APPENDIX B

Riprap Design

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B.1 INTRODUCTION

B.1.1 Objective and Scope

The objective of this appendix is to present methods of designing riprap protection for rivers and open channels, including methods for determining stone sizes and other important factors. Design against wave action is not addressed in detail, but pertinent references are presented. References to more detailed design information are cited throughout the text.

Riprap, mostly in the form of natural stone, is one of the most commonly used materials for erosion protection in revetments, dikes and groins, toe protection, and other types of hydraulic structures. Riprap consists of loose, coarse elements whose stability is derived mainly from their submerged weight and in some cases from interlocking forces with adjacent elements. The use of stone to prevent erosion or provide stability has a long history. A still widely used equation by Isbash (1935), relating the required stone diameter to the square of the velocity, was apparently anticipated by a similar relationship presented by A. Brahms in 1753 (Forchheimer 1914).

B.1.2 Advantages and Disadvantages of Riprap

The primary advantages of riprap are flexibility, tendency to be self-healing, relative ease of construction, and extensive experience and design guidance to support its use. In many parts of the world, stone is one of the most abundant and long-lasting building materials: Roman aqueducts built in Spain in the first century A.D. are still standing today. Local failures are easily repaired if done promptly. To some, riprap has a reasonably natural appearance, and vegetation can be incorporated into it to provide a more natural appearance.

Disadvantages of riprap include its limited availability and relatively high cost in some areas, environmental restrictions on use, variations in quality, and difficulties of transport and placement in some locations.

B.1.3 Design Factors

Existing engineering literature on riprap focuses predominantly on the stable sizes required to resist movement from waves and currents. However, size is only one of many important aspects of riprap design. Thorne et al. (1995a) present five requirements in the design of riprap structures:

- The structure must be capable of withstanding the combined impact of all the forces of water flow and wave attack responsible for erosion and destabilization. This determination is based on such factors as stable stone size, lateral and vertical extent of protection, and alignment.
- The structure must be safe with regard to geotechnical stability, foundation settlement, and groundwater seepage.
- The structure must be built using sufficiently durable materials to retain the required erosion resistance and mass stability over the design life of the project.
- The ecological impacts and aesthetics of the structure have to be acceptable to today's society.
- The structure must be economical to build using available materials, equipment, and labor.

B.2 RIPRAP STRUCTURE TYPES

B.2.1 Bank Revetment

In many applications, riprap bank revetments have traditionally been placed from the toe of the slope to the top of the bank and have generally been kept relatively free of vegetation. There are exceptions, however. On some large rivers such as the Mississippi, riprap on the upper part of the bank is often combined with articulated concrete mattress on the lower portion, because of the difficulty and uncertainty of placing riprap underwater in large depths and of high velocities. On some small to intermediate streams, on the other hand, riprap is used on the lower portion of the bank, with planted or adventitious vegetation on the upper portion (Fig. B-1). Reasons for these mixed treatments include reduced costs and environmental benefits.

Riprap bank revetment is also used to control the effects of rapid water-level drawdown caused by large-displacement vessels in confined navigation channels (Schulz 1995). In these applications, special attention must be paid to filters and to layering within the revetment. This is also important where revetments are designed to prevent piping due to water surcharge into streambanks, either from overbank sources or from rising river levels.

B.2.2 Revetment Adjacent to Hydraulic Structures

Riprap is widely used to protect zones upstream and downstream of hydraulic structures such as spillways and outlet works. Many forms of energy dissipators use riprap downstream of the structures to resist streambed scour.

In some cases, riprap is used to form grade control structures on small to intermediate streams. Riprap is also widely used to prevent scour downstream of culverts.

B.2.3 Toe Protection and Launchable Stone

Because of its flexibility and self-healing nature, riprap is often used as toe protection for bank revetments and other channel control works. Toe protection can be placed either down to the anticipated maximum scour depth, or in an enlarged section at the toe of the bank that will "launch" as scour occurs.

Various forms of launchable riprap used by the Corps of Engineers include weighted riprap toe, placed at the toe of the slope; trenchfill revetment, placed at the low-water reference plane, often around midbank height; and windrow revetment, placed on the top of the bank. The launching action should be gradual, causing the rock to creep rather than avalanche down the slope—generally, this requires that launchable riprap be restricted to noncohesive beds and banks.

Fig. B-1. Riprap protection on lower bank only. Photo by author.

According to Simons (1995), the launchable stone method was first used for large alluvial rivers in India—"falling aprons" were described by Spring (1903). A somewhat similar concept in the wave environment is the dynamic revetment, designed to be reworked by wave activity into a stable, relatively flat slope (Ahrens 1995).

B.2.4 Dikes, Groins, and Bendway Weirs

Riprap is often used to form or cover dikes, groins, and bendway weirs for river training and bank protection. In large rivers, these structures may be used to improve navigation depth and alignment. In major rivers such as the Mississippi, dikes are often constructed in stages, allowing the response of the river to be monitored at each stage.

Bendway weirs have been used on the Mississippi River to allow a wider navigation channel in bendways. They form submerged sills attached to the outer bank and angled upstream, with lengths of one-third to one-half of the channel width. On smaller streams, bendway weirs may be used for bank protection, to redirect flow away from eroding banks (Derrick and Northcutt 1996). A relatively short type of riprap groin called a hardpoint has been used to resist bank erosion in moderately curved reaches of the Missouri River (USACE 1981).

On rivers like the Mississippi, riprap gradation for these types of structure is generally "quarry-run": the stone receives little quarry processing other than removing the largest sizes. Besides reducing costs, quarry-run riprap is considered by many to have the advantage of providing its own filter.

B.2.5 Bridge Piers and Abutments (See Chapter 11)

B.2.6 Wave Protection Including Boat Waves

Riprap design in the marine-wave environment (which has been the main focus of wave riprap research) is not covered in this appendix; however, some wave problems occur in the riverine environment. Riprap is frequently used to protect the upstream faces of dams from wind-generated waves and to protect navigation channels from boat-generated waves.

On most rivers, where fetch is generally limited, maximum short-period wave heights are caused by boat waves rather than wind waves. Few river revetments, however, have been constructed solely as protection against boat waves.

B.2.7 Steep Chutes and Channels

The term "steep" refers here to slopes of 2 to 50%. Riprap applications on steep slopes include resisting the overtopping of dams, levees, and roadways and capping and sealing waste-disposal impoundments.

On a steep chute, the flow remains supercritical for a significant distance down the slope. Supercritical flow has a tendency to concentrate in any locally weak spots, leading to local erosion and further concentration. Flow concentration may also result from less-than-ideal entrance conditions at the top of the slope. The problem of flow concentration and channelization can be addressed by using conservatively high estimates of unit discharge for design, with relatively uniform riprap gradations. If more widely graded rock is used, strict quality control is required to prevent size segregation during construction.

Grouted riprap is often used on steep slopes, especially if unit discharges are so high that stable riprap sizes become impracticably large.

B.2.8 Bed Protection

Riprap may be used as bed protection for berthing and fleeting areas in navigation channels, around bridge foundations, and over pipelines and as a cap for contaminated sediments. Such protection may have to resist river and tidal flows, wave attack, and wash from vessel propellers and jets. Filters are normally incorporated into the design.

B.2.9 River Closure Structures

River closures using dumped stone may proceed by horizontal closure, vertical closure, or a combination of both. The best-known publication is by Isbash (1935). Isbash's equation relating stone size to velocity still serves as a basis for riprap design in river closures and elsewhere.

B.3 PHYSICAL CHARACTERISTICS OF RIPRAP STONE

B.3.1 Rock Type/Sources

Riprap is mostly obtained from rock quarries. Other sources and substitutes include boulder-containing deposits of glacial and fluvial origin, broken concrete or soil cement, and slag from mining operations.

A European publication (CUR 1995) lists rock geological types used in hydraulic engineering, with their advantages and disadvantages. Geological rock type alone is not a useful guide to acceptability, because of the wide variation in properties within a given type.

B.3.2 Testing/Sampling

Difficulties are often experienced in ensuring that in-place riprap meets specified size gradations. Compliance testing generally involves taking one sample per so many units of weight or volume placed. Laan (1995) states: "With large batches a check may be carried out every 10,000 to 30,000 tonnes." The USACE (1990d) suggests a gradation test for every 10,000 cu yds (7651 cu m).

Another sampling question involves the size of each sample. In most sampling to check gradation, each particle is weighed rather than measured. According to (USACE 1990d), a sample should weigh about 100 times the average stone weight, assuming a maximum riprap size of about 0.9 m (36 inches). Laan (1995) recommends that samples of rock smaller than 300 kg maximum weight should contain a minimum of 200 pieces. These guidelines refer generally to relatively uniform riprap; widely graded mixtures may require a larger number of pieces. Additional information on testing and sampling is given by CUR (1995) and Thorne et al. (1995a).

B.3.3 Stone Density

According to (USACE 1994), stone should have a specific weight of 2,400 kg/m^3 (150 lb/ft^3) or greater. Most stone used for riprap has a unit weight between 2,500 and 2,700 kg/m^3. The densities of various rock types are given in Table 2–2 of Chapter 2.

B.3.4 Shape and Porosity

Riprap stone should be blocky rather than elongated, and angular rather than rounded. Tests by Olivier (1967) and by Abt and Johnson (1991) confirmed that for equivalent stability, rounded stone must be larger in dimension than angular stone.

Stones can be considered to have three mutually perpendicular axes:

- The major axis *a,* representing the maximum length.
- The intermediate axis *b,* defining the maximum width. (The major and intermediate axes define the orientation of the minor axis.)
- The minor axis *c,* defining the thickness. CUR (1995) defines *c* as the minimum distance between two parallel planes between which the stone could pass.

It is often specified that the ratio *a/c* should be less than 3, except for a small percentage of stones (USACE 1994; CUR 1995).

The relationship between stone size and weight may be computed on the basis of a sphere, or a cube, or halfway between—depending on typical shapes of the material in question.

Porosity is usually in the range of 40 to 45% for uniform riprap, and 25 to 35% for graded riprap.

B.3.5 Durability

Riprap durability is a key consideration. Riprap should be able to withstand transport, handling, placement, and freeze-thaw without significant size deterioration. Problems with durability increase with increasing stone size and become very important for the large stone sizes used in many coastal projects. CUR (1995) reports that durability is well correlated with density, and that breakage of stone is of two types: (1) along existing flaws; and (2) along new fractures, usually involving loss of edges and corners. For rock weights less than 300 kg, the first type is more common. USACE (1990b) gives additional information on durability.

B.3.6 Gradation

Riprap gradation affects many aspects of revetment design including stability, filter requirements, unit cost, and placement. The degree of nonuniformity is usually expressed by the ratio D_{85}/D_{15}. Gradation types can be classified as shown in Table B-1.

Very widely graded stone has a relatively low unit cost at the quarry, and the substantial proportion of fines usually present is considered by some to provide a filter. Quarry-run stone with a D_{85}/D_{15} ratio of about 6 has been widely used in the lower Mississippi River basin for dike construction. A disadvantage of widely graded riprap, however, is its tendency to segregate during placement—if this occurs, the effective size for stability may be highly uncertain. Flume tests have consistently shown that for the same median size and layer thickness, hydraulic stability decreases with wider gradations (Anderson et al. 1970; Abt et al. 1988; Maynord 1988). As a general rule, very widely graded riprap must be placed to greater thicknesses to achieve equivalent protection.

Although uniform riprap gradations exhibit relatively greater stability, their higher porosities can allow hydraulic action to reach the underlying soil. They may require several mineral filter layers to bridge the size gap between the smallest riprap stones and the underlying material. Where it is impracticable to provide a filter, as with launchable riprap, wider gradation is preferable.

Riprap gradation is often specified in the form of upper and lower limit curves, any intermediate gradation being regarded as acceptable. Generally, the narrower the specified limits, the higher the production costs. Rock sizing procedures discussed subsequently should be used to define the lower limit curve.

Table B-1 Gradation Types

Descriptive term	D_{85}/D_{15}
Uniformly or narrowly graded	<1.5
Widely graded	1.5 to 2.5
Very widely graded, including quarry-run	>2.5

A typical gradation plot based on standard gradations used in USACE (1994) is shown in Fig. B-2. Standardized gradations are frequently used on a local basis, but have not been adopted on a national basis in the United States.

A European reference (CUR 1995) classifies rock gradations as follows:

- *Heavy:* requiring handling of individual pieces.
- *Light:* often processed using "grizzlies" or large bar sieves.
- *Fine:* processed by screens with square openings less than 200 mm.

The light CUR gradations are similar to the USACE (1994) gradations shown in Fig. B-2. This Figure also shows gradations by AASHTO (Brown and Clyde 1989) and by Simons and Senturk (1977), as well as a very wide gradation used for dikes on the Mississippi River.

B.3.7 Revetment Thickness

Up to a point, revetment thickness has an effect on the stability and durability of riprap protection. Thickness is generally specified as a multiple of maximum size D_{100} or of median size D_{50}. For relatively low-turbulence applications such as bank protection, USACE (1994) specifies a minimum thickness of D_{100} or $1.5 \times D_{50}$, whichever is greater. For high-turbulence applications, such as below energy dissipators, the same reference specifies $1.5 \times D_{100}$. Escarameia (1998) recommends a minimum thickness of $2 \times D_{50}$.

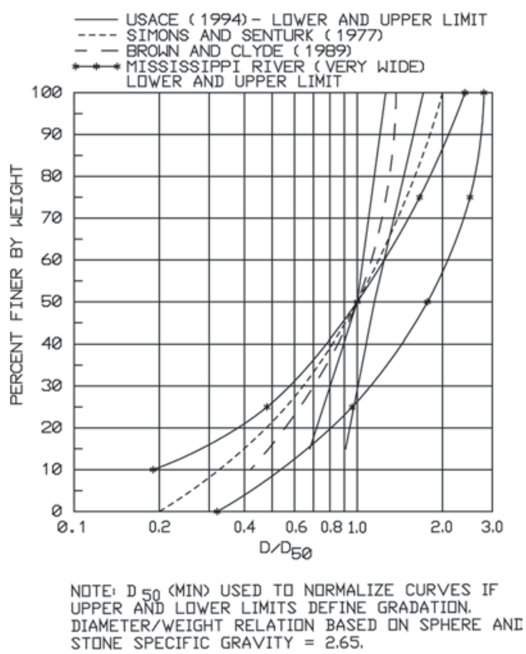

Fig. B-2. Standard gradation curves. Adapted from USACE (1994), Simons and Seuturk (1977), and Brown and Clyde (1989).

Stability tests (Abt et al. 1988; Maynord 1988) show that additional thickness above these minima generally results in increased stability—consequently, a greater thickness of a smaller gradation may sometimes provide equivalent stability. The increase in stability with thickness is greatest for very wide gradations and relatively small for uniform gradations. According to Simons (1995), the improvement in stability with increasing thickness results because more material is available to move to damaged areas, and more energy has to be dissipated before the filter or underlying soil is exposed.

It is common practice to use 50% greater thickness under water, because of uncertainties in placement. On the basis of flume studies, Hunt (1998) presented relations for the required excess thickness as a function of depth and velocity.

B.3.8 Roughness

Hydraulic roughness and flow resistance are covered generally in Chapter 2.

Most riprap applications involve relatively high relative roughness, with fully rough turbulent flow. The Strickler equation relating roughness to grain size is therefore applicable. The Manning/Strickler roughness coefficient for riprap is formulated in USACE (1994) as

$$n = K \times D_{90}^{1/6} \tag{B-1}$$

where D_{90} is in feet and $K = 0.036$, assuming slopes less than 2% depth/D_{90} ratios from 3 to 30, and above-water placement. For underwater placement, K is increased by about 15%.

Alternatively, CUR (1995) uses the logarithmic form of the flow formula, where $k_s = 2\ D_{90}$ is used to define grain roughness height. Other values of k_s are given in Table 2–1 of Chapter 2.

For riprap on steep slopes, Rice, et al. (1998) present different equations for Manning's n and Darcy's f.

In the design of porous structures formed of riprap, it may be necessary to calculate head losses for through-flow. Guidance can be found in Keulegan (1973); Stephenson (1979); Jain et al. (1988); and CUR (1995).

B.4 SIGNIFICANCE OF HYDRAULIC LOADING

Most stone-sizing equations make stone size dependent on velocity or wave height raised to a power of 2 or greater. This makes determination of the hydraulic loading a key element in design. For the coastal environment, accurate determination of wave loading has received detailed attention in the literature. In many river applications, on the other hand, the data required for sophisticated formulations are often unavailable, and relatively simple methods for quantifying the hydraulic loading are required.

B.4.1 Descriptors of Channel Type and Bend Severity

For the purpose of discussing the significance of channel cross-section and alignment for riprap design, channels can be classified as (1) natural irregular or (2) artificial trapezoidal:

- Natural irregular channels have irregular alignments and cross sections, with erodible beds and sediment transport leading to toe scour and bar building, often concentrating flow along the outer bank. In such channels, additional roughness due to bank riprap is usually of little hydraulic significance.
- Artificial trapezoidal channels do not usually exhibit bars or toe scour, because rates of sediment transport are generally low and the bed is usually formed in bedrock or lined with riprap. If much of the perimeter is lined with riprap, the increase in roughness may be substantial, with effects on depth and velocity that affect riprap sizing. An iterative solution for riprap sizing is then required.

In channel bends, the most significant parameter with respect to riprap sizing and scour depth estimation is the ratio of centerline radius of curvature to water-surface width at the entrance to the bend. These dimensions should be based on flow in the main channel, excluding overbank areas. Another significant parameter is the total deflection angle.

B.4.2 Parameters Defining Hydraulic Loading

The hydraulic loading classification shown in Table B-2 is due to Escarameia (1998).

Escarameia (1998) proposes that these classifications be used to assess which protection systems are appropriate for a given class of hydraulic loading: for example, bioengineering is considered appropriate only for light hydraulic loading, as tabulated. Other investigators, however, might rate bioengineering as sometimes suitable for heavier loading classes. Theoretically, riprap is appropriate for all loadings. In practice, however, it may be impracticable to obtain or handle stone sizes large enough for the heaviest loadings, unless grouting or other forms of reinforcement are used.

Hydraulic loading can be evaluated using various techniques: physical models and/or numerical models, empirical methods, and prototype data. Physical and numerical models are excellent tools for determining design velocities, but the necessary input data are not always available for projects such as local bank protection. Empirical methods use observed data; their most significant limitation is that prototype data can seldom be obtained for design flood conditions.

A possible exception is in multichannel or braided streams, where critical erosion conditions due to severe impingement on banks may occur under bank-full or other discharges considerably smaller than the maximum (Maynord 1993).

To characterize the hydraulic loading in an open channel, boundary shear stress is theoretically the most appropriate parameter because it represents forces exerted on the riprap that can easily be compared with other stabilizing forces. The average boundary shear stress is easily determined from simple open channel theory. However, riprap design should be based on local maximum values, for example along the outer bank of a bend.

To estimate maximum shear stress along the outer bank of a channel bend, USACE (1970) multiplies the average shear stress by a factor dependent on the ratio of radius to width *(R/W),* as shown in Fig. B-3. Another method is to calculate local shear stress using velocity distribution relations given in Chapter 2. Data on near-bed velocity provide the best shear stress estimates, but are seldom available. Local depth-averaged velocity is an alternative, but difficulties arise over the appropriate choice for roughness height k_s, the location of the virtual bed or velocity-profile origin when the bed consists of large coarse particles, and the validity of velocity-profile equations for high relative roughness (van Rijn 1982).

Because of these difficulties in evaluating local boundary shear stress, many practitioners have a preference for velocity predictors. Velocity parameters that have been used include

- *Average cross-sectional velocity:* Riprap design equations using this parameter include those of Blodgett and McConaughy (1986) and Brown and Clyde (1989).
- *Local depth-averaged velocity:* Design equations using this parameter include USACE (1994) and Pilarczyk (1990).
- *Local near-bed velocity:* Design equations using this parameter include those of Isbash (1935) and Escarameia and May (1992).

Another advantage of velocity-based relations is that, generally, velocity can be visualized and measured more easily than shear stress.

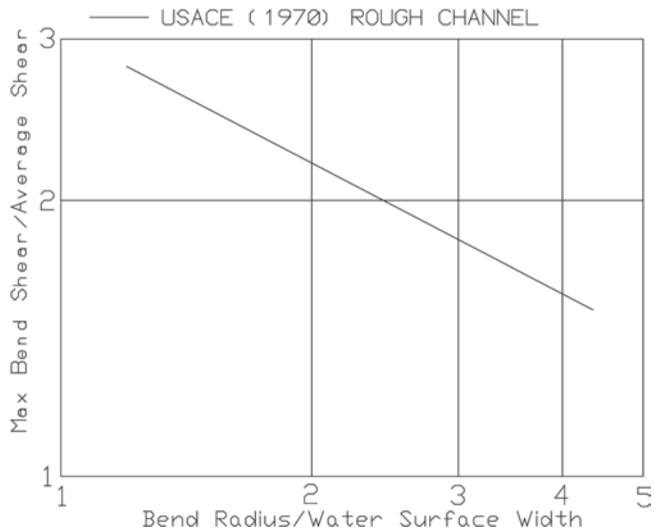

Fig. B-3. Peak shear stress in bend/average shear stress. Reprinted by permission from USACE (1970).

To characterize hydraulic loading due to waves, wave height is the preferred parameter. Numerous studies have addressed selection of the most appropriate wave height. (USACE 1984a; CUR 1995).

B.4.3 Hydraulic Loading for Bank Protection Design

Use of average cross-sectional velocity ($V_a = Q/A$) in a riprap design equation may be problematic because of wide variability in cross sections and in alignments. However, empirical methods are available to use V_a along with geometric parameters to estimate local depth-averaged velocity *V.*

USACE (1994) presents velocities along the outer bank of bends as a function of *R/W* and V_a for natural channels (Fig. B-4), and also for trapezoidal channels. The outer bank velocity is defined as the depth-averaged velocity at 20% of the bank length upslope from the toe—this definition was selected to accord with the location of maximum

Table B-2 Classification of Hydraulic Loading

Hydraulic loading		
Mean channel velocity, V_a, m/s	Significant wave height or maximum boat wave height, H, m	Classification
<1	<0.15	Light
1–2.5	0.15–0.5	Moderate
2.5–4	0.5–1.0	Heavy
4–7	>1.0	Very heavy
Downstream of hydraulic structures, around sharp bends, bridge piers, transitions		High turbulence

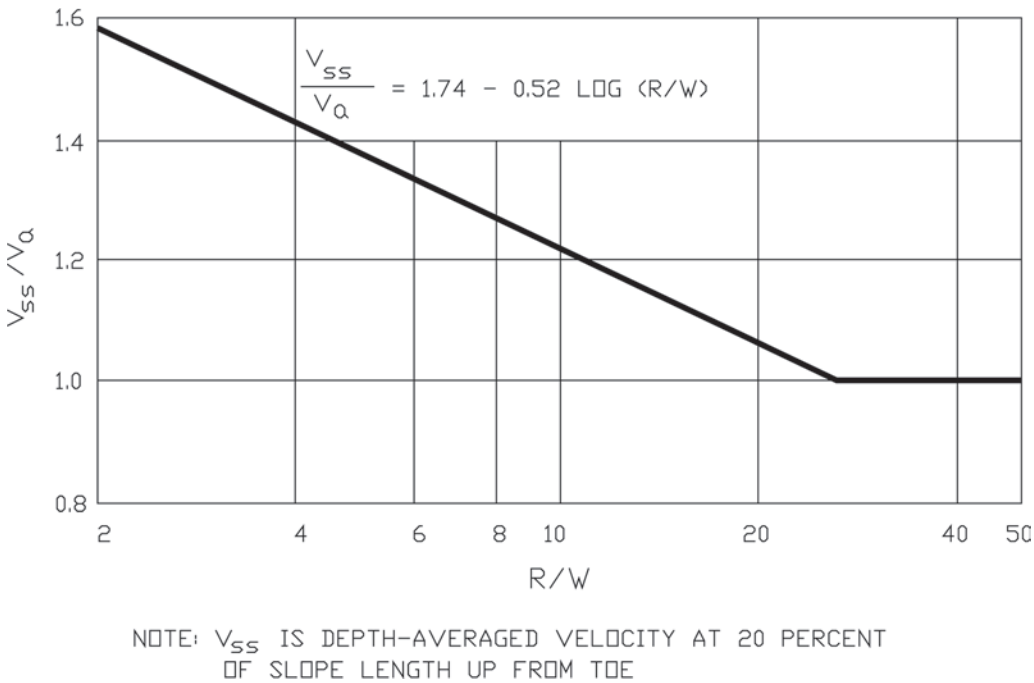

Fig. B-4. Near bank velocity of natural channel. Reprinted by permission from USACE (1994).

side-slope shear stress as found in studies of straight channels (Chow 1959). In computing these parameters, dimensions and discharge should represent the main channel only, excluding overbank areas.

According to Thorne et al. (1995b), an analytical model by Bridge (1982) provides a good estimate of outer bank velocity in natural channel bends.

B.4.4 Hydraulic Loading for Steep Slopes and River Closures

For riprap on steep chutes or river-closure structures, velocity may not be an appropriate surrogate for shear stress. Because of the difficulty of defining the water surface and depth on steep slopes, design equations for these situations often use either unit discharge and slope, or head and slope, to represent hydraulic loading. Abt and Johnson (1991) reported that flow concentration factors (= local unit q/average unit q) of up to 3 are possible and should be considered in design. Additional guidance is given by Robinson et al. (1997).

B.4.5 Hydraulic Loading below Energy Dissipators

In large projects, physical model studies are often used to determine or confirm riprap sizing, but for smaller projects this is often impracticable.

Near-bottom velocity is the most reliable parameter for riprap sizing, but is generally available only from physical model studies. Analytically, velocity distributions below end sills of energy dissipator basins cannot be predicted reliably—there is no boundary layer development. Depending on discharge, hydraulic jump characteristics, basin length, baffle block geometry and arrangement, end sill configuration, and number of gates open, the location of maximum velocity can range from near the bottom to near the surface.

To characterize hydraulic loading, the average velocity over the end sill is often used. Where the spillway is nongated, the average velocity is calculated using total discharge, basin width, and minimum tailwater depth. Where the spillway is gated, a more conservative approach is advisable because operators may open only one or more gates to pass ice or debris. The tailwater is then lower than with all gates operating, and attack on riprap can be severe (USACE 1987).

B.4.6 Hydraulic Loading from Propeller Jets

Loadings from propeller jets should be based on a near-bottom velocity, because a depth-averaged velocity has little meaning for propeller jet flows. A difficulty is how to decide the value of propeller thrust to be used in the bottom velocity equation. If the maximum installed power of a vessel is used in every case, the resulting stone size may be larger than necessary. The duration of the design thrust is not taken into account in most design equations.

B.4.7 Hydraulic Loading from Waves

Guidance on hydraulic loading from wind waves can be found in USACE (1984a) and CUR (1995).

Guidance on hydraulic loading from boat waves can be found in PIANC (1987a), Bhowmik et al. (1992), CUR (1995), and Maynord (2005). According to Hemphill and Bramley (1989), in navigation channels the blockage ratio (BR), defined as vessel cross-section area over channel cross-section area, determines the dominant type of wave loading. For BR > 0.1, a long-period drawdown wave caused by the displacement effects of the vessel will be dominant. For $0.05 < \mathrm{BR} < 0.1$, the drawdown wave and short-period secondary waves will be of similar magnitude. For BR <0.05, secondary waves will likely dominate.

B.5 GEOTECHNICAL REQUIREMENTS FOR RIPRAP

As stated by Thorne et al. (1995a), geotechnical requirements for riprap installations include safety against geotechnical instability, foundation settlement, and groundwater seepage. The first two items are beyond the scope of this appendix. Groundwater seepage effects, which may require a filter design, are addressed below.

B.5.1 Seepage Effects

Seepage emerging from a streambank may be a major cause of bank instability (Hagerty 1991a; 1991b). Observed bank failures due to seepage occurred in layered and lensed alluvial deposits. Water recharged by floods into streambanks, or derived from infiltration on overbank areas, can penetrate pervious alluvial layers and emerge from bank faces, eroding particles from sandy layers. Hagerty reports that failures due to seepage effects also occur at riprap revetments both with and without filters—filters over silty or gap-graded soils easily become clogged.

One Ohio River site experiencing significant seepage-induced bank instability was protected with an extensive filter overlain with riprap, and has performed well W. A. Cutter and R.C. Waterman unpublished manuscript, November 1984. A section through this revetment is shown in Fig. B-5 (replotted from Cutter and Waterman 1984).

B.5.2 Filter Objectives

Filters (mineral or fabric) are placed beneath riprap revetments to meet one or more of the following objectives:

- To prevent groundwater behind the revetment from transporting bank material through the riprap (piping). This is usually the primary purpose of a filter. Filters must meet two basic requirements: stability (to prevent piping) and permeability. The filter should be fine enough to prevent the base material from passing through, but more permeable than the base soil being protected. See Section B.5.1.

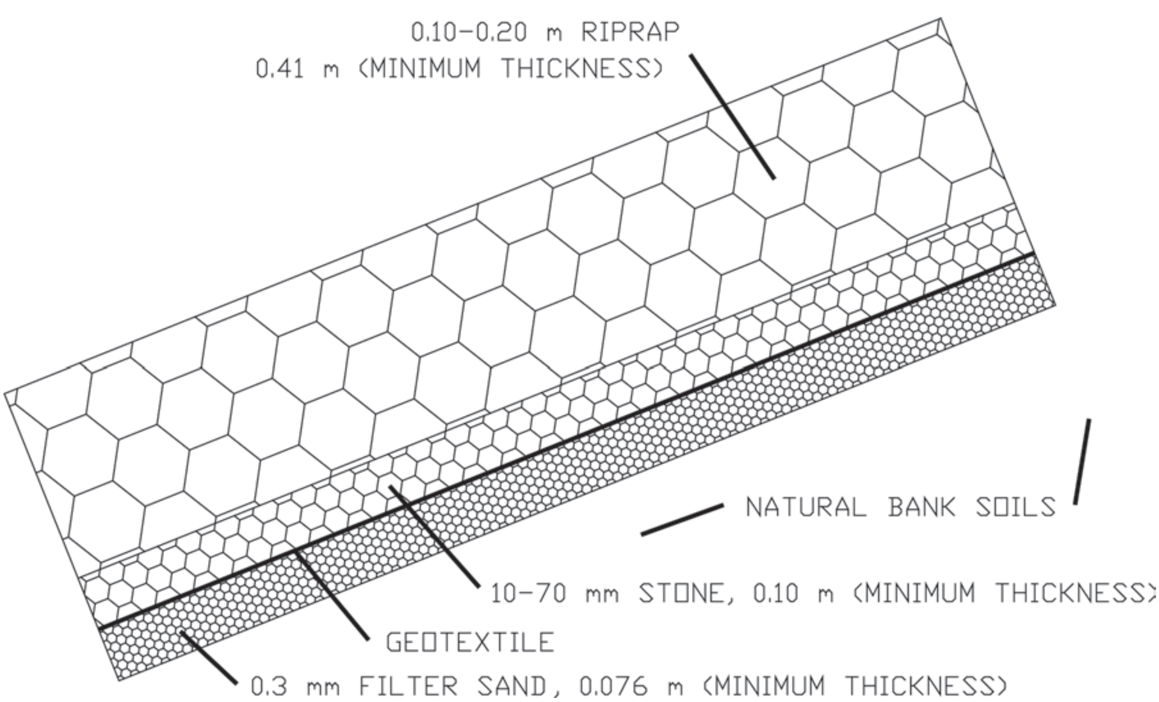

Fig. B-5. Minimum section recommended for Ohio River revetment to address seepage failure. Adapted from W.A Cutter and R.C. Waterman unpublished manuscript November (1984).

- To prevent large-scale turbulence in front of the revetment from sucking bank material through the riprap. Turbulence is certainly capable of this in the wave environment and in high-energy zones below hydraulic structures and on steep slopes. In these environments, the filter dissipates the large-scale turbulence before it reaches the bank material. In the smaller-scale turbulence environments of typical streambank protection projects, through-erosion by turbulence is generally limited to banks composed of fine or weak material.
- To serve as a foundation to distribute load in poorly consolidated soils. Where soils have not been sufficiently compacted, filters can prevent individual stones or armor units from sinking into the base soil.

Filters are widely used beneath revetments in the wave environment, adjacent to hydraulic structures, on steep chutes, and in bed protections such as vessel berthing areas.

B.5.3 Geotextile Filters

Filter fabrics have been widely used in bank and channel protection projects. Advantages include ease of installation; economy; consistent quality; tensile strength; general availability; and small thickness. Disadvantages include problems with placement underwater; unproven durability; bacterial activity, which can affect performance; relative movement between fabric and bank material; failure on steep slopes; requirements for edge protection; susceptibility to damage; difficulty of repair; and the careful design and installation needed to accommodate settlement.

Because of sliding problems with filter fabric, its use is generally limited to slopes of 1V:2H or flatter. To prevent damage during riprap placement, some designers specify a bedding layer of small rock or gravel on top of the fabric, or high-strength fabrics more resistant to puncture. Clogging of geotextile fabrics is a major concern, and proper selection of the fabric mesh to match the bank soil gradation is essential. Some fabrics incorporate a roughness layer attached beneath the geotextile, to restrict water movement beneath the fabric. This roughness layer makes the overall fabric 2 to 3 cm thick and increases overall strength and resistance to puncture.

Additional guidance for selection of geotextiles is given in USACE (1984b); USACE (1986a); Brauns et al. (1993); CUR (1995); and Escarameia (1998).

B.5.4 Mineral Filters

Advantages of mineral filters are that they are self-healing; generally durable; deformable without serious damage; and relatively easy to repair. Disadvantages include the careful control required to achieve specified gradation and thickness; difficulty of compaction on steep slopes; and difficulties in control of underwater placement. Mineral filters are more widely used adjacent to hydraulic structures than geotextile filters due to concerns about clogging of geotextiles over the design life of the structure.

Traditional guidance for mineral filters used in dam construction requires consideration of stability and permeability (USACE 1986b). To prevent piping and ensure stability requires D_{15} (filter)/D_{85} (soil) < 5, and D_{50} (filter)/D_{50} (soil) < 25. To ensure adequate permeability requires D_{15} (filter)/D_{15} (soil) > 5. CUR (1995) provides similar guidance. USACE (1986b) states that these criteria are applicable to all soils whose gradation curve is parallel to the chosen filter material. Application of these criteria can result in multiple filter layers being required when riprap is large and/or uniform and when the base soil is sand or silt.

Worman (1989) and Bakker et al. (1994) found that for revetments subjected primarily to stream turbulence, stability and permeability criteria can be relaxed using what are called "open" filter criteria. Worman's equation, based on riprap around bridge piers, is

$$\frac{V^2}{gS} = 6\frac{D_{85}(\text{base})}{D_{15}(\text{riprap})} \tag{B-2}$$

where

V = mean flow velocity above the revetment;
g = acceleration due to gravity;
S = revetment thickness;
D_{85} (base) = 85% passing size of the base material;
D_{15} (riprap) = 15% passing size of the riprap.

B.6 ENVIRONMENTAL REQUIREMENTS FOR RIPRAP

CUR (1995) classifies environmental impacts into construction impacts and long-term impacts. Sources of construction impacts include the following:

- Quarrying and dredging of materials;
- Materials transport;
- Noise, vibration, dust, odor, and pollution from equipment;
- Effects on local community in regard to employment, commerce, recreation, and access to the site.

Long-term impacts include the following:

- Changes in bathymetry and landscape due to construction;
- Changes in existing processes such as littoral transport;
- Effects on ecology;
- Visual effects, e.g., how the structure fits in with the landscape;

- Socioeconomic effects, such as changes in local employment, changes in access and safety during access, and relief of risk of flooding;
- Geological, archeological, historical, and cultural impacts;
- Pollution of air, water, and soil.

Only the ecological aspects of riprap use are addressed herein.

B.6.1 Ecological Impacts of Riprap

Shields et al. (1995) evaluated the effects of riprap and river-training structures on riverine fish and macroinvertebrates and related impacts to three spatial scales:

- At a microscale represented by median stone diameter, riprap supports dense, diverse populations of macroinvertebrates. Farabee (1986) found that uniform riprap supports higher fish biomass than graded riprap, presumably because the larger interstitial openings provide better habitat.
- At a mesoscale represented by channel width, intermittent structures such as dikes provide better habitat than continuous bank revetments.
- At a macroscale represented by a length of 10 or more channel widths, planform stabilization of large rivers can have significant effects, but these are not attributable to the use of riprap.

Lister et al. (1995) found certain species and life stages of salmon and trout of higher densities near large riprap than around small riprap or cobble structures. On the other hand, at Bodkin Island in Chesapeake Bay, a design concern was that ducklings leaving the island after hatching would fall into the interstices of large riprap used for wave protection.

B.6.2 Reduced Riprap Protection—Lower Bank Only

Sotir and Nunnally (1995) observe that although erosion protection can be accomplished with vegetation only at some sites, most applications require some use of rock to be effective. Riprap revetments for bank protection have traditionally extended from toe of slope to top of bank, or to design water surface plus freeboard. However, ecological benefits and lower project costs can sometimes be obtained by restricting riprap to the lower bank only. On upper banks, flows are of lower duration and frequency and exert smaller hydraulic forces, so that riprap can sometimes be dispensed with.

The minimum required height of riprap depends on the magnitude and duration of hydraulic forces on the upper bank, on upper bank soil strength, and on the strength and flow resistance of bank vegetation used instead of riprap. Recent bioengineering techniques often use soil reinforcement above the riprap to ensure that the upper bank is not damaged before vegetation becomes established (Sotir and Nunnally 1995; Sotir 1998). Benefits of less than full-height riprap include increased habitat in the riparian zone, shade from upper bank vegetation, and improved appearance.

B.6.3 Vegetation in Riprap Revetments

The need to maintain or enhance riparian habitat frequently leads to the concept of allowing vegetation to grow through the riprap. Advantages may include less maintenance, environmental benefits, and velocity reduction due to increased resistance. Disadvantages may include inspection difficulty, increased water levels, large-scale turbulence around large isolated trees, and tree failure, leading to large holes in the revetment.

It is common practice to disallow vegetation on mainline levees or where the increased flow resistance will create unacceptable increases in stage. Shields et al. (1990) found that on a pilot reach of the Sacramento River, “damage rates for revetments supporting woody vegetation tended to be lower than for revetments of the same age and located on banks of similar curvature but without woody vegetation.”

On small to intermediate streams, the larger vegetation is often selectively removed from revetments. In some cases, however, vegetation is planted within the revetments to provide environmental benefits (Haltiner 1995; Sotir and Nunnally 1995; Dittrich 1998).

B.7 SCOUR PROTECTION REQUIREMENTS FOR BANK REVETMENTS

The perimeter of a bank protection revetment is a zone of vulnerability for almost any protection technique. Once failure starts at the toe of the slope, at the top of the bank, or at the upstream or downstream end, the entire revetment may be in jeopardy. Although the flexibility and self-healing nature of riprap make it less susceptible than many alternative forms of protection, careful attention to perimeter details is necessary.

B.7.1 Toe Scour alongside Bank Revetments

River-bed scour at the toe of riprap protection can sometimes result from longterm bed degradation due to upstream or downstream changes, or more frequently from various forms of local scour—at bends, at confluences, or in the vicinity of bridges and hydraulic structures.

Toe scour in bends is a common reason for failure of bank protection projects. (USACE 1981). Potential reasons for local bend scour include the following:

- Cross-sections may become deeper and narrower as a result of making the banks erosion-resistant. Revetments placed down to the preexisting bed level may then be undermined.

- Whether or not banks are protected, scour tends to occur near the outer banks of bendways in high flows. Protection placed down to the normal low-flow bed can then be undermined by high flows.

Notwithstanding the above remarks, Harvey and Sing (1989) reported no tendency for bends to deepen and narrow due to revetment construction, although the thalweg scoured on rising stages. Vanoni (1975) states that sufficiently rough revetments create a zone of low velocity and intensified turbulence that keeps the maximum scour away from the toe of the bank. In some bend reaches of the Missouri River where the elevation of maximum scour may be 9 m (30 ft) lower than the elevation of the revetment toe, no damage has occurred because the scour is far enough away from the toe. However, the zone of lower velocity but stronger turbulence near the toe may result in a greater tendency for bank material to leach through the riprap.

Estimation of local scour depth in bends is theoretically a complex problem involving planform, cross-section shape, hydraulic forces, water and sediment hydrographs, and bed material gradation. The level of effort that would be required to take account of all these factors cannot be justified for most bank protection projects. Consequently, several more empirical methods have evolved, as follows:

1. It is assumed that scour will occur below the existing bed. The maximum depth of scour is assumed to vary with stream size and is estimated on the basis of past experience with the same or similar streams.
2. Scour depth data are collected and plotted against pertinent cross-sectional and planimetric variables, as shown in Fig. B-6 (Maynord 1996). The vertical axis of this plot shows the ratio of the maximum *total* depth in the bend to the average water depth in the approach channel. As shown in Fig. B-6, the aspect ratio (water surface width/average channel depth) has a significant effect on scour depth.
3. Blench (1969) presented a "regime" method for estimating maximum scour depth in alluvial channels. The method is based on estimating the unit discharge adjacent to the bank and then calculating a regime depth, which is a function of the unit q and a "zero bed factor." The zero bed factor is a function of the median diameter of the bed material and is defined by a curve presented by Blench. The maximum scoured depth is determined by multiplying the regime depth by a "z factor" that varies from 1.5 to 2.75 depending on the geometry of the problem, such as flow parallel to a bank or at right angles. Additional information on the Blench method is given in Neill (1973).

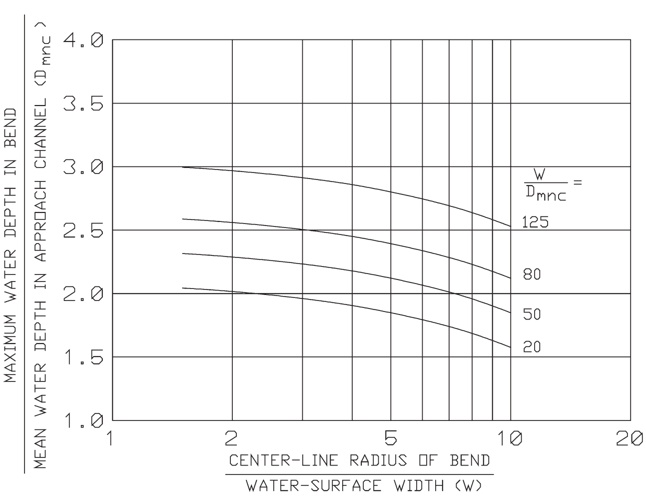

Fig. B-6. Bend depth/average depth versus centerline radius/water surface width. Reprinted by permission from Maynord (1996).

Zimmerman (1997) states that analytical methods for scour depth estimation (such as Zimmermann and Kennedy 1978) are preferable to the empirical method embodied in Fig. B-6.

B.7.2 Requirements for Toe Scour Protection

Two basic design alternatives are available for toe scour protection. The first is to extend the protection into the bed to the depth of maximum scour. This is relatively easy where construction is in the dry, but is difficult and expensive under water. The second alternative is to place some form of protection on the existing bed that will adjust to the maximum scour; suitable systems include riprap toe sections, gabion mattresses, and cabletied concrete blocks.

Scour below a riprap toe section (Fig. B-7) causes the riprap to "launch" and protect the exposed material below. A schematic of the toe-launching process is shown in Fig. B-7. According to USACE (1994), launchable riprap should have $D_{85}/D_{15} > 2$ to prevent large interstitial spaces after launching. With recommended toe shapes, riprap launches on noncohesive sediments at a slope of about 1V:2H. The riprap thickness H_B before launching governs the rate at which rock is launched. If it is too thick, rock will be released at too high a rate and wasted in the process. If it is too thin, rock will be released too slowly and coverage of the slope will be sparse. The recommended thickness H_B (Fig. B-7 is 2.5 to 4.0 T with an optimum value of 3T, where T is the normal revetment thickness on the slope. The volume of riprap required per unit length of bankline is

$$\text{VOLUME/LENGTH} = T \times \text{SCOUR DEPTH} \times \sqrt{5} \quad \text{(B-3)}$$

The $\sqrt{5}$ converts the scour depth to a slope length for volume computation, assuming that the rock launches at a IV:2H slope. To account for stone lost during launching and

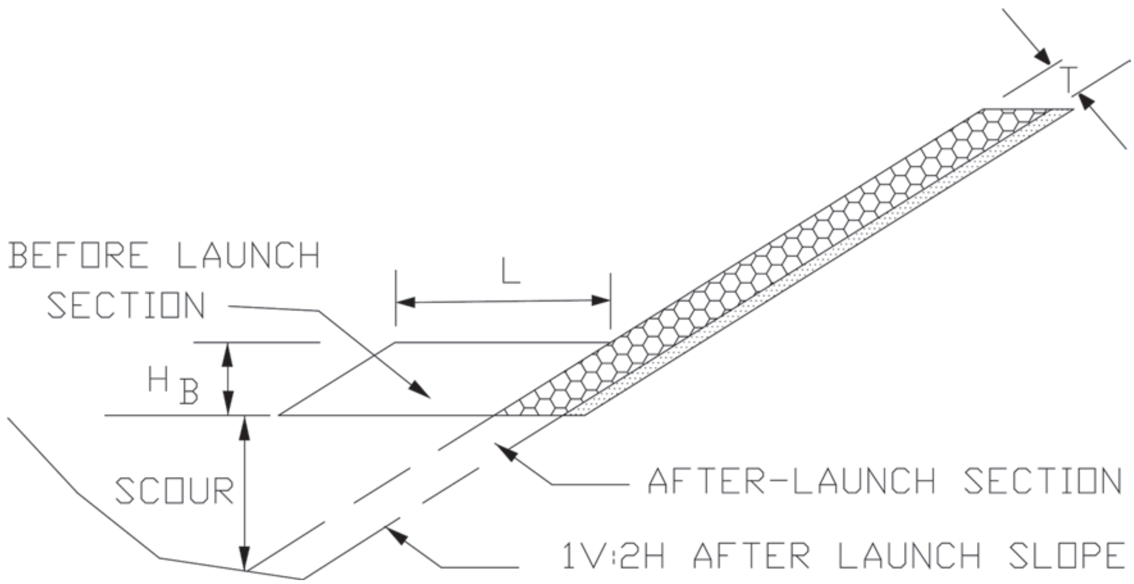

Fig. B-7. Launched stone schematic. Before launch section can be placed above, on, or below streamed. Developed by author.

for the uncertainty of placement underwater, USACE (1994) and CUR (1995) recommend increasing the volume given by Eq. (B-3) by 25 to 75%.

B.7.3 Scour and Protection at Revetment Ends and Tops

Scour at the downstream end of bank revetments presents a maintenance problem for many installations. In some cases, the protection was not carried far enough downstream, so that the unprotected banks is still subject to high velocities. This situation can be resolved only by extending the revetment farther downstream. In other cases, a reduction in hydraulic roughness from the riprap to the natural bank results in separation eddies at the downstream end of the revetment. As erosion progresses, the eddy strength grows, leading to faster erosion.

One technique to reduce eddy action is to terminate the revetment gradually by inclining its downstream end, so that the toe terminus is located three to four bank heights downstream of the top-of-bank terminus.

For both the ends and the top of the revetment, scour protection can be provided by placing a large thickness of riprap at the end or top. This technique uses the launching process to prevent undermining. Various designs of riprap end sections are presented in USACE (1994).

B.8 SIZE REQUIREMENTS FOR RIPRAP

B.8.1 Forces on Riprap

The key displacing forces on riprap particles on a level bed are instantaneous peaks of lift and drag, which are related to near-bed instantaneous velocity components. The submerged weight is usually the main resisting force, but additional resistance may result from contact with adjacent particles. On side slope particles, the downslope component of weight also contributes to instability, but in compensation there may be greater inter-particle resistance.

Figure 2-16 shows the various forces acting on a loose particle. Equation 2-53 shows a theoretical balance of forces at first displacement, when the boundary shear stress equals the resisting force due to submerged weight. Lane (1955), Stevens et al. (1976), Ikeda (1982), and Christensen (1995) have all presented moment-balance equations. Some formulations introduce lift as well as drag and use different assumptions for side slopes and channel beds. Lane's equation, although it ignores lift, is widely used to derive the ratio K_{sl} of critical shear stress on a side slope to critical shear stress on a level bed:

$$K_{sl} = \frac{\tau_s}{\tau_b} = \sqrt{1 - \frac{\sin^2\alpha}{\sin^2\phi}} = \cos\alpha\sqrt{1 - \frac{\tan^2\alpha}{\tan^2\phi}} \qquad \text{(B-4)}$$

where

τ_s = critical shear stress on side slope;
τ_b = critical shear stress on bed;
α = angle of side slope to the horizontal;
ϕ = angle of repose of riprap material.

Equation (B-4) is plotted in Fig. B-8 using an angle of repose of 40°, which is commonly assumed for riprap.

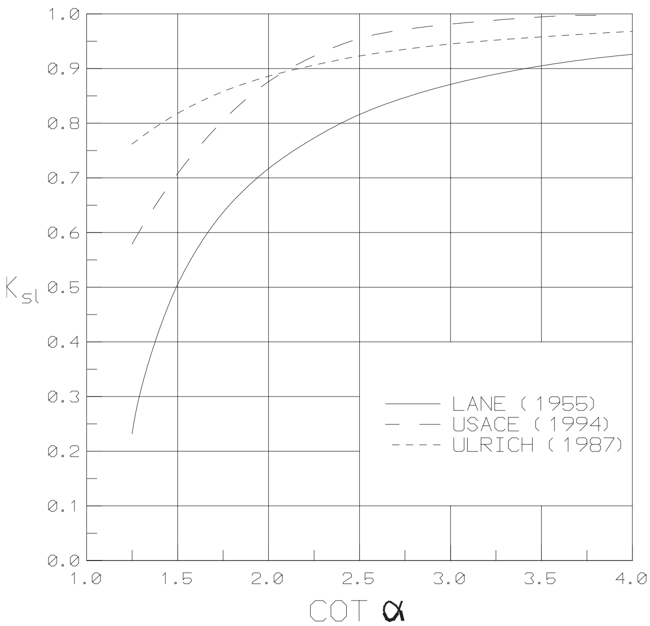

Fig. B-8. Ratio of critical side slope to bed shear stress versus cotangent of side slope angle. Adopted from Lane (1955), USACE (1994), and Ulrich (1987).

Maynord (1988) conducted flume tests of riprap on level beds and on side slopes ranging from 1V:1.25H to 1V:4H. Measured near-bed velocities were used to compute the bed shear stress at failure. The best-fit curve of the data, shown in Fig. B-8 as the USACE (1994) curve, indicates that the Lane equation is conservative. For riprap on side slopes, Ulrich (1987) recommended using an angle of 75° instead of the actual repose angle in the Lane equation—the corresponding curve is also plotted in Fig. B-8: The Froehlich and Benson (1996) expression for *K,* using 75° instead of angle of repose, produces the same results as the Ulrich curve in Fig. B-8. Experimental results for sand-size particles in an air tunnel by Ikeda (1982) show that "the effects of lift force seem to be negligible in describing the most probable values of critical tractive forces of both level and laterally sloping boundaries." On the other hand, some reformulations of moment-balance equations to include lift suggest that the Lane equation is nonconservative.

Fig. B-8 indicates that the effects of side slope on riprap stability are relatively small except when the slope angle increases to near the angle of repose. Correspondingly, riprap design procedures presented by Hughes et al. (1983) and by Escarameia and May (1992) do not change rock size for side slopes of 1V:2H or flatter. Also, wave riprap equations by Hudson (1958), based on stability tests, are less sensitivity to slope angle than the Lane (1955) equation with a repose angle of 40°.

A reason that some theoretical formulations of riprap stability may be overconservative is that a steady gravity force is used in conjunction with time-averaged values of hydraulic forces. In fact, however, riprap displacement may be sensitive to time-peak values of hydraulic forces, which can be several times higher than their time-averaged values. The result is to exaggerate the relative influence of downslope gravity forces for riprap on side slopes—at least when the slope is notably flatter than the angle of repose.

B.8.2 Design for Maximum Force

Riprap should be designed for whatever hydraulic conditions determine the largest rock size. These conditions do not necessarily corespond to the design or maximum discharge—in some cases bank-full conditions may be critical.

In bendways, riprap sizing is normally based on the most severe flow conditions found along the bend. If the approach alignment is stable, bendway revetments could be designed with variable sizing along the length of the bend, and possibly up and down the slope. Such multiple gradations have not been common practice, however, usually because of cost and inspection difficulties in projects of limited scale.

B.8.3 Effects of Velocity Profile and Turbulence on Stability

Two secondary factors that affect stone size are (1) the development and form of the velocity profile and (2) the turbulence characteristics of the flow:

- Velocity profile development refers to how well the velocity distribution has adjusted to the boundary roughness at the point of interest. At many riprap design locations, the velocity profile has not fully adjusted from the upstream channel boundary roughness to the greater roughness of the riprap. Also, in bendways, the flow may be accelerating up to the point of most severe attack, hindering normal boundary layer development. For the same depth-averaged velocity, near-bed velocity and bed shear stress will generally be greater if boundary layer development is incomplete. The vertical velocity profile can also be distorted in bendways, where secondary currents move the point of maximum velocity closer to the bed.
- Intensity and scale of turbulence can also affect required riprap size. Flow exiting from energy dissipators is a common case. Other examples are flow exiting from propellers and jets; downstream of dikes and groins; sharp bends with low radius/width ratios; bridge piers and abutments; and transitions in roughness or channel dimensions, particularly width expansions. In such high-turbulence areas, the ratios of peak to time-averaged values of velocity and shear stress may be much higher than in straight channels of uniform cross section. Escarameia (1998) presents a riprap sizing design procedure in which turbulence intensity—but not scale—is included.

B.8.4 Characteristic Size and Gradation Effects

In many studies and design procedures, the median riprap size D_{50} has been used to characterize the stability of a mixture. Many design procedures either use D_{50} for all types of gradation, or else restrict the procedure to a recommended range of gradations.

Other characteristic measures have also been used. USACE (1994) used D_{30}, assuming a thickness of $1 \times D_{100}$. The California Division of Highways (CDH 1970) used a characteristic weight W_{33}, than which 33% by weight of the mixture is lighter. Shen and Lu (1983) used D_{30} for armor layers. (In sediment-transport studies, Einstein (1950) used D_{35} and Ackers and White (1973) used D_{30}.)

Anderson et al. (1970) found in flume studies that uniform riprap was more stable than graded riprap of the same D_{50}. This conclusion was confirmed in flume studies by Abt et al. (1988), Ahmed (1988), and Maynord (1988). In their sizing equation, Abt et al. (1988) retain D_{50} as the characteristic size but add a gradation coefficient.

B.8.5 Riprap Sizing Methods

Some methods such as those of CUR (1995) and Escarameia (1998) address riprap sizing for a range of applications. Other methods address specific applications only. The following sections describe various riprap sizing methods for different river applications.

The basic form of many sizing equations is given by

$$\frac{V_c}{\left[g\left(S_g - 1\right)D_c\right]^{1/2}} = C_{\text{ref}}\left(\frac{h}{D_c}\right)^{P_{\text{ref}}} \tag{B-5}$$

where

V_c = characteristic velocity (which may be near-bed velocity, depth-averaged velocity, or cross-sectional average velocity)
S_g = specific gravity of riprap stone = ρ_s/ρ,

where

ρ_s = stone density and
ρ = water density;
C_{ref} = a numerical coefficient, usually based on experimental data;
h = local water depth;
D_c = characteristic particle size (D_{30} to D_{90} depending on investigator);
P_{ref} = an exponent dependent on the hydraulic environment and the way the characteristic velocity V_c is defined.

P_{ref} generally varies from 0 to 0.167. For problems having little or no boundary layer development, such as below energy dissipators or within propeller jets, P_{ref} is often taken as zero, which means no dependency on depth. Some investigators suggest $P_{\text{ref}} = 0$ even for bank protection. For a completely developed boundary layer, $P_{\text{ref}} = 0.167$ when using the Manning-Strickler resistance equation.

In the following subsections, riprap sizing equations are presented in the form recommended by their authors. Values of P_{ref} and C_{ref} are quoted to show how each equation relates to the basic form of Eq. (B-5).

B.8.5.1 Bank and Bed Revetments (Not around Hydraulic Structures)

B.8.5.1.1 CUR Manual CUR (1995) present an equation developed by Pilarczyk (1990) for stability under current attack:

$$\Delta D_n = \phi_c k_t \frac{0.035}{\psi_{cr}} k_h k_{sl}^{-1} \frac{V^2}{2g} \tag{B-6}$$

where

Δ = relative submerged density of stone = $(\rho_s - \rho)/\rho$;
D_n = characteristic stone size = $0.85\ D_{50}$;
ϕ_c = geometry correction factor to account for edges or transitions: 0.75 for continuous protection, 1.0 to 1.5 for edges and transitions, 1.5 for exposed rock on a sill;
k_t = turbulence factor: 1.0 for normal turbulence in rivers; 1.5 for increased turbulence as in bends; 2.0 for high-turbulence hydraulic jumps, sharp bends, local disturbances; and 3.0 for propeller jets;
ψ_{cr} = Shields parameter = 0.035 for loose rock;
k_h = depth factor dependent on velocity profile: $2/(\log 10h/k_s)^2$ for fully developed boundary layer; $(h/D_n)^{-0.2}$ for a velocity profile not fully developed;
k_{sl} = slope factor, for flow along or down a slope; defined for flow along a slope by the Lane (1955) equation.

The left side of the Pilarczyk (1990) equation represents the resisting strength of the revetment and the right side the load or disturbing force.

In relation to the basic form of Eq. (B-5), the Pilarczyk equation varies P_{ref} and C_{ref} depending on the application. A specific form of the Pilarczyk equation is used in PIANC (1987a) and Hemphill and Bramley (1989).

B.8.5.1.2 California Bank and Shore Protection (CBSP) Manual The basic equation of the California Division of Highways (CDH 1970) for stable stone weight is

$$W_{33} = \frac{0.00002\ V_{\text{cbsp}}^6\ S_g}{(S_g - 1)^3 \sin^3(\beta - \alpha)} \tag{B-7}$$

where

V_{cbsp} = velocity in ft/sec, defined as 2/3 of average channel velocity in straight reaches, and 4/3 of average velocity on the outsides of bends;

β = angle for determining side slope effect, 70° for randomly placed rubble.

It is not clear how β relates to the more commonly used angle of repose. Other investigators also proposing a relatively large angle to address side slope effects are Froehlich and Benson (1996), who proposed a "particle angle of initial yield" of about 70–75° for typical gradations, and Ulrich (1987), who used a "bearing angle" of 75°.

In relation to the basic form of sizing equation (B-5), the CBSP equation has $P_{\text{ref}} = 0$ (no dependance on depth) and a variable C_{ref} depending on the application.

In a supplemental report, Racin (1996) explains that the CBSP approach emphasizes relatively uniform rock placed in two or more layers instead of graded rock and evaluates the CBSP procedure relative to field data and other procedures. The CBSP manual also provides guidance for shape, durability, specific gravity, layer thickness, gradation, filter requirements, and placement methods.

B.8.5.1.3 Wallingford Design Manual for River and Channel Revetments Escarameia and May (1992) provide a general equation for riprap, loose or interlocking concrete blocks, and gabion mattresses,

$$D_{50} = C\frac{U_b^2}{2g(S_g - 1)} \tag{B-8}$$

where

D_{50} = characteristic particle size, related to weight on the basis of a cubical shape;
C = coefficient dependent on turbulence intensity $= 12.3\text{TI} - 0.2$ for riprap bank or bed protection on side slopes of 1V:2H or flatter;
TI = turbulence intensity at 10% of flow depth above the bed;
U_b = velocity at 10% of flow depth above the bed $= (1.04 - 1.48\text{TI})V$.

The incorporation of turbulence intensity TI is a unique feature of this equation.

For specific application to bed and bank protection, Escarameia and May (1992) present an equation using local depth-averaged velocity, applicable to bank slopes of 1V:2H or flatter and normal river flow:

$$D_{50} = C\frac{V^2}{S_g - 1} \tag{B-9}$$

where $C = 0.05$ for continuous revetments, and 0.064 for the edges of revetments. For bank protection, V should be measured at the toe of the slope.

Relative to the basic form of sizing equation (B-5), the Escarameia and May equations have $P_{\text{ref}} = 0$ (no dependance on depth) and a variable C_{ref} depending on the application. The general Eq. (B-8) uses bottom velocity, and the specific riprap Eq. (B-9) uses depth-averaged velocity.

B.8.5.1.4 FHWA Manual—Design of Riprap Revetment (HEC-11) Brown and Clyde (1989) combine the Manning-Strickler equation with the Shields relation to produce an equation for stable rock size. A similar approach was used earlier by Straub (1953); Grace et al. (1973); and Reese (1984). The Brown and Clyde equation is

$$D_{50} = C_{sg}C_{sf}\frac{0.001\,V_a^3}{h_{\text{avg}}^{0.5}\,K_{sl}^{1.5}} \tag{B-10}$$

where h_{avg} is the average depth in the main channel, $C_{sg} = 2.12/(S_g - 1)^{1.5}$, and $C_{sf} = (\text{SF}/1.2)^{1.5}$. SF is a stability factor dependent on the ratio R/W of radius of curvature to channel width. For $R/W > 30$, SF = 1.2; for $R/W = 10$ to 30, SF = 1.3 to 1.6; and for $R/W < 10$, SF = 1.7.

Relative to the basic form of sizing equation presented in Section B.8.5, the Brown and Clyde equation has $P_{\text{ref}} = 1/6$ (generally applicable to complete boundary layer development) and a variable C_{ref} depending on the application.

B.8.5.1.5 Safety Factor Methods Most of the methods reviewed above address riprap stability in an overall sense, without considering in detail the stability of individual particles. The methods reviewed below involve more detailed consideration of forces and moments on an individual particle, including lift—which does not appear in the classical Lane (1955) equation.

Stevens et al. (1976) developed a safety factor method based on the hypothesis that a particle is stable if the sum of the moments acting to displace it is less than the moment of its submerged weight.

Wittler and Abt (1988) modified Stevens's analysis to add contact and frictional forces from adjacent particles; they tested their equation against stability data for flow down slopes of 2 to 20% and found good agreement, but did not test it for flow along a side slope.

Ahmed (1988) compared seven safety factor methods (including two of his own but not the Wittler and Abt modification) against flume data for flow along a riprapped 1V:1.5H side slope, and found that all appeared to underestimate stability significantly. He reported that non-safety-factor approaches by Anderson et al. (1970) and by the California Division of Highways (CDH 1970) gave better results.

B.8.5.1.6 Corps of Engineers Manual—Hydraulic Design of Flood Control Channels The USACE (1994) method is intended for sizing riprap in rivers and channels, except immediately downstream of hydraulic structures that create highly turbulent flow. Source data were limited to slopes of 2% or less and to values of D_{30}/d exceeding 0.02. The sizing equation is

$$D_{30} = S_f\,C_s\,C_V C_T h\left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right)^{1/2}\frac{V}{\sqrt{K_{slcoe}\,gh}}\right]^{2.5} \tag{B-11}$$

where

D_{30} = riprap size for which 30% by weight is finer.
S_f = safety factor, minimum = 1.1.
C_s = stability coefficient for incipient failure = 0.30 for angular rock or 0.375 for rounded rock. A revetment thickness of D_{100} or 1.5 D_{50}, whichever is greater, is assumed, and a gradation factor D_{85}/D_{15} in the range of 1.7 to 5.2.
C_V = vertical velocity distribution coefficient = 1.0 for straight channels or inside of bends; 1.25 downstream of concrete channels or at the ends of dikes; and $1.283-0.2\log(R/W)$ for outsides of bends.
C_T = thickness coefficient = 1.0 for a thickness of D_{100}, with smaller values for greater thickness depending on D_{85}/D_{15}.
h = local depth, defined for side slope riprap at a point 20% upslope from toe for slope.
γ_w = unit weight of water.
γ_s = unit weight of stone.
V = local depth-averaged velocity, symbolized as V_{ss} for bank protection riprap and defined at depth-average at a point 20% upslope from the toe.
K_{slcoe} = side slope correction factor = $-0.672 + 1.492 \cot(\alpha) - 0.449 \cot^2(\alpha) + 0.045 \cot^3(\alpha)$ where α is angle to the horizontal.

Equations of basically similar form but without most of the modifying factors were presented by Neill (1967); Bogardi (1978); and Pilarczyk (1990).

In the USACE equation, an incipient failure criterion is used to determine the stability coefficient, and defined as the condition when the fabric or bank material beneath the riprap is first exposed. Incipient failure was used instead of incipient motion or displacement, to cover a wide range of gradations and to allow for the effects of blanket thickness.

Significant differences between the USACE method and others are the use of D_{30} as characteristic size, an empirical relation rather than the Lane equation to account for side slope, a coefficient for thickness, and the provision of guidance for determining the near-bank velocity V_{ss} (Fig. B-4). In relation to the basic form of stone sizing equation (B-5), the USACE equation implies $P_{ref} = 0.1$ (intermediate between zero and complete boundary layer development), and a variable C_{ref} depending on the application.

B.8.5.1.7 Method Based on Field Data Blodgett and McConaughy (1986) developed a method based on analysis of field riprap stability data and presented the equation

$$D_{50} = 0.01\, V_a^{2.44} \tag{B-12}$$

This relationship is intended for straight and curved channels having side slopes of 1V:1.5H or flatter. The Blodgett and McConaughy report also addresses other factors important in riprap design. Equation (B-12) does not fit the standard form of the stone-sizing equation.

B.8.5.1.8 Probabilistic Methods Probability-based methods for design of riprap against currents have been presented by Li et al. (1976); PIANC (1987b); and Froehlich and Benson (1996). With increasing emphasis on risk-based design procedures, probability-based methods will see increased usage. One of their advantages lies in the ability to combine effects from different mechanisms, such as waves and currents. Although the probability of the hydraulic forces has been the focus of most probabilistic methods, the uncertainties in estimates of stone size, stone density, channel depth, etc. will also have to be addressed in a risk-based procedure.

B.8.5.2 Riprap Sizing for Steep Slopes Olivier (1967) developed an equation for stone size on steep slopes, specifically the downstream faces of through-and-overflow rockfill dams. By including *g,* the equation can be written in dimensionally homogeneous form as

$$D_{50} = \frac{5.63\, q^{2/3}\, S^{7/9}}{(S_g - 1)^{10/9}\, g^{1/3}} \tag{B-13}$$

where

q = unit discharge and
S = slope.

The equation is based on data for crushed rock on slopes from 8 to 45%. In relation to the basic form of stone-sizing equation (B-5), the Olivier equation has $P_{ref} = 1/6$ and a variable C_{ref} depending on the application.

Knauss (1979) evaluated loose rock on slopes as steep as 67%. He reports that the Olivier equation is conservative for slopes steeper than about 20%, because of air entrainment that develops on steeper slopes. For slopes from about 20 to 67%, a stone specific gravity of 2.7, and typical placement methods, Knauss recommends the equation

$$D_{50} = \frac{q^{2/3}}{g^{1/3}[2.4 - 3\sin(\eta)]^{2/3}} \tag{B-14}$$

where η is the slope expressed as the angle to the horizontal.

Abt and Johnson (1991) conducted large-scale flume studies on slopes from 1 to 20%, and presented the empirical relation

$$D_{50} = 5.23\, S^{0.43}\, q^{0.56} \tag{B-15}$$

where D_{50} is in inches and q is in cfs/ft. Other variables incorporated into their design procedure include (1) definitions of conditions at both first movement and at complete failure, (2) channelization, gradation, and thickness effects, and (3) rounded versus angular stone shape.

Design methods for riprap on steep slopes have also been presented by Stephenson (1979); USACE (1994); Robinson et al. (1997); and Chang (1998). Some of these methods should be used with caution outside the range of data used in their development, because of dimensional inconsistency in the presented relationships.

B.8.5.3 Riprap around Hydraulic Structures Most equations for sizing riprap around hydraulic structures use some form of the Isbash (1935) equation. For riprap below energy dissipators, the USACE (1987; 1990b) uses the equation

$$D_{50} = \frac{V_a^2}{C^2 2g\left(\dfrac{\gamma_s - \gamma_w}{\gamma_w}\right)} \qquad \text{(B-16)}$$

where

$C = 0.86$ for high turbulence (applicable to stilling basins) and
$C = 1.2$ for low turbulence (applicable to bank protection).

Average velocity over the end sill is used in the equation, except for structures where only one gate might be opened—see USACE (1987). In relation to the basic form of stone-sizing equation (B-5), the USACE equation has $P_{\text{ref}} = 0$ (no dependence on depth) and a variable C_{ref} depending on the application.

Peterka (1963) presented a riprap-sizing curve used by the Bureau of Reclamation for riprap below energy dissipators. The curve gives results similar to those for the Isbash equation with $C = 0.88$ and with D_{40} as the characteristic size. CUR (1995) presents the Pilarczyk (1990) equation discussed in Section B.8.5.1 for areas below energy dissipators.

Guidance references for riprap protection at highway drainage culverts include the FHWA's HEC-14 (Corry et al. 1983), and Shafei-Bajestan and Albertson (1993).

B.8.5.4 River Closures The study by Isbash (1935) represents the most widely known investigation of riprap structures for river closure. Further detailed guidance for closure structures can be found in CUR (1995) and Thorne et al. (1995a).

B.8.5.5 Propeller Jets Most equations for sizing riprap from protection against scour from propeller jets use some form of the Isbash (1935) equation. References providing guidance for near-bed velocity and stone sizing include Blaauw and van de Kaa (1978); Fuehrer et al. (1981); Bergh and Magnusson (1987); and PIANC (1987a). Because of wide variations in the definition of near-bed velocity, it is recommended that the near-bed velocity and stone sizing both be determined by a single investigator.

B.8.5.6 Waves (Including Boat Waves) Riprap sizing for wave conditions is not generally covered in this appendix. References providing detailed guidance include USACE (1984a); PIANC (1987a); Hemphill and Bramley (1989); CUR (1995); Thorne et al. (1995a); and Escarameia (1998).

B.8.5.7 Combined Action of Waves and Currents Revetments sometimes undergo attack from both currents and waves. The combined action is complex, and a probability approach should be considered, because the probability of simultaneous maximum forces from the two sources is often quite small. As a rough design approach, Escarameia (1998) suggests that the stone sizes determined separately for waves and currents be added together. This approximation seems rather conservative: at the waterline, where wave forces tend to be highest, current forces are often well below the maximum which occurs farther down the slope. Additional guidance for combined wave-current action may be found in CUR (1995).

B.8.6 Ice and Debris Effects on Riprap Stability

Potential ice and debris effects on riprap stability are generally addressed by increasing size and/or thickness. The USACE (1994) suggests increasing thickness by 150 to 300 mm (6 to 12 inc.) when heavy debris is present, and increasing rock size in proportion to the increased thickness.

Sodhi et al. (1996) summarize earlier guidance on ice and present results from flume tests. Ice can damage riprap slope protection by shoving action, or by plucking when water and ice levels rise. To avoid shoving damage, the maximum riprap size should be two times the ice thickness for 1V:3H side slopes, and three times the ice thickness for 1V:1.5H side slopes. To avoid plucking damage, D_{50} should be greater than the maximum winter ice thickness. Additional guidance is given by Wuebben (1995).

B.9 CONSTRUCTION AND MAINTENANCE

As stated by Thorne et al. (1995a), constructibility and maintainability constitute one of five key factors in a successful riprap project. Some related considerations are as follows:

1. Cost of transportation is often a major portion of the total cost.
2. Excessive stockpiling and handling can lead to size degradation unless rock quality is high.
3. During placement, dumping and spreading may promote size segregation and breakage. Rock should be released from the equipment as close as possible to its final position.
4. In the case of underwater placement, rock tends to move in a downstream direction—see Prezedwojski et al. (1995). It also tends to disperse—see Hunt (1998).

5. A large, heavy plate is sometimes recommended to compact the riprap and thereby increase its stability (Maynord 1992; Simons 1995).
6. Riprap should be inspected annually and after each significant flood event.

Additional guidance on construction and maintenance can be found in USACE (1990ab); CUR (1995); and Escarameia (1998).

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