 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 riprap filter design. For riprap installations, the permeability of the geotextile should be at least 10 times greater than 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 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 fabric has a much greater capacity because of its three-dimensional (3-D) microstructure. Transmissivity is not particularly relevant to riprap 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. 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.** Force required to puncture a geotextile using a standard penetration apparatus. 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 riprap 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 riprap, these stresses do not represent the actual environment that the geotextile will experience in the long term.

7.2 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:

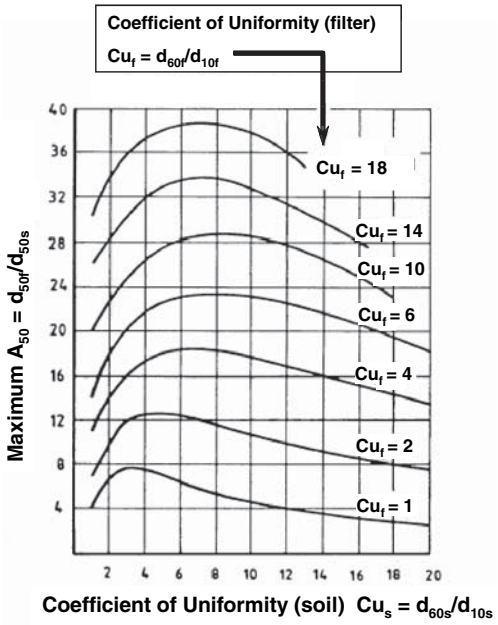
- **Particle size distribution.** As a rule of thumb, the gradation curve of the granular filter material should be approximately parallel to that of the base soil. Parallel gradation curves minimize the migration of particles from the finer material into the coarser material. Heibaum (2004) presents 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 C7.1 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 riprap 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 C7.1. Granular filter design chart according to Cistin and Ziems.

7.3 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

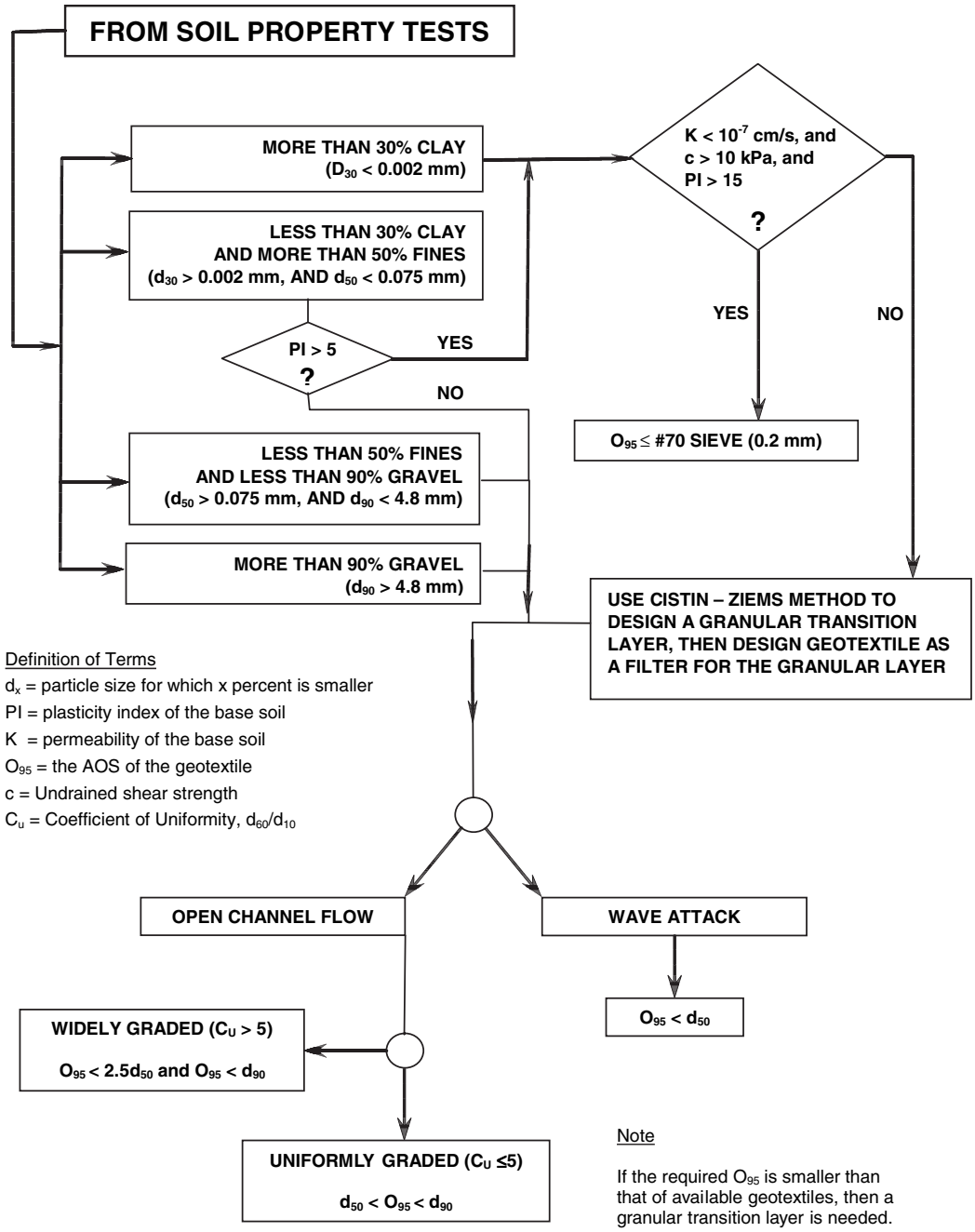
A decision tree is provided as Figure C7.2 to assist in determining the appropriate soil retention criterion for the geotextile. The figure includes guidance when a granular transition layer (i.e., composite filter) is necessary. A composite filter is typically required when the base soil is greater 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:

$K < 1 \times 10^{-7}$ cm/s
PI > 15
c > 10 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



Source: modified from Koerner (1998)

Figure C7.2. Geotextile selection based on soil retention.

greater (Holtz et al., 1995). For riprap applications, it is recommended that the larger of these values (i.e., $K_f/K_s > 10$) be used for designing a filter. If the permeability of the base soil K_s 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. Note that the subscript "s" is used to represent the base (finer) soil, and "f" is used to represent the filter (coarser) layer.

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 C8.4 for recommended values based on AASHTO standard 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).

7.4 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 30% 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 C7.1) 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_{50}max 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 Compatibility with Riprap

Repeat Steps 1 through 4 above, considering that the filter material is now the "finer"soil and the rock riprap is the "coarser" material. If the Cistin–Ziems criterion is not met, then multiple layers of granular filter materials should be considered.

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). Single-layer thicknesses up to 15 in (38 cm) may be warranted where large riprap 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 as recommended in HEC-11 (Brown and Clyde, 1989).

7.5 Example Application

Revetment riprap using gradation Class II is to be placed on a channel bank. The native soil on the channel banks is a silty sand. A locally produced sand is proposed as a granular filter material for the riprap. The grain size distribution of the native soil and candidate filter material are shown in Figure C7.3. Other characteristics of the design are listed in Table C7.1.

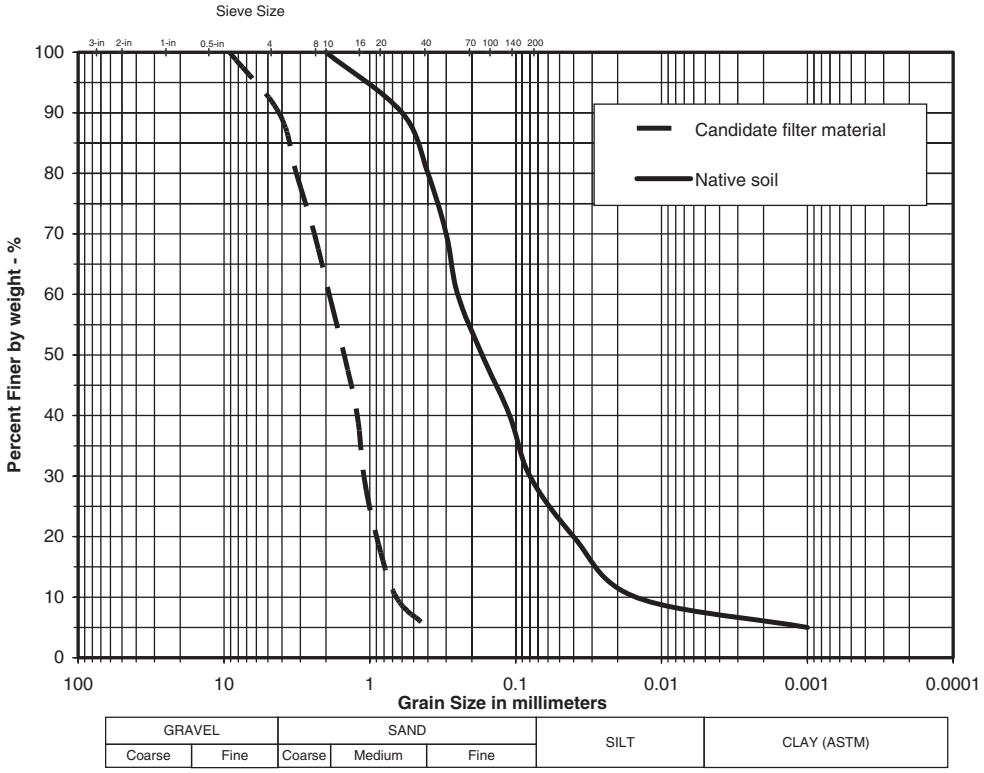


Figure C7.3. Grain size curves for example application.

Table C7.1. Design characteristics for the example application.

Soil Property	Native Soil	Filter	Riprap Class II
Hydraulic conductivity K, cm/s	4.2×10^{-4}	2.3×10^{-2}	n/a
Coefficient of uniformity $C_u = d_{60}/d_{10}$	.25/.015 = 16.6	1.9/.66 = 2.9	2.1
Median diameter d_{50}, mm	0.17	1.5	230 (9 in)
Plasticity index	3.3	(np)	(np)

Step 1: Assess the suitability of the candidate filter material for compatibility with the native soil:

Enter the Cisten–Ziems chart (Figure C7.1) with C_u = 16.6 of the native soil on the x-axis. Chart vertically up to a location corresponding to a C_u of 2.9 for the candidate material. Read a maximum allowable value A_{50} of approximately 12 on the y-axis.

Step 2: Compute the maximum allowable d_{50} of the filter material:

Max. allowable $d_{50f} = A_{50}(d_{50s}) = 12(0.17) = 2.0$ mm
Because the actual d_{50f} of the candidate material is 1.5 mm, this material is suitable as a filter for the native soil based on its particle retention function.

Step 3: Check the permeability ratio: $K_f/K_s = (2.3 \times 10^{-2})/(4.2 \times 10^{-4}) = 55$

Because this ratio is greater than 10, the filter is OK from a permeability standpoint.

Step 4: Assess the suitability of the riprap for compatibility with the candidate filter material:

Enter the Cisten–Ziems chart (Figure C7.1) with C_u = 2.9 of the filter material on the x-axis. Chart vertically up to a location corresponding to a Cu of 2.1 for the riprap. Read a maximum allowable value A50 of approximately 13 on the y-axis.

Step 5: Compute the maximum allowable d_{50} of the riprap:

Max. allowable $d_{50r} = A_{50}(d_{50f}) = 13(1.5) = 19.5$ mm

Because the actual d_{50r} of the riprap is 230 mm, **the filter particles will leach through the voids of the Class II riprap.** Therefore, a second (coarser) filter layer will need to be designed to retain the first filter layer, while simultaneously being retained by the Class II riprap. A coarse, gravelly material must be found and analyzed as a candidate material for the second filter layer.

Because the above example resulted in a two-layer granular filter system, a geotextile option will be explored. Using the same native soil characteristics as the previous example, the following steps are outlined:

Step 1: Knowing the base soil characteristics, enter the flowchart on Figure C7.2 with a soil that is "less than 50% fines and less than 90% gravel."

Step 2: Follow down the decision tree to the "open channel flow" box, and select the "widely graded" branch, because the native soil has a Cu of 16.6, which is greater than 5.

Step 3: Determine the allowable limits on the O_{95} of the geotextile. O_{95} is also known as the AOS:

$O_{95} < 2.5(d_{50})$	so	$O_{95} < 2.5(0.17$ mm) or 0.425 mm
$O_{95} < d_{90}$	so	$O_{95} < 0.6$ mm

The first inequality is more restrictive than the second, so the geotextile must have an AOS that is less than 0.425 mm. This is approximately equivalent to a #40 U.S. standard sieve size.

Step 4: Specify the geotextile, considering that its hydraulic conductivity should be at least 10 times greater than that of the native soil (Table C7.2).

8 Materials

8.1 Riprap Size, Shape, and Gradation

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

Table C7.2. Specifications for geotextile.

Geotextile Property	Non-woven, Needle-punched Fabric	Woven, Monofilament Fabric
Maximum AOS, U.S. standard sieve	# 40	# 40
Minimum hydraulic conductivity, cm/s	4.2×10^{-3}	4.2×10^{-3}
Minimum mass per unit area, oz/yd^2	12	n/a
Minimum open area, percentage	n/a	4.0
Minimum porosity, percentage	30	n/a
Minimum strength properties	Per Table C8.4	Per Table C8.4

and shapes, the required size of stone is often stated in terms of a minimum allowable representative size. For example, the designer may specify a minimum d_{50} or d_{30} for the rock composing the riprap, thus indicating the size for which 50% or 30% (by weight) of the particles are smaller. Stone sizes can also be specified in terms of weight (e.g., W_{50} or W_{30}), 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 C8.1.

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, because the B axis is intermediate between the two extremes of length, A, and thickness, C. A maximum allowable value of 3.0 is recommended:

$$\frac{A}{C} \leq 3.0 \qquad (C8.1)$$

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:

$$S_g = \frac{\gamma_s}{\gamma_w} \qquad (C8.2)$$

Typically, 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.

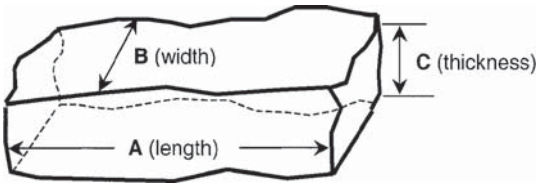


Figure C8.1. Riprap shape described by three axes.

Size and Weight

Based on field studies, the recommended relationship between size and weight is given by:

$$W = 0.85(\gamma_s d^3) \qquad (C8.3)$$

where W = Weight of stone, lb (kg)
γ_s = Density of stone, lb/ft^3 (kg/m^3)
d = Size of intermediate (B) axis, ft (m)

Table C8.1 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., 2006a). 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, d_{85}/d_{15}, is 2.0 and the allowable range is from 1.5 to 2.5.

Based on Equation C8.3, which assumes the volume of the stone is 85% of a cube, Table C8.2 provides the equivalent particle weights for the same 10 classes, using a specific gravity of 2.65 for the particle density.

Table C8.1. Minimum and maximum allowable particle size in inches.

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: Particle size d corresponds to the intermediate (B) axis of the particle.

Table C8.2. Minimum and maximum allowable particle weight in pounds.

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: Weight limits for each class are estimated from particle size by: $W = 0.85(\gamma_s d^3)$ where d corresponds to the intermediate (B) axis of the particle, and particle specific gravity is taken as 2.65.

8.2 Physical Properties and Recommended Tests

Recommended standard test methods relating to material type, characteristics, and testing of riprap and aggregates typically associated with riprap installations (e.g., filter stone and bedding layers) are provided in this section. 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 only break with difficulty, have no earthy odor, no 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 riprap. Table C8.3 summarizes the recommended tests and allowable values for rock and aggregate. Table C8.4 provides the recommended tests and allowable values for geotextiles.

Table C8.3. Recommended tests for riprap 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 / Application > 90 Severe > 80 Moderate > 70 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., 2006a), 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., 2006a) 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 C8.4. Recommended tests 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 7 of this design guide)		Maximum allowable value
ASTM D 4491	Permittivity and Permeability	Per design criteria (Section 7 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).

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APPENDIX D

Guidelines for the Construction, Inspection, and Maintenance of Rock Riprap Installations

1 Introduction, D-2
2 Construction Aspects, D-2
3 Inspection, D-8
4 Maintenance, D-11
5 References, D-19

1 Introduction

When properly designed and used for erosion protection, riprap has an advantage over rigid materials 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 guideline considers construction aspects and recommended inspection and maintenance of riprap installations.

Design of a properly functioning riprap system requires knowledge of river bed and foundation material; flow conditions including velocity, depth, and orientation; riprap characteristics of size, density, durability, and availability; and the type of interface material between the riprap and underlying foundation. At bridges, the size, shape, and skew angle of piers with respect to the flow direction must be known, and the location and type of abutments (spill-through or vertical wall) must be determined. 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.

Section 2 provides a discussion of various construction aspects. Section 3 provides guidance on the inspection of riprap installations and includes recommended coding guidance for inspectors. Section 4 discusses aspects of maintenance, including a description of riprap failure modes.

The guidance provided in this document has been developed primarily from the results of NCHRP Project 24-23 (Lagasse et al., 2006a) and FHWA Hydraulic Engineering Circular No. 23 (HEC-23) (Lagasse et al., 2001). The guidelines should be closely examined and modified, as appropriate, for local design practices; specification tests; specification values; and procedures for materials testing, construction inspection, and periodic maintenance inspection.

2 Construction Aspects

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 different problems such as rocks too large or with unsatisfactory length to width ratios for riprap.

Quarry stones are generally the best source for obtaining large rock specified for riprap. However, not all quarries can produce large stone because of rock formation characteristics or limited volume of the formation. Because quarrying generally uses blasting to fracture the formation into rock suitable for riprap, cracking of the large stones may only become evident after loading, transporting, and dumping at the quarry or after moving 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 on the slope of the bank 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 to maximize 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 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 ER 1180-1-6, 1995, "Construction Quality Management") 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 Appendix C, Guidelines for the Design and Specification of Rock Riprap Installations. 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 revetment. 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 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 basic information and descriptions 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. Onsite 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 the final report for NCHRP Project 24-23 (Lagasse et al., 2006a) 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 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/yd^2 (400 g/m^2). ***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.

2.2.3 Subgrade Soils

When placing 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, backfilled with approved material and compacted prior to placement of the riprap. Unsatisfactory soils may also be defined—such soils 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 placing 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 placing 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 placing the filter and riprap system, the prepared subgrade must be inspected.

2.3.2 Placing the Filter

Whether the filter is composed of one or more layers of granular material or 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 shall be placed directly on the prepared area, in intimate contact with the subgrade. When placing a geotextile, it should be rolled or spread out directly on the prepared area and shall 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, placement of the overlying materials should be conducted 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 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 precut to length have been placed using a roller assembly, with 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 D2.1. The fabric for the geocontainers should be selected in accordance with appropriate filter design criteria, and placed such that they overlap to cover the required area. For more information, see Lagasse et al. (2006a and b).

Placement of Granular Filter. When 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

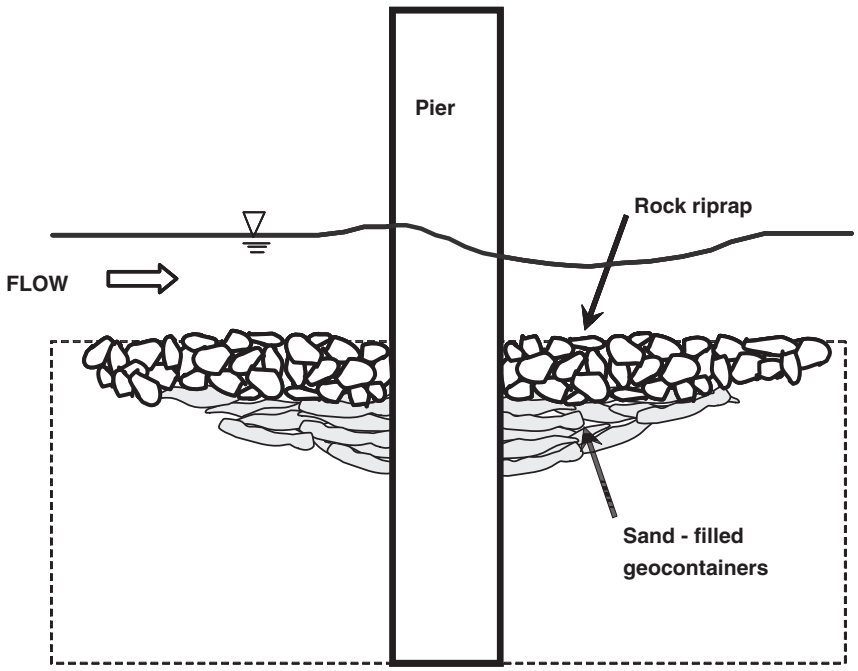


Figure D2.1. Schematic diagram showing the use of sand-filled geocontainers as a filter.

between the filter and the riprap. When placing a granular filter 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 can be found in Appendix C, Guidelines for the Design and Specification of Rock Riprap Installations. 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 being created.

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 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 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 (ROV) can provide some information about the riprap placement under water.

2.3.4 Inspection

The subgrade preparation, geotextile placement and riprap system installation, and overall finished condition including termination trenches shall be inspected before accepting the work. Inspection guidelines for the riprap installation are presented in detail in Section 3 of this document.

2.4 Measurement and Payment

Riprap satisfactorily placed can be paid for based on either volume or weight. When using a weight basis, commercial truck scales capable of printing a weight ticket including time, date, truck number, and weight should be used. When using a volumetric basis, 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 his own expense.

3 Inspection

3.1 Inspection During Construction

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 ft^2 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 Inspection

If the riprap installation is part of channel stability works in the vicinity of a bridge, it is typically inspected during the bi-annual 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.

3.3 Post-Construction/Post-Flood Inspection

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 subgrade 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 inspecting riprap, the following 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?

3.4 Inspection Coding Guide

To guide the inspection of a riprap installation, a coding system is presented in this section. 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.

A single-digit code is used as indicated in Table D3.1 to identify the current status of the rock riprap regarding its condition compared to the design intent, and the immediacy of need for maintenance activities to return it to the design condition.

This guidance covers riprap installations that may be (1) located on stream banks for lateral stream stability purposes, (2) placed against bridge piers or abutments for protection against scour at the structure, (3) placed across the stream to provide vertical grade stabilization, or (4) other applications in riverine environments (e.g., guide banks or spurs).

Table D3.1. Inspection coding guide.

Code	Description
U	Uninspectable: The riprap is uninspectable, because of burial by sediment, debris, or other circumstance. Until the condition of the riprap can be reliably determined, a plan of action should be developed that considers the degree of risk posed by potential failure of the installation.
9	The riprap installation is stable: Riprap stones are angular to subangular with no evidence of deterioration or segregation of sizes; **and** the distribution of stone sizes and overall thickness of riprap layer conform to design specifications; **and** there is no evidence of displacement of individual stones.
8	The riprap installation is stable: Riprap stones are angular to subangular with no evidence of deterioration or segregation of sizes; **and** the distribution of stone sizes and overall thickness of riprap layer conform to design specifications; **and** some displacement of individual stones is evident, but only smaller sized particles significantly smaller than the design d_{50} size have moved.
7	The riprap installation is stable: Evidence of some deterioration of stones due to surficial weathering (e.g., abrasion, freeze-thaw, or wet-dry spalling); **and** stone shape is primarily subangular. *OR* A minor decrease in overall layer thickness is evident, and/or particle displacement noted with displaced particles approaching the design d_{50} size; **and** the geotextile or granular filter has *not* been exposed.
6	The riprap installation has experienced erosion: Individual stones are primarily subrounded in shape due to surficial weathering; **and** the distribution of stone sizes still exhibits a d_{50} particle greater than the minimum allowable d_{50} size. *OR* Minor decrease in overall layer thickness is evident; **and** some particles greater than the design d_{50} size have been displaced; **and** the geotextile or granular filter has *not* been exposed.
5	The riprap installation has experienced erosion: Similar condition as Code 6, except that the geotextile or granular filter has been exposed in local areas or around the periphery of the installation. The inspector should attempt to identify whether stone displacement has occurred because of gravity slump or slide, or by hydraulic forces.
4	The riprap installation has experienced significant erosion: Individual stones are subrounded to rounded in shape due to significant deterioration, **and** the distribution of stone sizes exhibits a d_{50} particle smaller than the minimum allowable d_{50} size. *OR* Significant decrease in overall layer thickness is evident in local areas; **and** some particles greater than the design d_{50} size have been displaced; **and** the geotextile or granular filter has been exposed in local areas.
3	The riprap installation is unstable: The riprap matrix consists primarily of stones smaller than the minimum allowable d_{50} particle size; **and** the overall layer thickness is less than 50% of specification. *OR* A significant portion of the particles greater than the design d_{50} size has been displaced, **and** the geotextile or granular filter has been exposed over more than 20% of the installation area.
2	The riprap installation is unstable: The riprap matrix consists almost entirely of stones smaller than the minimum allowable d_{50} particle size; **and** the overall layer thickness is less than 2 particles thick. *OR* Most of the particles greater than the design d_{50} size has been displaced, **and** the geotextile or granular filter has been exposed over more than 50% of the installation area.
1	The riprap installation is eroded and can no longer serve its function. Immediate maintenance is required: Most of the riprap matrix has been displaced or is missing; **and** native subgrade soil is exposed. *OR* Large patches or voids in the riprap matrix have been opened; **and** stones are no longer in contact with structural elements.
0	The riprap installation is essentially gone and scour is imminent. Immediate maintenance is required: The riprap has deteriorated to the point that it cannot perform its protective function even in minor events.

Particle Size Distribution

Size distribution should be determined by the method of Wolman (1954) using three transects having 100 specimens per transect, where feasible. See also Lagasse et al. (2006a).

Recommended Action

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 nondestructive testing using ground-penetrating radar or seismic methods.

Codes 9, 8, or 7. Continue periodic inspection program at the specified interval.

Codes 6, 5, or 4. 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.

Codes 3 or 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 standard riprap might be considered.

Code 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 may also need to be notified.

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.

The evaluation of any revetment 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 revetment 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.

4.1 Revetment Riprap Failure Modes

Illustration of the most common modes of riprap failure provides 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.

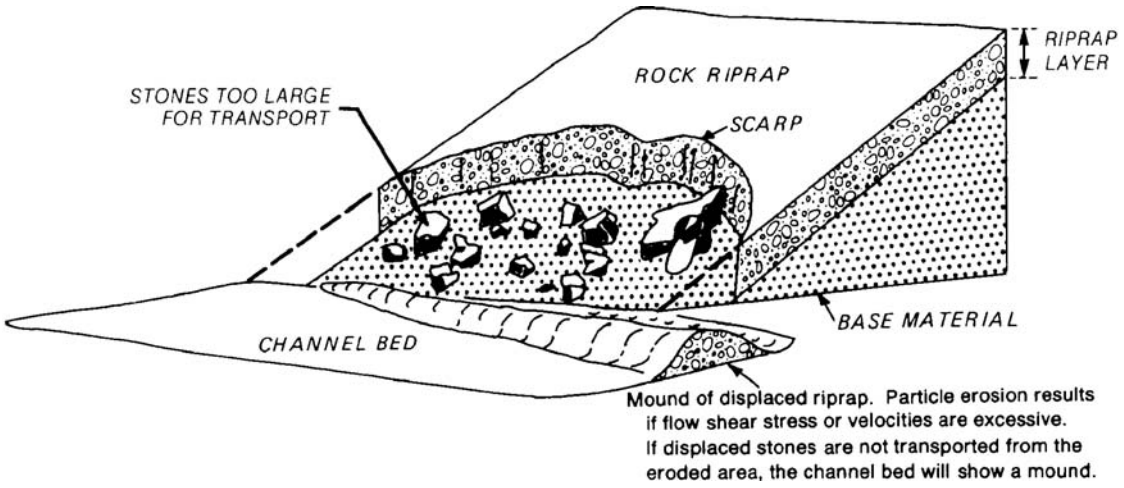
In a preliminary evaluation of various riprap design techniques, Blodgett and McConaughy (1986) concluded that a major shortcoming of all present design techniques is their assumption that failures of revetment riprap are due only to particle erosion. Procedures for the design of riprap protection need to consider all the various modes of failures: (1) particle erosion; (2) translational slide; (3) modified slump; and (4) slump.

Particle Erosion

Particle erosion is the most commonly considered erosion mechanism (Figure D4.1). Particle erosion occurs when individual particles are dislodged by the hydraulic forces generated by the flowing water. Particle erosion can be initiated by abrasion, impingement of flowing water, eddy action/reverse flow, local flow acceleration, freeze/thaw action, ice, or toe erosion. Probable causes of particle erosion include (1) stone size not large enough; (2) individual stones removed by impact or abrasion; (3) side slope of the bank so steep that the angle of repose of the riprap material is easily exceeded; and (4) gradation of riprap too uniform. Figure D4.2 provides a photograph of a riprap failure due to particle displacement.

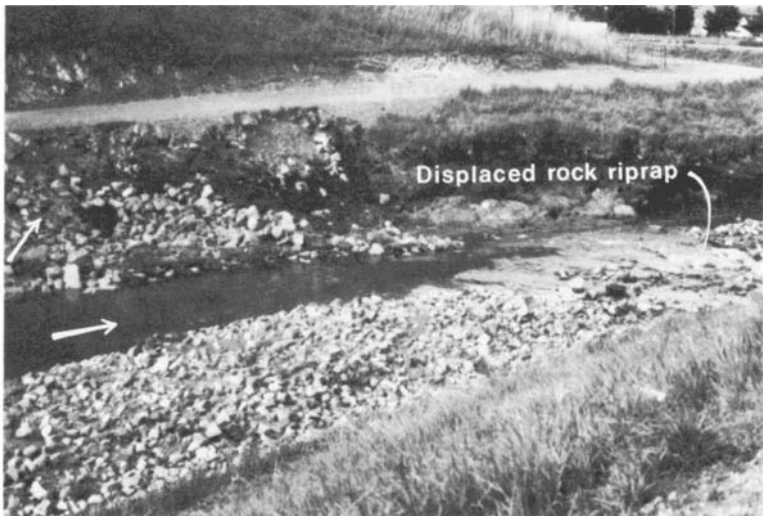
Translational Slide

A translational slide is a failure of riprap caused by the downslope movement of a mass of stones, with the fault line on a horizontal plane (Figure D4.3). The initial phases of a translational slide are indicated by cracks in the upper part of the riprap bank that extend parallel to the channel. This type of riprap failure is usually initiated when the channel bed scours and undermines the toe of the riprap blanket. This could be caused by particle erosion of the toe material, or some other mechanism which causes displacement of toe material. Any other mechanism that would cause the shear resistance along the interface between the riprap blanket and base material to be reduced to less than the gravitational force could also cause a translational slide. It has been suggested that the presence of a filter blanket may provide a potential failure plane for translational slides. Probable causes of translational slides are (1) bank side slope too steep, (2) presence of excess hydrostatic (pore) pressure, and (3) loss of foundation support at the toe of the riprap blanket caused by erosion of the lower part of the riprap blanket. Figure D4.4 provides a photograph of a riprap failure due to a translational sliding-type failure.



Source: Blodgett and McConaughy (1986)

Figure D4.1. Riprap failure by particle erosion.

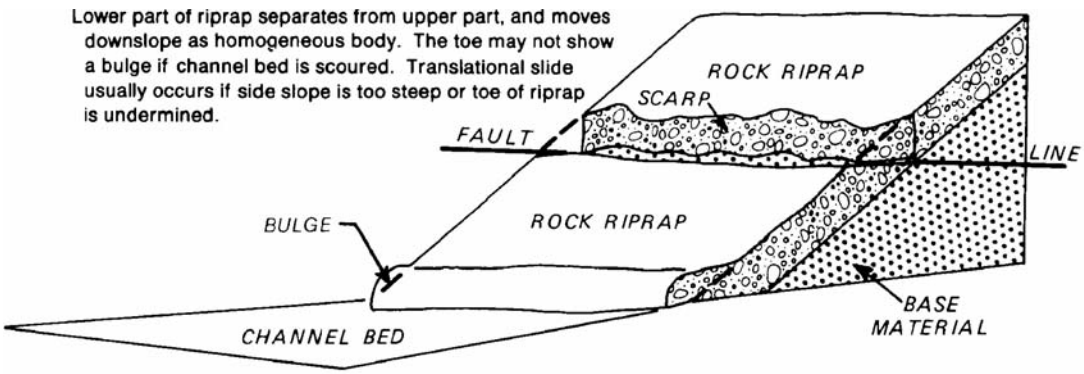


Note deposition of displaced riprap from upstream locations in channel bed (photographed March 1982).
Source: Blodgett & McConaughy (1986)

Figure D4.2. Damaged riprap on left bank of Pinole Creek at Pinole, California, following flood of January 4, 1982.

Modified Slump

Modified slump failure of riprap (Figure D4.5) is the mass movement of material along an internal slip surface within the riprap blanket. The underlying material supporting the riprap does not fail. This type of failure is similar in many respects to the translational slide, but the geometry of the damaged riprap is similar in shape to initial stages of failure caused by particle erosion. Probable causes of modified slump are (1) bank side slope is so steep that the riprap is resting very near the angle of repose, and any imbalance or movement of individual stones creates a situation of instability for other stones in the blanket and (2) material critical to the support of upslope riprap is dislodged by settlement of the submerged riprap, impact, abrasion, particle erosion, or some other cause. Figure D4.6 provides a photograph of a riprap failure due to a modified slump-type failure.



Source: Blodgett & McConaughy (1986)

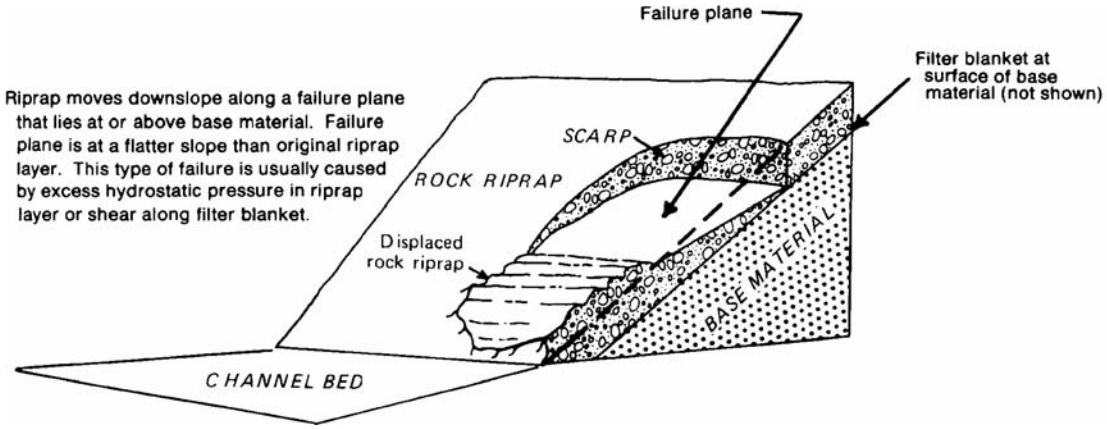
Figure D4.3. Riprap failure by translational slide.

Source: Blodgett & McConaughy (1986)

Figure D4.4. Riprap on Cosumnes River at Site 2 near Sloughhouse, California, looking downstream, showing translational slide failure (photographed May 31, 1983).

Slump

Slump failure is a rotational-gravitational movement of material along a surface of rupture that has a concave upward curve (Figure D4.7). The cause of slump failures is related to shear failure of the underlying base material that supports the riprap. The primary feature of a slump failure is the localized displacement of base material along a slip surface, which is usually caused by excess pore pressure that reduces friction along a fault line in the base material. Probable causes of slump failures are (1) non-homogeneous base material with layers of impermeable material that act as a fault line when subject to excess pore pressure; (2) side slopes too steep and gravitational forces exceeding the inertia forces of the riprap and base material along a friction plane; and (3) too much overburden at the top of the slope (may be caused in part by the riprap). Figure D4.8 provides a photograph of a riprap failure due to a slump-type failure.

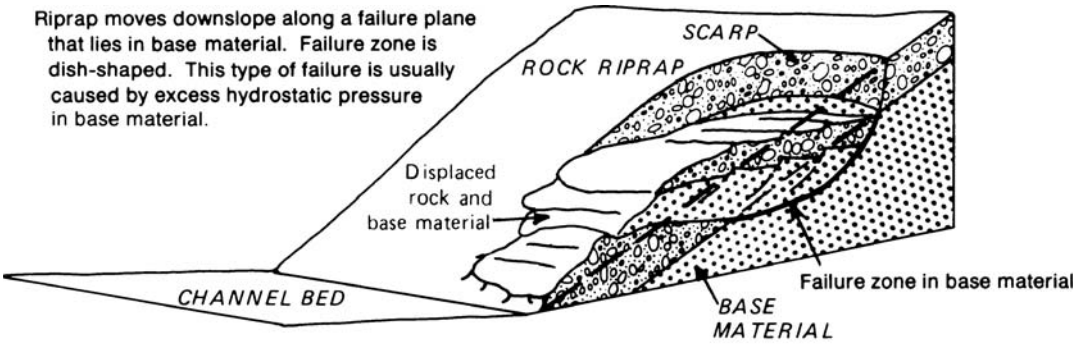


Source: Blodgett & McConaughy (1986)

Figure D4.5. Riprap failure by modified slump.

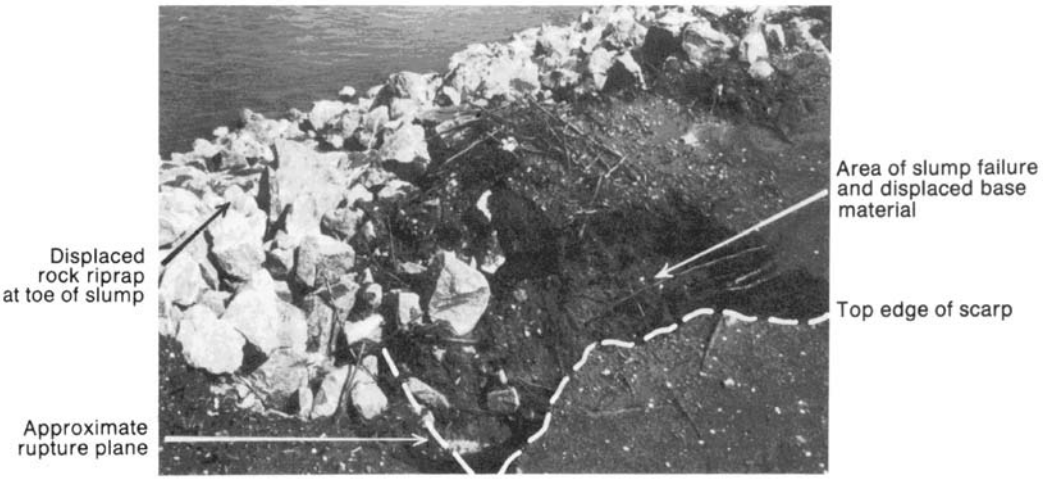
Source: Blodgett & McConaughy (1986)

Figure D4.6. Riprap on Consumnes River at Site 3 near Sloughhouse, California, looking downstream, showing modified slump failure (photographed May 31, 1983).



Source: Blodgett & McConaughy (1986)

Figure D4.7. Riprap failure due to slump.



Source: Blodgett & McConaughy (1986)

Figure D4.8. Riprap on left bank of Cosumnes River at Site 1 near Sloughhouse, California, showing slump failure (photographed May 31, 1983).

Summary

Blodgett and McConaughy (1986) conclude that certain hydraulic factors are associated with each of the four modes of riprap failure (particle erosion, translational slide, modified slump, and true slump). 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, they identify the following reasons for riprap failures:

- Particle size was too small because
  - Shear stress was underestimated
  - 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
- Channel changes caused
  - Impinging flow
  - Flow to be directed at ends of protected reach
  - Decreased channel capacity or increased depth
  - Scour
- Riprap material had improper gradation
- Material was placed improperly
- Side slopes were too steep
- No filter blanket was installed or blanket was inadequate or damaged
- Excess hydrostatic pressure caused failure of base material or toe of riprap
- Differential settlement occurred during submergence or periods of excessive precipitation

4.2 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 m^3/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 stratified glacial drift, which was considered non-erodible by the designers (Figure D4.9). 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 to 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 to 50 scale and a 1 to 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 D4.10).

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 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."

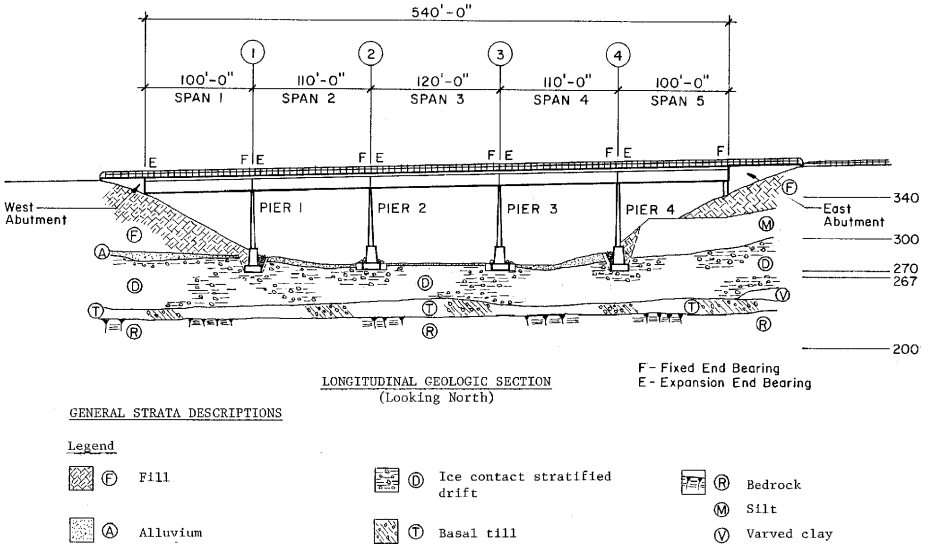


Figure D4.9. South elevation of Schoharie Creek Bridge showing key structural features and a schematic geological section.

Note: Pier 2 is in the foreground with Pier 3 in the background.

Figure D4.10. Pier scour holes at Schoharie Creek bridge in 1987.

The NYSTA inspected the bridge annually or biennially with the last inspection on April 1, 1986. A 1979 inspection by a consultant hired by NYSDOT indicated that most of the riprap around the piers was missing (Figures D4.11 and D4.12); 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,

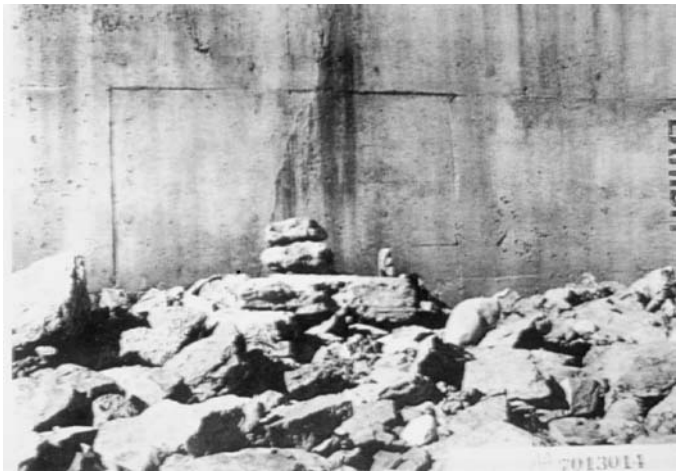
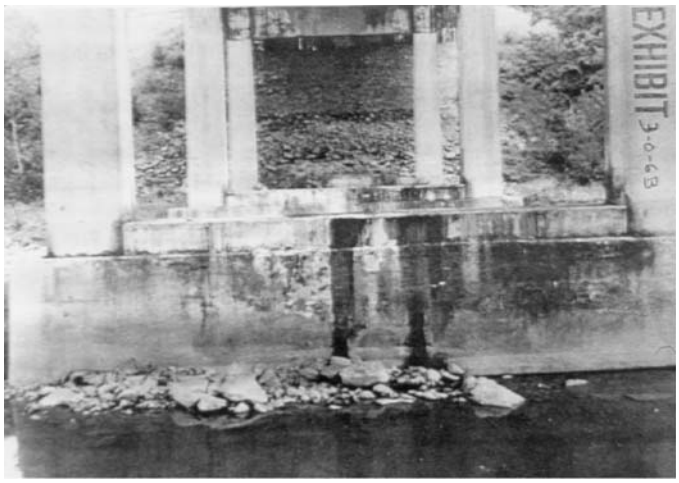


Figure D4.11. Photograph of riprap at Pier 2, October 1956.



Note: Flow is from right to left.

Figure D4.12. Photograph of riprap at Pier 2, August 1977.

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
  - 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

5 References

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Abbreviations and acronyms used without definitions in TRB publications:

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
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	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
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