Bridge Scour and Stream Instability Countermeasures

HEC 23
July 1997

Welcome to HEC 23-Bridge Scour and Stream Instability Countermeasures

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Introduction

Bendway weirs, also referred to as stream barbs, bank barbs, and reverse sills, are low elevation stone sills used to improve lateral stream stability and flow alignment problems at river bends and highway crossings. Bendway weirs are used for improving inadequate navigation channel width at bends on large navigable rivers. They are used more often for bankline protection on streams and smaller rivers. The stream barb concept was first introduced in the Soil Conservation Service (now the Natural Resource Conservation Service, NRCS) by Donald Reichmuth (1993) who has applied these rock structures in many streams in the western United States.

The U.S. Army Corps of Engineers Waterways Experiment Station (WES) developed a physical model to investigate the bendway weir concept in 1988 (Prokrefke 1995). Since then WES has conducted 11 physical model studies on the use of bendway weirs to improve deep and shallow-draft navigation, align currents through highway bridges, divert sediment, and protect docking facilities. WES has installed bendway weirs to protect eroding banklines on bends of Harland Creek near Tchula, Mississippi. The U.S. Army Corps of Engineers, Omaha District, has used bendway weirs on the Missouri River in eastern Montana. The Missouri River Division (MRD) Mead Hydraulic Laboratory has also conducted significant research and testing of underwater sills. Bendway weirs are a relatively new river training structure and research is providing useful information on their use and effectiveness.

Design Concept

Bendway weirs are similar in appearance to stone spurs, but have significant functional differences. Spurs are typically visible above the flow line and are designed so that flow is either diverted ***around*** the structure, or flow along the bank line is reduced as it passes ***through*** the structure. Bendway weirs are normally not visible, especially at stages above low water, and are intended to redirect flow by utilizing weir hydraulics ***over*** the structure. Flow passing over the bendway weir is redirected such that it flows perpendicular to the axis of the weir and is directed towards the channel centerline. Similar to stone spurs, bendway weirs reduce near bank velocities, reduce the concentration of currents on the outer bank, and can produce a better alignment of flow through the bend and downstream crossing. Experience with bendway weirs has indicated that the structures do not perform well in degrading or sediment deficient reaches.

Bendway weirs have been constructed from stone, tree trunks, and grout filled bags and tubes. Design guidance for bendway weirs has been provided by the U.S. Army Corps of Engineers, Omaha District, WES, and the NRCS. The following geometric design guidelines for stone bendway weirs reflect guidance provided by LaGrone (1996), Saele (1994) and Derrick (1994 and 1996). The formulas provided by LaGrone were developed to consolidate many of the "rules of thumb" that currently exist in the field. The formulas are not based on exhaustive research, but appear to match well to current practices. Installation examples were provided by Colorado Department of Transportation (CDOT) and Washington State Department of Transportation (WSDOT).

Design Guidelines

1. HEIGHT - The height of the weirs, H, is determined by analyzing the depth of flow at the project site (Refer to **Figures 1.1 and 1.2**). The bendway weir should be between 30 to 50 percent of the depth at the mean annual high water level. The height of the structure should also be below the normal or seasonal mean water level and should be equal to or above the mean low water level. The weir must be of adequate height to intercept a large enough percentage of the flow to produce the desired results. For applications relating to improved navigation width, the weir must be at an elevation low enough to allow normal river traffic to pass over the weir unimpeded.

2. ANGLE - The angle of projection, θ , between the bendway weir axis and the upstream bankline tangent typically ranges from 50 to 85 degrees. Experience has indicated that it is easier to measure this angle from the chord between two weirs in the field rather than using the bankline tangent. The chord is drawn from the points of intersection with the weirs and the bankline (see Figure 1.1). The angle of projection is determined by the location of the weir in the bend and the angle at which the flow lines approach the structure. Ideally, the angle should be such that the high-flow streamline angle of attack is not greater than 30 degrees and the low flow streamline angle of attack is not less than 15 degrees to the normal of the weir centerline of the first several weirs. If the angle of flow approaching the upstream weirs is close to head-on, then the weir will be ineffective and act as a flow divider and bank scalloping can result. If the angle of flow approaching the upstream weirs is too large then the weir will not be able to effectively redirect the flow to the desired flow path. Ideally, the angle should be such that the perpendicular line from the midpoint of an upstream weir points to the midpoint of the following downstream weir. All other factors being equal, smaller projection angles, θ , would need to be applied to bends with smaller radii of curvature to meet this criteria and vice versa. Experiments by Derrick (1994) resulted in a weir angle of 60 degrees being the most effective for the desired results in a physical model of a reach on the Mississippi River. Observations by LaGrone indicate that the angle, θ , of the upstream face of the structure is most important in redirecting flows. The upstream face should be a well defined straight line at a consistent angle.

3. CROSS SECTION - The transverse slope along the centerline of the weir is intended to be flat or nearly flat and should be no steeper than 1V:5H. The flat weir section normally transitions into the bank on a slope of 1V:1.5H to 1V:2H. The structure height at the bankline should equal the height of the maximum design high water. This level is designed using sound engineering judgment. The key must be high enough to prevent flow from flanking the structure. The bendway weir should also be keyed into the stream bed a minimum depth approximately

equal to the D_{100} size.

4. LENGTH - The bendway weir length (L) should not exceed 1/3 the mean channel width (W). A weir length greater than 1/3 of the width of the channel can alter the channel patterns which can impact the opposite bankline. Weirs should be long enough to cross the stream thalweg. Weirs designed for bank protection need not exceed ¼ the channel width. A length of 1.5 to 2 times the distance from the bank to the thalweg has proven satisfactory on some bank stabilization projects. The length of the weir will affect the spacing between the weirs.

Maximum Length $L = W/3$ (typically: $W/10 < L < W/4$)

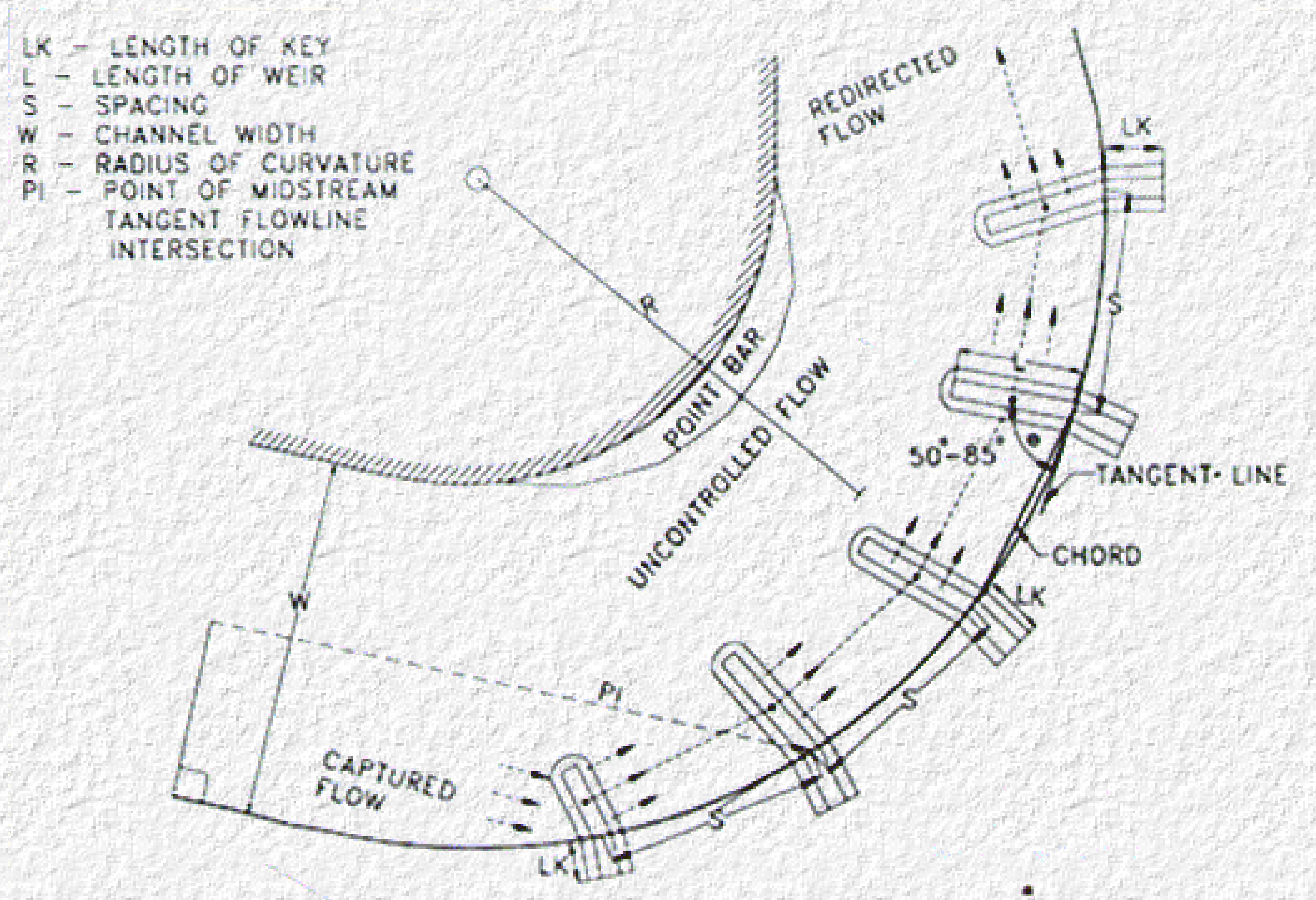


Figure 1.1 Bendway Weir Typical Plan View

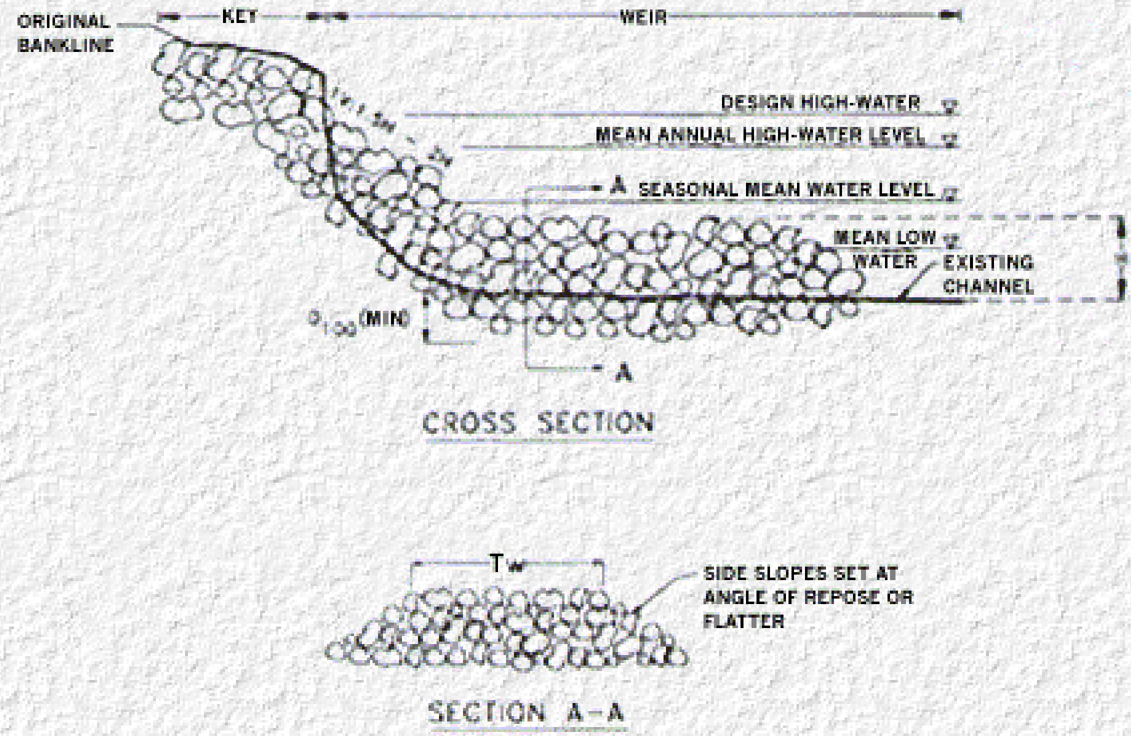


Figure 1.2 Bendway Weir Typical Cross Section

5. LOCATION - Ideally, a short weir should be placed a distance (S) upstream from the location where the midstream tangent flow line (midstream flow line located at the start of the curve) intersects the bankline (PI). Additional bendway weirs are then located based on the site conditions and sound engineering judgment. Typically, the weirs are evenly spaced a distance (S) apart.

6. SPACING - Bendway weir spacing is influence by several site conditions. The following guidance formulas are based on a cursory review of the tests completed by WES on bendway weirs and on tests completed by MRD on underwater sills. Based on the review, bendway weirs should be spaced similarly to hardpoints and spurs. Weir spacing is dependent on the streamflow leaving the weir and its intersection with the downstream structure or bank. Weir spacing (S) is influenced by the length of the weir (L), and the ratios of weir length to channel width (W) and channel radius of curvature (R) to channel width. Spacing can be computed based on the following guidance formulas:

$$S = 1.5L\left(\frac{R}{W}\right)^{0.8}\left(\frac{L}{W}\right)^{0.3}$$ (LaGrone 1995)

$S = (4 \text{ to } 5)L$ (Saele 1994)

Maximum Spacing (S_{max}) is based on the intersection of the tangent flow line with the bankline assuming a simple curve. The maximum spacing is not recommended, but is a reference for designers. In situations where some erosion between weirs can be tolerated, the spacing may be set between the recommended and the maximum.

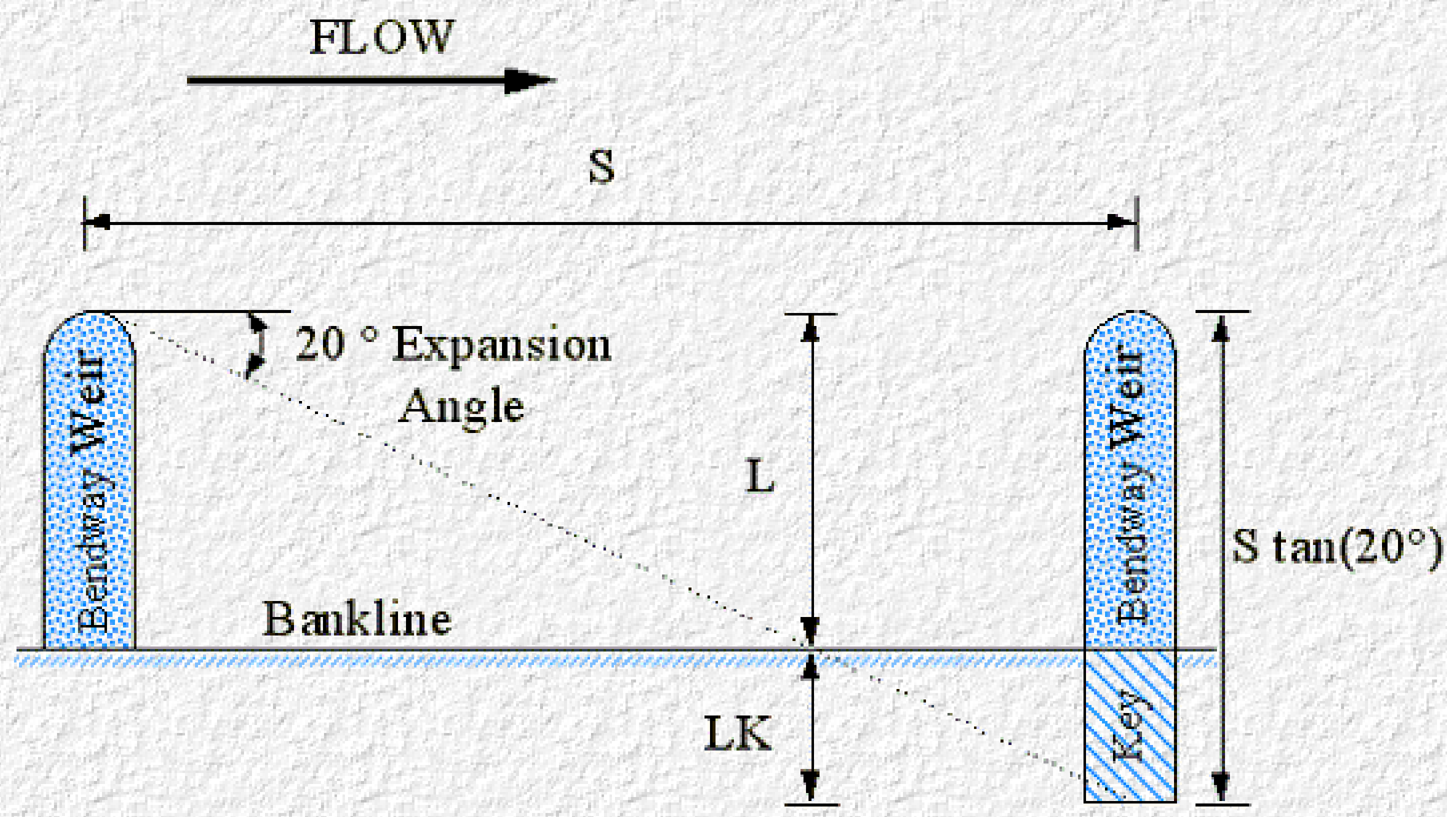
$$S_{max} = R\left(1-\left(1-\frac{L}{R}\right)^{2}\right)^{0.5} \qquad \text{(LaGrone 1995)}$$

Results from the spacing formulas should be investigated to determine that the weir spacing, length, and angle will redirect the flow to the desired location. Streamlines entering and exiting the weirs should be analyzed and drawn in planform.

7. LENGTH OF KEY - Bendway weirs like all bankline protection structures should be keyed into the bankline to prevent flanking by the flow. Typically the key length (LK) is about half the length of the short weirs and about one fifth the length of the long weirs. Tests conducted by MRD found that lateral erosion between spurs on nearly straight reaches could be estimated by using a 20 degree angle of expansion (see **Figure 1.3**). The following guidance formulas for LK were therefore developed. These formulas compute minimum LK and should be extended in critical locations. The need for a filter between the weir key and the bank material should also be determined. Guidelines for the selection, design, and specification of filter materials can be found in Brown and Clyde (HEC-11) (1989) and Holtz et al. (FHWA HI-95-038) (1995).

When the channel radius of curvature is large (R > 5W) and S > L/tan(20°)

LK = S tan(20°) - L (LaGrone 1995)



$$LK = S \tan(20°) - L$$

Figure 1.3 Length of Key for Mild Bends

When the channel radius of curvature is small ($R < 5W$) and $S < L/\tan(20°)$

$$LK = \frac{L}{2}\left(\frac{W}{L}\right)^{0.3}\left(\frac{S}{R}\right)^{0.5} \qquad \text{(LaGrone 1995)}$$

NOTE: LK should not be less than 1.5 times the total bank height.

The NRCS guideline for length of key (LK) for short weirs or barbs (Saele 1994) is to key the barb into the bank a minimum distance of 2.4 m (8 ft) or 4 (D_{100}) whichever is greater.

8. TOP WIDTH - The top width of the weir may vary between 1 m and 4 m (3 and 12 ft), but should be no less than (2 to 3)*D_{100}. Weirs over 9 m (30 ft) in length will have to be built either from a barge or by driving equipment out on the structure during low flows. Structures built by driving equipment on the weir will need to be at least 3 to 5 m (10 to 15 ft) wide. Side slopes of the weirs can be set at the natural angle of repose of the construction material (1V:1.5H) or flatter.

9. NUMBER OF WEIRS - The smallest number of weirs necessary to accomplish the project purpose should be constructed. The length of the weirs and the spacing can be adjusted to meet this requirement. Typically, not less than three weirs are used together on unrevetted

banks.

10. CONSTRUCTION - Construction of the bendway weirs are typically conducted during low flow periods for the affected river. Construction methods will vary depending on the size of the river. Construction on larger rivers may be conducted using a barge which would allow the rock to be placed without disturbing the bankline. For rivers where a barge is not available and where the bendway weir is longer than 9 m (30 ft), access will need to be made from the bank and equipment may need to be driven out on the weir as it is being constructed.

Supplemental information on the use of bendway weirs on tight bends (small radius of curvature) and complex meanders can be found in LaGrone 1996.

Material Specifications

1. Stone should be angular, and not more than 30 percent of the stone should have a length exceeding 2.5 times its thickness.
2. No stone should be longer than 3.5 times its thickness.
3. Stone should be well graded but with only a limited amount of material less than half the median stone size. Since the stone will most often be placed in moving water, the smaller stone will be subject to displacement by the flow during installation.
4. Construction material should be quarry run stone or broken, clean concrete. High quality material is recommended for long-term performance.
5. Material sizing should be based on standard riprap sizing formulas for turbulent flow. Typically the size should be approximately 20% greater than that computed from nonturbulent riprap sizing formulas. The riprap D_{50} typically ranges between 300 mm and 910 mm (1 and 3 ft) and should be in the 45 kg to 450 kg (100 to 1,000 lb) range. The D_{100} rock size should be at least 3 times the calculated D_{50} size. The minimum rock size should not be less than the D_{100} of the streambed material.
6. Guidelines for the selection, design, and specification of filter materials can be found in Brown and Clyde (HEC-11) (1989) and Holtz et al. (FHWA HI-95-038) (1995).

Installation Examples

Some illustrations of bendway weirs in use are shown in Figures 1.4n1.7. Figures 1.4 and 1.5 show short bendway weirs shortly after installation by CDOT on the Blue River near Silverthorne, Colorado in February 1997. These weirs were designed with weir lengths of 3.5n6 meters at θ angles of 75° to the bankline tangent. The CDOT engineer indicated that adjustments in the field are equally as important and necessary as original design plans. It can be observed that the bendway weirs are being constructed at low flow conditions as discussed previously.

Figures 1.6 and 1.7 show bendway weirs installed by WSDOT on the Yakima River, Washington in 1994. Figure 1.6 shows the weirs at low flow conditions and Figure 1.7 shows the submerged weirs at normal to high flow conditions. Surface disturbances as flow passes over the weirs can be observed in Figure 1.7. These weirs were designed at θ angles of 50° to the bankline tangent to direct flow away from a critical pier at a bridge just downstream of this bend.

Figure 1.4 Bendway Weirs Installed on the Blue River Near Silverthorne, Colorado (CDOT)

Figure 1.5 Bendway Weirs Installed on the Blue River Near Silverthorne, Colorado (CDOT)

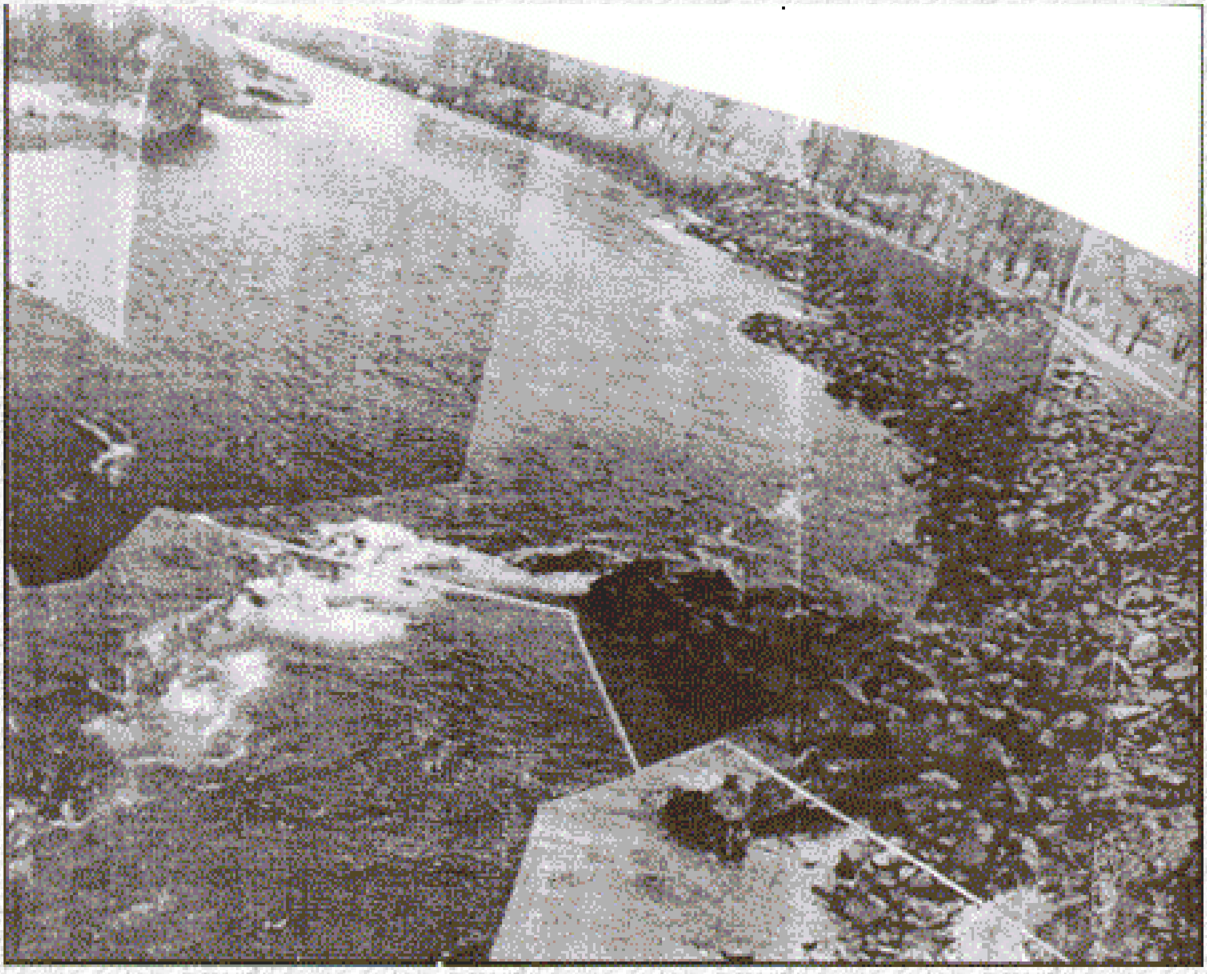


Figure 1.6 Bendway Weirs on the Yakima River, Washington at Low Flow (WSDOT)

Figure 1.7 Submerged Bendway Weirs on the Yakima River, Washington at High Flow (WSDOT)

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Go to Design Guideline 2

Design Guideline 2 : HEC 23
Soil Cement

Go to Design Guideline 3

Introduction

In areas where high quality rock is scarce, the use of soil cement can provide a practical countermeasure alternative for channel stability and scour protection. Soil cement has been used to construct drop structures and armor embankments, dikes, levees, channels, and coastal shorelines. Soil cement is frequently used in the southwestern United States because the limited supply of rock makes it impractical to use riprap for large channel protection projects.

Design Guidelines

The following design guidelines reflect guidance in information provided by the Pima County Department of Transportation in Tucson, Arizona and the Portland Cement Association. Typically, soil cement is constructed in a **stair-step** configuration by placing and compacting the soil cement in horizontal layers (**see Figure 2.1**). However, soil cement can be placed parallel to the face of an embankment slope rather than in horizontal layers. This technique is known as **plating**.

Figure 2.1 Stair Step Facing on Bonny Reservoir, Colorado after 30 years (PCA)

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1. Facing Dimensions for Slope Protection Using Stair-Step Method

In stair-step installations soil cement is typically placed in 2.4-m-wide horizontal layers. The width should provide sufficient working area to accommodate equipment. The relationship between the horizontal layer width (W), slope of facing (S), thickness of compacted horizontal layer (v), and minimum facing thickness measured normal to the slope (t_n) is quantified by the following equation and is shown graphically in **Figure 2.2**:

$$W = t_n \sqrt{S^2 + 1} + Sv \qquad \text{(PCA)}$$

As illustrated in **Figure 2.3**, for a working width of 2.4 m, a side slope of 1V:3H, and individual layers of 150 mm thick, the resulting minimum thickness of facing would be 620 mm measured normal to the slope. Bank stabilization along major rivers in Pima County, Arizona is constructed by using 150 mm lifts of soil cement that are 2.4 m in width and placed on a 1V:1H face slope.

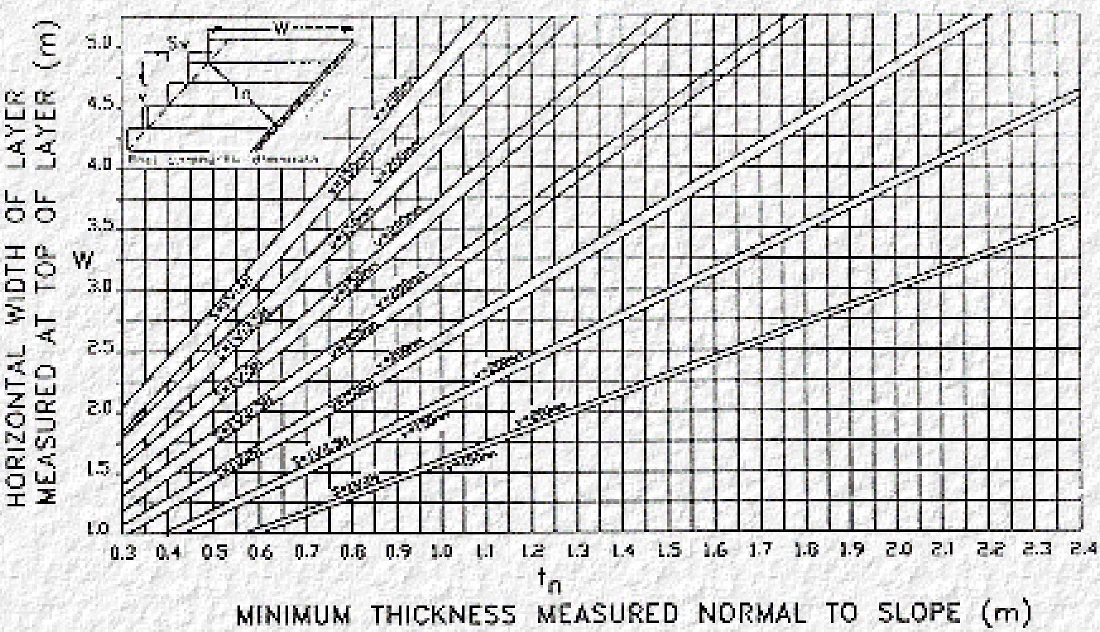

Figure 2.2 Relationship of Slope, Facing Thickness, and Horizontal Width of Soil Cement

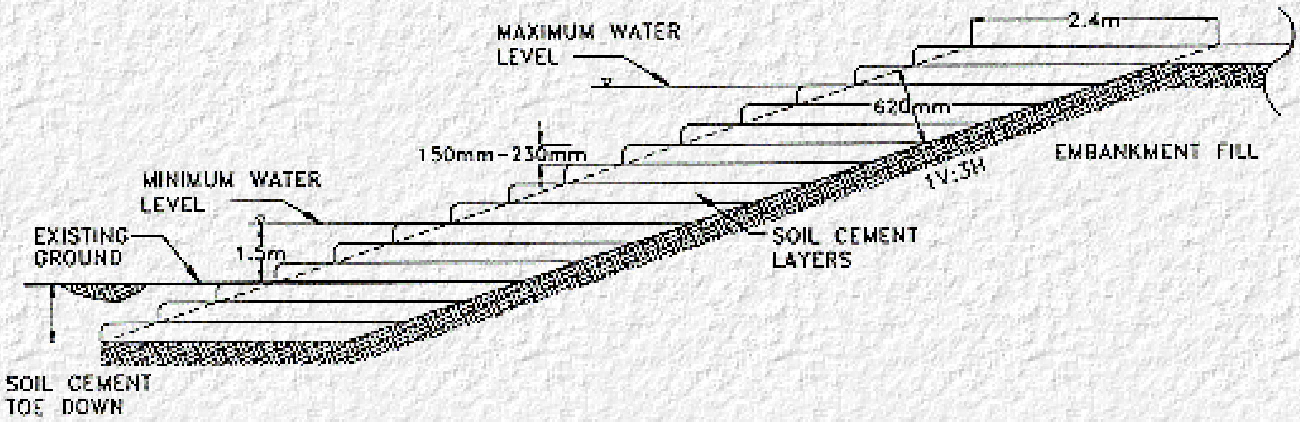

Figure 2.3 Typical Section for Soil Cement Slope Protection (Stair-step Method)

When horizontal layer widths do not provide adequate working widths, the stair-step layers can be sloped on a grade of 1V:8H or flatter toward the water line. Sloping the individual layers will provide a greater working surface without increasing the quantity of soil cement.

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2. Facing Dimensions for Slope Protection Using Plating Method

On smaller slope protection projects a single layer of soil cement can be placed parallel to the embankment. In this technique, known as plating, a single lift of soil cement is applied on slopes of 1V:3H or flatter (see **Figure 2.4**).

All extremities of the soil cement facing should be tied into nonerodible sections or abutments to prevent undermining of the rigid layer. Some common methods used to prevent undermining are placing a riprap apron at the toe of the facing, extending the installation below the anticipated scour depth or providing a cutoff wall below the anticipated scour depth.

As with any rigid revetment, hydrostatic pressure caused by moisture trapped in the embankment behind the soil cement facing is an important consideration. Designing the embankment so that its least permeable zone is immediately adjacent to the soil cement facing will reduce the amount of water allowed to seep into the embankment. Also, providing free drainage with weep holes behind and through the soil cement will reduce pressures which cause hydrostatic uplift.

Figure 2.4 Soil Cement Placed in the Plating Method Parallel to the Slope (PCA)

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3. Grade Control Structures

Grade control structures (drop structures) are commonly used in Arizona to mitigate channel bed degradation (see **Figure 2.5**). The location and spacing of grade control structures should be based on analysis of the vertical stability of the system. Toe-down depths for soil cement bank protection below drop structures should be deepened to account for the increased scour. Some typical sections of soil cement grade control structures are shown in **Figure 2.6**.

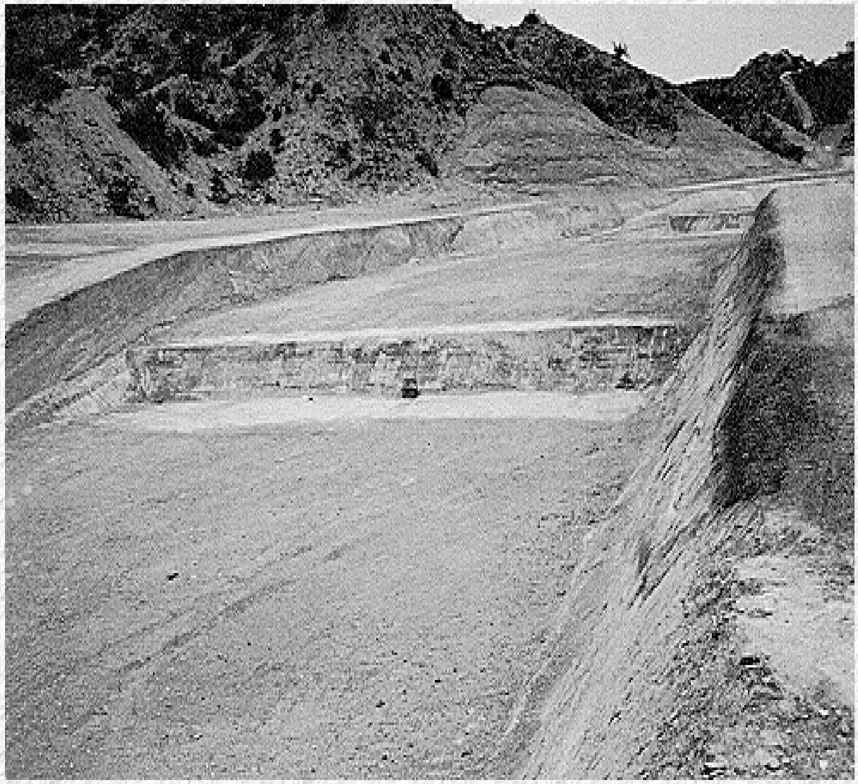

Figure 2.5 Soil Cement Bank Protection and Drop Structures in Laughlin, NV (Hansen)

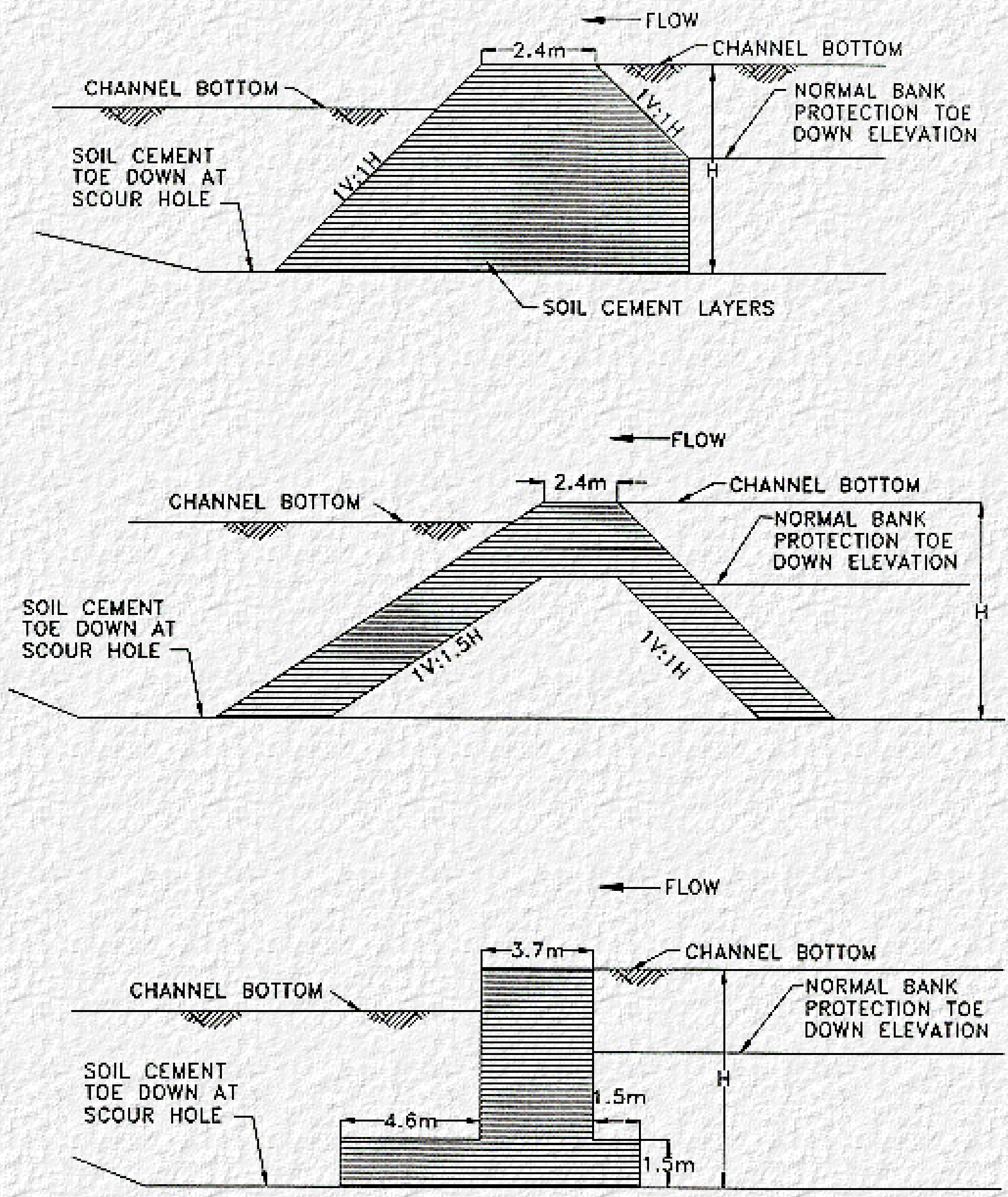

Figure 2.6 Typical Sections for Soil Cement Grade Control Structure (PCA)

Specifications

In addition to application techniques, construction specifications are equally important to the use of soil cement for channel instability and scour countermeasures. Important design considerations for soil cement include: types of materials and equipment used, mix design and methods, handling, placing and curing techniques. The following list of specifications reflects guidance in the **Pima County Department of Transportation's guidelines** on applications and use of soil cement for Flood Control Projects.

Portland Cement

Portland Cement shall comply with the latest Specifications for Portland Cement (ASTM 150, CSA A-5, or AASHTO M85) Type II.

Fly Ash

The Portland Cement Association recommends that fly ash, when used, conform to ASTM Specification C-168.

Water

Water shall be clear and free from injurious amounts of oil, acid, alkali, organic matter or other deleterious substance.

Aggregate

The soil used in the soil cement mix shall not contain any material retained on a 38.1 mm (1-1/2-inch) sieve, nor any deleterious material. Soil for soil cement lining shall be obtained from the required excavations or from other borrow areas and stockpiled on the job site. The actual soil to be used shall be analyzed by laboratory tests in order to determine the job mix. The distribution and gradation of materials in the soil cement lining shall not result in lenses, pockets, streaks, or layers of material differing substantially in texture or gradation from surrounding material. Soil shall conform to the following gradation:

Sieve Size	Percent Passing (Dry Weight)
38.1 mm (1n1 2")	98%n100%
No. 4	60%n90%
No. 200	5%n15%

The Plasticity Index (PI) shall be a maximum of 3. Clays with a PI greater than 6 generally require a greater cement content and are more difficult to mix with cement.

Clay and silt lumps larger than 12.7 mm (1/2 inch) shall be unacceptable, and screening, in addition to that previously specified, will be required whenever this type of material is encountered.

Mix Design

The design requirements for the soil cement shall be such that it has a compressive strength of 5170 kPa (750 psi) at the end of 7 days unless otherwise specified. A 24-hour test will be run to monitor the mix design on a daily basis. Experience has shown that 24-hour compressive strength results for moist cured samples are approximately 50 to 60 percent of the seven day strength (moist cured for six days and soaked in water for 24 hours). Once the design strength mix is determined, a 24-hour test will be run using the mix to obtain a 24-hour compressive strength which will be used to monitor the daily output of the central plant. Seven (7) day samples will also be taken for final acceptance. The amount of stabilizer thus determined by laboratory testing shall continue to be monitored throughout the life of the project with modifications as required for existing field conditions.

NOTE: The **stabilizer** is defined as the cementitious portion of the mix which may be composed of portland cement only or a mixture of portland cement and fly ash or other supplement.

The cementitious portion of the soil-cement mix shall consist of one of the following alternatives:

(1) One hundred percent (100%) portland cement

(2) Eighty five percent (85%) portland cement and fifteen percent (15%) fly ash by weight of stabilizer.

The ratio of replacement shall be one kilogram of fly ash to one kilogram of portland cement removed meaning one to one replacement by weight.

Mixing Method

Soil Cement shall be mixed in an approved central plant having a twin shaft continuous-flow or batch-type pugmill. The plant shall be equipped with screening, feeding and metering devices that will add the soil, cement, fly ash (if utilized), and water into the mixer in the specified quantities. **Figure 2.7** illustrates a typical continuous flow mixing plant operation. In the production of the soil cement, the percent of cement content and the percent of the cement plus fly ash shall not vary by more than +/- 0.3 percent from the contents specified by the Engineer.

NOTE: Soil cement can also be mixed in place, although for most bank protection projects the central plant method is preferred.

Blending of Cement and Fly Ash

The blending procedure shall provide a uniform, thorough, and consistent blend of cement and fly ash. The blending method and operation shall be approved before soil cement production begins. In blending of the stabilizer, the percent of fly ash content shall not vary by more than +/- 0.50 percent of the specified content.

Scales are required at both the cement and fly ash feeds. An additional scale may also be required at the stabilizer feed.

Required Moisture

The moisture content of the mix shall be adjusted as needed to achieve the compressive strength and compaction requirements specified herein.

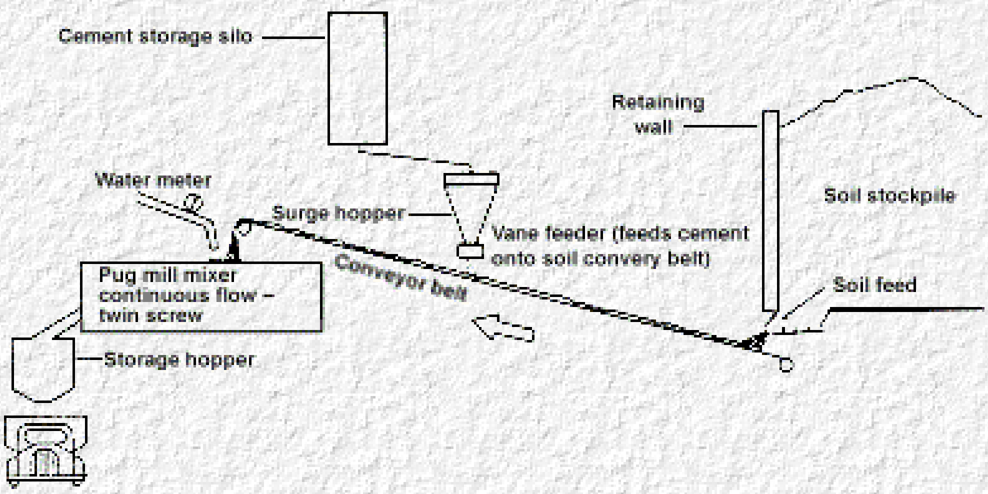

Figure 2.7 Schematic of Continuous Flow Mixing Plant for Soil Cement (Hansen)

Handling

The soil cement mixture shall be transported from the mixing area to the embankment in clean equipment provided with suitable protective devices in unfavorable weather. The total elapsed time between the addition of water to the mixture and the start of compaction shall be the minimum possible. In no case should the total elapsed time exceed thirty (30) minutes. This time may be reduced when the air temperature exceeds 32º C (90º F), or when there is a wind that promotes rapid drying of the soil cement mixture.

Placing

The mixture shall be placed on the moistened subgrade embankment, or previously completed soil cement, with spreading equipment that will produce layers of such width and thickness as are necessary for compaction to the required dimensions of the completed soil cement layers. The compacted layers of soil cement shall not exceed 200 mm (8 inches), nor be less than 100 mm (4 inches) in thickness. Each successive layer shall be placed as soon as practical after the preceding layer is completed and certified.

All soil cement surfaces that will be in contact with succeeding layers of soil cement shall be kept continuously moist by fog spraying until placement of the subsequent layer, provided that the contractor will not be required to keep such surfaces continuously moist for a period of seven (7) days.

Mixing shall not proceed when the soil aggregate or the area on which the soil cement is to be placed is frozen. Soil cement shall not be mixed or placed when the air temperature is below 7º C (45º F), unless the air temperature is 5º C (40º F) and rising.

Compaction

Soil Cement shall be uniformly compacted to a minimum of 98% of maximum density as determined by field density tests. Wheel rolling with hauling equipment only is not an acceptable method of compaction.

At the start of compaction the mixture shall be in a uniform, loose condition throughout its full depth. Its moisture content shall be as specified in the section on Required Moisture (above). No section shall be left undisturbed for longer than thirty (30) minutes during compaction operations. Compaction of each layer shall done in such a manner as to produce a dense surface, free of compaction planes, in not longer than 1 hour from the time water is added to the mixture. Whenever the operation is interrupted for more than two (2) hours, the top surface of the completed layer, if smooth, shall be scarified to a depth of at least 24.5 mm (1 inch) with a spike tooth instrument prior to placement of the next lift. The surface after scarifying, shall be swept using a power broom or other method approved by the engineer to completely free the surface of all loose material prior to actual placement of the soil cement mixture for the next lift.

Finishing

After compaction, the soil cement shall be further shaped to the required lines, grades, and cross section and rolled to a reasonably smooth surface. Trimming and shaping of the soil cement shall be conducted daily at the completion of each day's production with a smooth blade.

Curing

Temporarily exposed surfaces shall be kept moist as set forth in the section on Placing (above). Care must be exercised to ensure that no curing material other than water is applied to the surfaces that will be in contact with succeeding layers. Permanently exposed surfaces shall be kept in a moist condition for seven (7) days, or they may be covered with some suitable curing material, subject to the Engineer's approval. Any damage to the protective covering within seven days shall be repaired to satisfaction of the Engineer.

Regardless of the curing material used, the permanently exposed surfaces shall be kept moist until the protective cover is applied. Such protective cover is to be applied as soon as practical, with a maximum time limit of twenty-four (24) hours between the finishing of the surface and the application of the protective cover or membrane. When necessary, the soil cement shall be protected from freezing for seven (7) days after its construction by a covering of loose earth, straw or other suitable material approved by the Engineer.

Construction Joints

At the end of each day's work, or whenever construction operations are interrupted for more than two (2) hours, a 15% minimum skew transverse construction joint shall be formed by cutting back into the completed work to form a full depth vertical face as directed by the Engineer.

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Go to Design Guideline 3

Design Guideline 3 : HEC 23

Wire Enclosed Riprap Mattress

Go to Design Guideline 4

Introduction

Wire enclosed riprap is commonly used in the state of New Mexico. The predecessor to this erosion control technique is known as rail bank protection and has been used in Arizona, Colorado and New Mexico since the 1970s. Wire enclosed riprap differs from gabions and gabion (Reno) mattresses in that it is a continuous framework rather than individual interconnected baskets. In addition, wire enclosed riprap is typically anchored to the embankment with steel stakes which are driven through the mattress. Construction of wire enclosed riprap is usually faster than gabions or gabion mattresses, and it also requires less wire mesh because internal junction panels are not used. Wire enclosed riprap is used primarily for slope protection. It has been used for bank protection, guide bank slope protection, and in conjunction with gabions placed at the toe of slope.

Design Guidelines

Guidelines for the dimensions, placement, anchoring, splicing, and quantity formulas are shown on **Figure 3.1**. Design procedures for the selection of rock fill for wire enclosed riprap can be found in Brown and Clyde (HEC-11) (1989), Simons et al. (1984) and Maynord (1995). Guidelines on selection and design of filter material can be found in HEC-11 (1989) and Holtz et al. (FHWA HI-95-038) (1995). The following guidelines and specifications reflect construction procedures for wire enclosed riprap recommended by the New Mexico State Highway and Transportation Department (NMSHTD).

1. Wire mesh fabric for riprap shall be hexagonal mesh or a "V" mesh meeting the requirements listed in the specifications.
2. Steel stakes may be railroad rails, not less than 14.9 kg/m (30 lb per yard), 102 mm (4 in.) O.D. standard strength galvanized steel pipe, or 102 mm X 102 mm X 9.5 mm (4" X 4" X 3/8") steel angles.
3. If length of slope is 4.6 m (15 ft) or less, only one row of steel stakes 610 mm (2 ft) from the top edge of the riprap will be required unless otherwise noted on the plans.
4. Dimensions of the thickness, top of slope and toe of slope extents, and total length of protection shall be designated on the bridge or roadway plans.
5. The wire enclosed riprap thickness is usually 300 mm (12 in) unless otherwise shown on the plans. Thickness is usually 460 mm (18 in) at bridges.
6. Longitudinal splices may be made with one lap of galvanized 9 gage tie wire, 9 gage hog rings or 11 ½ gage galvanized hard drawn interlocking wire clips.

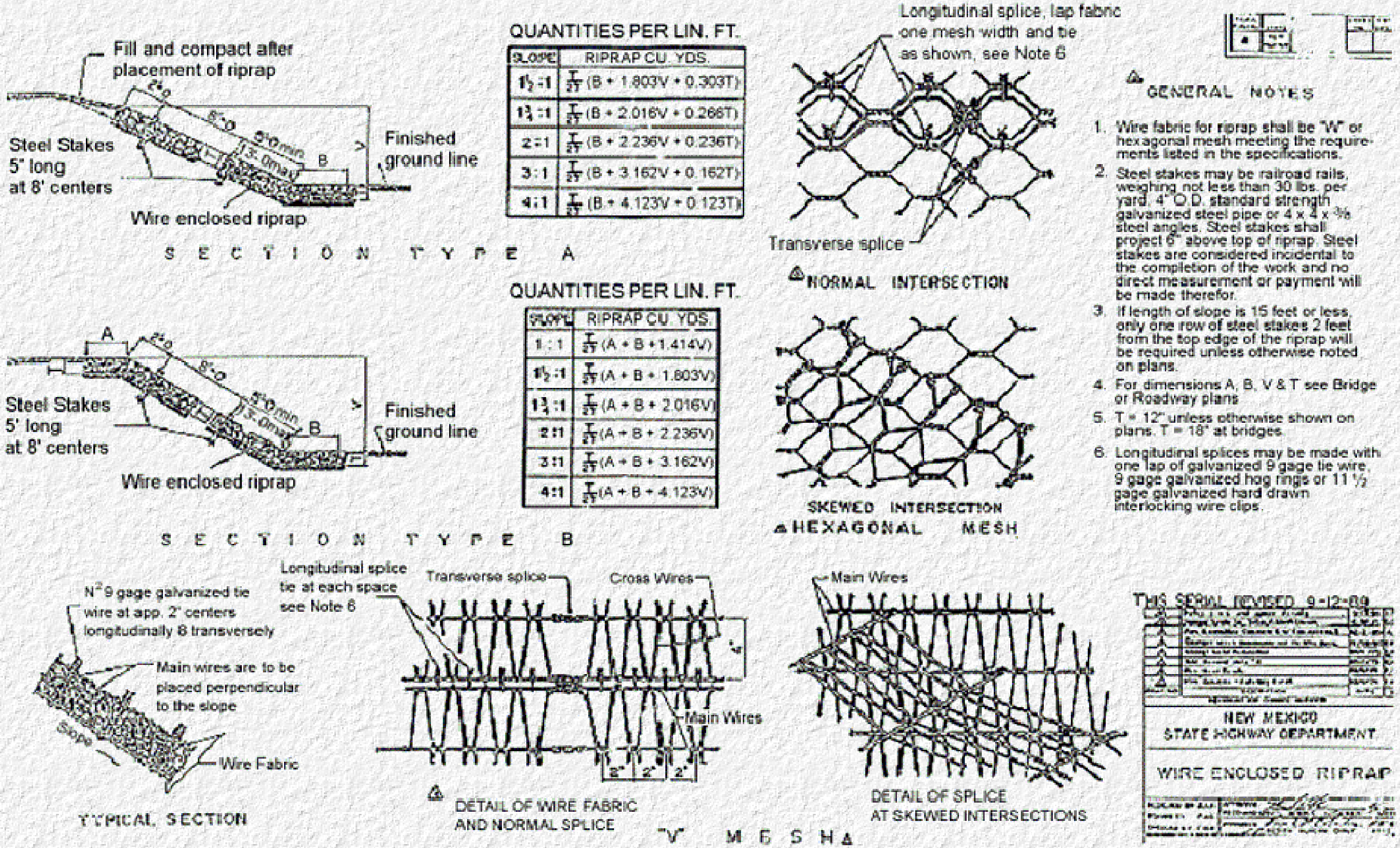

Figure 3.1 Wire Enclosed Riprap Plans (NMSHTD)

Specifications

Wire Enclosed Riprap:

Wire enclosed riprap shall consist of a layer of rock of the required thickness enclosed on all sides in wire fabric conforming with the details shown on the plans (see Figure 3.1). The wire fabric shall be drawn tightly against the rock on all sides and tied with galvanized wire, locking clips, hog rings or connectors. When ties, locking clips, hog rings or connectors are used for tying mesh sections and selvages together, they shall be spaced 76 mm (3 inches) apart or less as shown on the plans. Galvanized wire ties shall be spaced approximately 610 mm (2 feet) on center and shall be anchored to the bottom layer of wire fabric, extended through the rock layer, and tied securely to the top layer of wire fabric. When indicated on the plans, wire enclosed riprap shall be anchored to the slopes by steel stakes driven through the riprap into the embankment. Stakes shall be spaced as indicated on the plans.

Filter:

See Brown and Clyde (HEC-11) (1989) and Holtz et al. (FHWA HI-95-038) (1995) for selection, design, and specifications of filter materials.

Installation Example

A typical example of wire enclosed riprap installed by NMSHTD is shown in **Figure 3.2.** A side slope of a guide bank at the I-25 crossing of the Rio Galisteo protected with wire enclosed riprap is shown.

Figure 3.2 Wire Enclosed Riprap Used for Guide Bank Side Slope Protection at I-25 Crossing of Rio Galisteo, New Mexico (NMSHTD)

References

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Go to Design Guideline 4

Design Guideline 4 : HEC 23

Articulated Concrete Block System

Go to Design Guideline 5

Introduction

Articulated concrete block systems (ACB's) provide a flexible alternative to riprap, gabions and rigid revetments. These systems consist of preformed units which either interlock or are held together by steel rods or cables (see **Figure 4.1**), or abut together to form a continuous blanket or mat. This design guideline considers two applications of ACB's: Application 1 - bankline and abutment revetment and bed armor; and Application 2 - pier scour protection.

There is little experience with the use of articulated block systems as a scour counter-measure for bridge piers alone. More frequently, these systems have been used for revetments and channel armoring where the mat is placed across the entire channel width and keyed into the abutments or bank protection. For this reason, guidelines for placing articulated block systems at banklines and channels are well documented, but there are few published guidelines on the installation of these systems around bridge piers. Where articulated block systems have been installed as a countermeasure for scour at bridge piers, cable-tied concrete mats have more often been used.

Specifications and design guidelines for installation and anchoring of ACBs are documented in Brown and Clyde (HEC-11) (1989) and guidelines on the selection and design of filter material can be found in HEC-11 (1989) and Holtz et al. (FHWA HI-95-038) (1995). HEC-11 directs the designer to the manufacturer's literature for the selection of appropriate block sizes for a given hydraulic condition. Manufacturers of ACB's have a responsibility to test their products and to develop design criteria based on the results from these tests. Since ACB's vary in shape and performance from one proprietary system to the next, each system will have unique design criteria. A procedure to develop hydraulic design criteria for ACB's given the appropriate performance data for a particular block system is presented in this section.

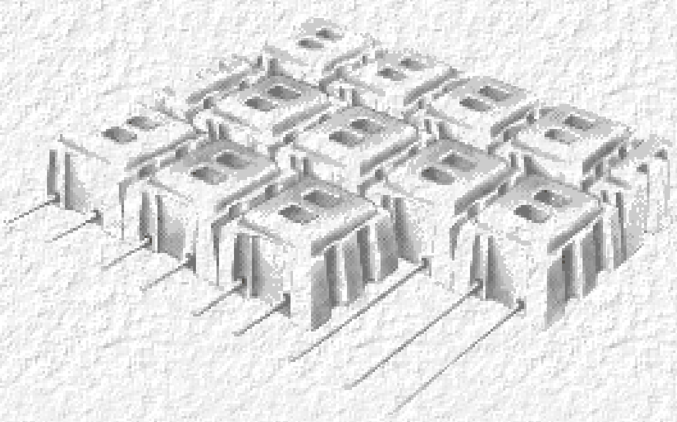


Figure 4.1 Examples of Interlocking Block and Cable-Tied Block Systems (left, courtesy American Excelsior; right, courtesy Armortec)

Background

Beginning in 1983, a group of agencies of the federal government, led by the Federal Highway Administration (FHWA), initiated a multi-year research and testing program in an effort to determine, quantitatively, the performance and reliability of commercially available erosion protection treatments. The research was concluded in July 1989, with the final two years of testing concentrating on the performance of ACB's. Testing methodologies and results for embankment overtopping conditions are published in Clopper and Chen (1988) and Clopper (1989).

The tests provided both qualitative and quantitative insight into the hydraulic behavior of these types of revetments. The mechanisms contributing to the hydraulic instability of revetment linings were identified and quantitatively described as a result of this research effort. Threshold hydraulic loadings were related to forces causing instability in order to better define selection, design, and installation criteria. Concurrently with the FHWA tests, researchers in Great Britain were also evaluating similar erosion protection systems at full scale. Both groups of researchers agreed that an accurate, yet suitably conservative, definition of "failure" for articulated revetment systems can be described as the local loss of intimate contact between the revetment and the subgrade it protects. This loss of contact can result in the progressive growth of one or more of the following destabilizing processes:

1. Ingress of flow beneath the armor layer, causing increased uplift pressure and separation of blocks from subgrade.
2. Loss of subgrade soil through gradual piping erosion and/or washout.
3. Enhanced potential for rapid saturation and liquefaction of subgrade soils, causing shallow slip geotechnical failure (especially in silt-rich soils on steep slopes).
4. Loss of block or group of blocks from the revetment matrix, directly exposing the subgrade to the flow.

Therefore, selection, design, and installation considerations must be concerned, primarily, with maintaining intimate contact between the block system and the subgrade for the stress levels associated with the hydraulic conditions of the design event.

Application 1: Hydraulic Design Procedure for ACB's for Revetment or Bed Armor

The design procedure quantifies the hydraulic stability of revetment block systems using a "discrete particle" approach (like many riprap sizing methods). This approach is in contrast to the "continuum method" typically used for selecting blankets or vegetative linings. The design approach is similar to that introduced by Stevens (1968) to derive the "**factor of safety**" method of riprap design as described in Richardson et al. (HIRE) (1990). The force balance has been recomputed considering the properties of concrete blocks, and the Shields relationship utilized in the HIRE approach to compute the critical shear stress has been replaced with actual test results. The design procedure incorporates results from hydraulic tests into a method which is based on fundamental principles of open channel flow and rigid body mechanics. The ratio of resisting to overturning moments (the "force balance" approach) is analyzed based on the size and weight characteristics of each class and type of block system and includes performance data from full-scale laboratory testing. This ratio is then used to determine the "**factor of safety**" against the initiation of uplift about the most critical axis of the block.

Considerations are also incorporated into the design procedure which can account for the additional forces generated on a block which protrudes above the surrounding matrix due to subgrade irregularities or imprecise placement. **Since finite movement constitutes "failure", as defined in the foregoing discussion, the analysis methodology purposely contains no explicit attempt to account for resistive forces due to cables or rods.** Similarly, the additional stability which may arise from vegetative root anchorage or mechanical anchoring devices, while recognized as significant, is ignored in the analysis procedures for the sake of conservatism in selection and design.

Selection of Factor of Safety

The designer must determine what factor of safety should be used for a particular design. Some variables which should affect the selection of the factor of safety used for final design are: risks associated with a failure of the project, the uncertainty of hydraulic values used in the design, and uncertainties associated with installation practices. Typically, a minimum factor of safety of 1.5 is used for revetment design when the project hydraulic conditions are well known and variations in the installation can be accounted for. Higher factors of safety are typically used for protection at bridge piers, abutments and at channel bends due to the complexity in computing shear stress at these locations. Research is being conducted to determine appropriate values for factors of safety at bridge piers and abutments.

Stability of a Single Concrete Block on a Sloping Surface

The stability of a single block on a sloping surface is a function of the magnitude and direction of stream velocity and shear stress, the depth of flow, the angle of the inclined surface on which it rests, geometric properties, and weight. Considering flow along a channel bank as shown on **Figure 4.2**, the forces acting on a concrete block are the lift force F_L, the drag force F_D, and the weight of the block, W_A. Block stability is determined by evaluating the moments about the point O about which rotation can take place. The components of forces relative to the plane of motion (assumed to act along the resultant force R) are shown in Figure 4.1.c. The relationship that defines the equilibrium of the block is:

$$\ell_2 W_A \cos\theta = \ell_1 W_A \sin\theta \cos\beta + \ell_3 F_D \cos\delta + \ell_4 F_L$$

where the symbols are shown in Figure 4.1 and described below:

W_A = weight of the block

ℓ_1 and ℓ_2 = moment arms of the weight of the block (side slope and longitudinal slope)

F_D = drag force on the block

F_L = lift force on the block

ℓ_3 and ℓ_4 = moment arms of the lift and drag forces on the block

θ = side slope angle relative to the horizontal plane

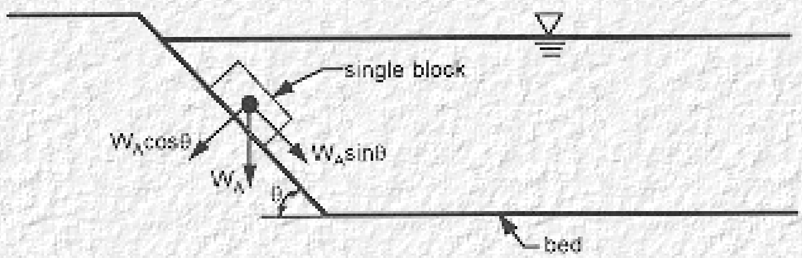
λ = angle between the horizontal and the velocity vector measured in the plane of the side slope. This derivation is valid for "horizontal condition" where $\lambda = \alpha$, where α = slope angle of a plane bed (i.e. uniform flow parallel to bed)

δ = angle between the drag force and particle movement direction = 90 - β - λ

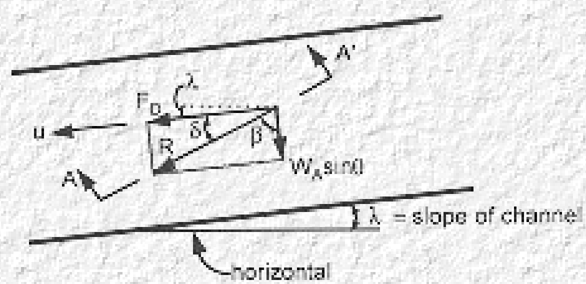
β = angle between the block movement direction and the vertical plane

The factor of safety, SF, for the block can be defined as the ratio of moments resisting motion to those tending to rotate the block out of its resting position. Accordingly:

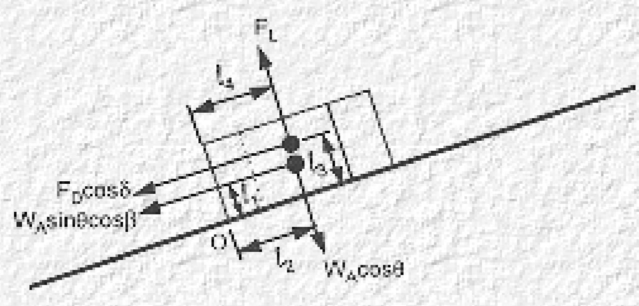
$$SF = \frac{\ell_2 W_A \cos\theta}{\ell_1 W_A \sin\theta \cos\beta + \ell_3 F_d \cos\delta + \ell_4 F_L} \qquad \text{(Factor of Safety)}$$



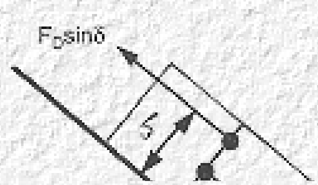
a. Cross section view



b. View normal to the sideslope



c. Section A-A'



d. View normal to Section A-A'

Figure 4.2 Forces Acting on a Single Block Resting on the Side Slope of a Channel

rearranging and substituting terms gives the final form of the factor of safety equations:

$$SF = \frac{\cos\theta\left(\frac{\ell_2}{\ell_1}\right)}{\eta\left(\frac{\ell_2}{\ell_1}\right) + \sin\theta\cos\theta} \qquad \text{(Factor of Safety)}$$

$$\beta = \tan^{-1}\frac{\cos\lambda}{\left(\frac{\frac{M}{N}+1}{\eta}\frac{\ell_1}{\ell_2}\right)\sin\theta + \sin\lambda}$$

The stability number, η , is defined as:

$$\eta = \frac{\tau_0}{\tau_c} \qquad \text{(Stability Number)}$$

where:

τ_o = the shear stress or tractive force acting on the channel boundaries and can be computed from design hydraulic conditions.

τ_c = the critical shear stress when "failure" occurs.

$$\eta' = \left\{\frac{\frac{M}{N} + \sin(\lambda + \beta)}{\frac{M}{N} + 1}\right\}\eta \qquad \text{(Stability Number on a side slope)}$$

where

$$\frac{M}{N} = \frac{\ell_4 F_L}{\ell_3 F_D}$$

The above equations can be solved by knowing τ_o and τ_c and the angles θ and λ, and assuming the ratios ℓ_1/ℓ_2, ℓ_3/ℓ_4 and F_L/F_D.

Incipient motion analysis identifies t_c as the loading which causes a single particle to begin to move. Critical shear stress for sediments can be estimated based on particle size diameter from relationships such as the Shields equation. Extensive research has been conducted for incipient motion analysis of sediments and larger sized rocks. However, there are limited test data on the performance of proprietary products such as ACB's. Therefore, hydraulic testing of ACB's must be conducted before a complete design procedure can be developed. Several manufacturers have performed these tests for their products. The hydraulic tests allow sizing and design criteria to be developed from the data generated. Using the procedure discussed above with hydraulic testing, a design methodology can be established for almost any size or shape of block.

Consideration of Additional Forces Due to Projecting Blocks

When the additional forces of projecting blocks are considered (see **Figure 4.3**) the factor of safety equation becomes:

$$SF = \frac{\cos\theta\left(\frac{\ell_2}{\ell_1}\right)}{\eta'\left(\frac{\ell_2}{\ell_1}\right) + \sin\theta\cos\beta + \frac{\ell_3 F_D' \cos\delta + \ell_4 F_L'}{\ell_1 W_A}}$$

(Factor of Safety including additional forces from block projecting above the matrix)

where F'_D and F'_L are the additional lift and drag forces caused by the projecting block. Numerical tests indicate that it is sufficiently accurate to compute the drag force on the block in the following manner:

$$F_D' = C\left(\Delta Z\, \omega \rho V^2\right)$$

where ΔZ is the projection height, ω is the width of projection, C is the momentum transfer coefficient assumed equal to 0.5, ρ is the fluid density, and V is velocity.

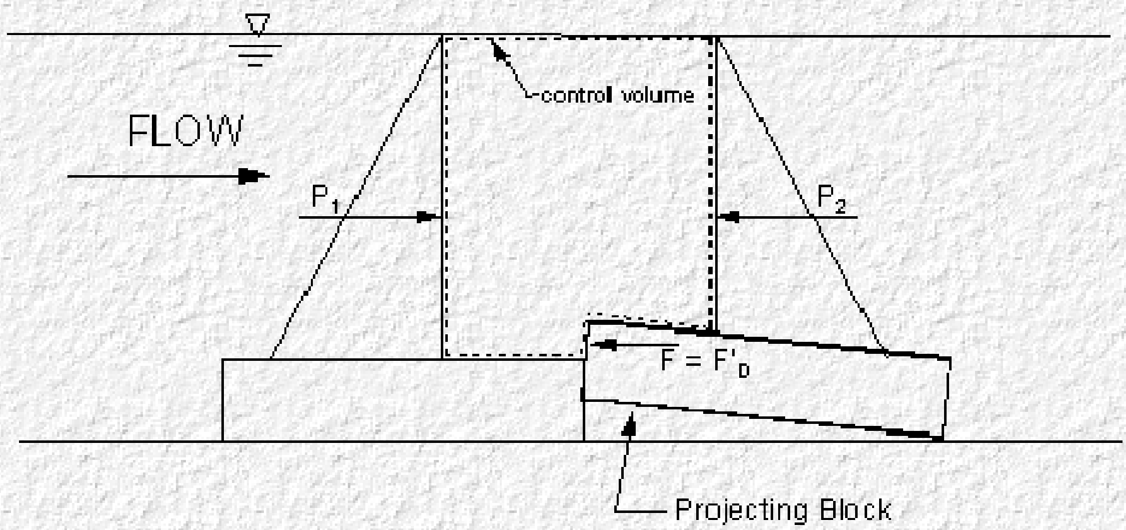


Figure 4.3 Control Volume for Computing Horizontal Force on a Projecting Block

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Hydraulic Design Example (Factor of Safety Method)

The following example illustrates the use of the factor of safety method in the selection of block sizes for ACB's for revetment or bed armor. Two generic block sizes are used to illustrate the use of design charts and the factor of safety equations. A design example using design charts similar to those which would be provided by a block manufacturer is presented and a design example using the factor of safety equations, directly, is presented. The examples assume that hydraulic testing has been performed for the block system to quantify a critical shear stress and to develop the design charts.

Given:

A trapezoidal channel with a bed slope of 0.039 m/m, side slopes 1V:2.5H, and the following hydraulic conditions:

Block Size 1	Block Size 2
n = 0.032	n = 0.026
Depth = 616 mm	Depth = 549 mm
Velocity = 3.78 m/s	Velocity = 4.36 m/s
Hyd. Radius = 475 mm	Hyd. Radius = 433 mm
Bed Shear, τ_o = 235.2 Pa	Bed Shear, τ_o = 209.8 Pa

Block Size 1 has a greater open area and therefore yields a higher Manning's n value.

Design Chart Example

Design charts can be developed from the factor of safety method given block properties and hydraulic test results. These are normally developed by the ACB manufacturer for use by the design engineer. Typically these curves relate the allowable shear stress or velocity to channel bed slope for a given factor of safety as shown in **Figure 4.4**. This chart represents the stability of the ACB's placed flat on the channel bed neglecting the influence of the side slope. Charts which account for the effect of channel side slope on the factor of safety are also provided by the manufacturer (see **Figure 4.5**). The factor of safety can then be computed by taking the ratio of the allowable shear stress or velocity to the design conditions as follows:

$$SF = \frac{\tau_a}{\tau_o}(SF_a)K_1 \quad \text{or} \quad SF = \frac{V_a}{V_o}(SF_a)K_1$$

where:

τ_a and V_a = the allowable shear stress and velocity for the factor of safety for which the chart was developed (SF_a). τ_a and V_a are synonymous to critical shear stress and velocity for a factor of safety of one.

τ_o and V_o = the design shear stress and velocity.

K_1 = side slope correction factor.

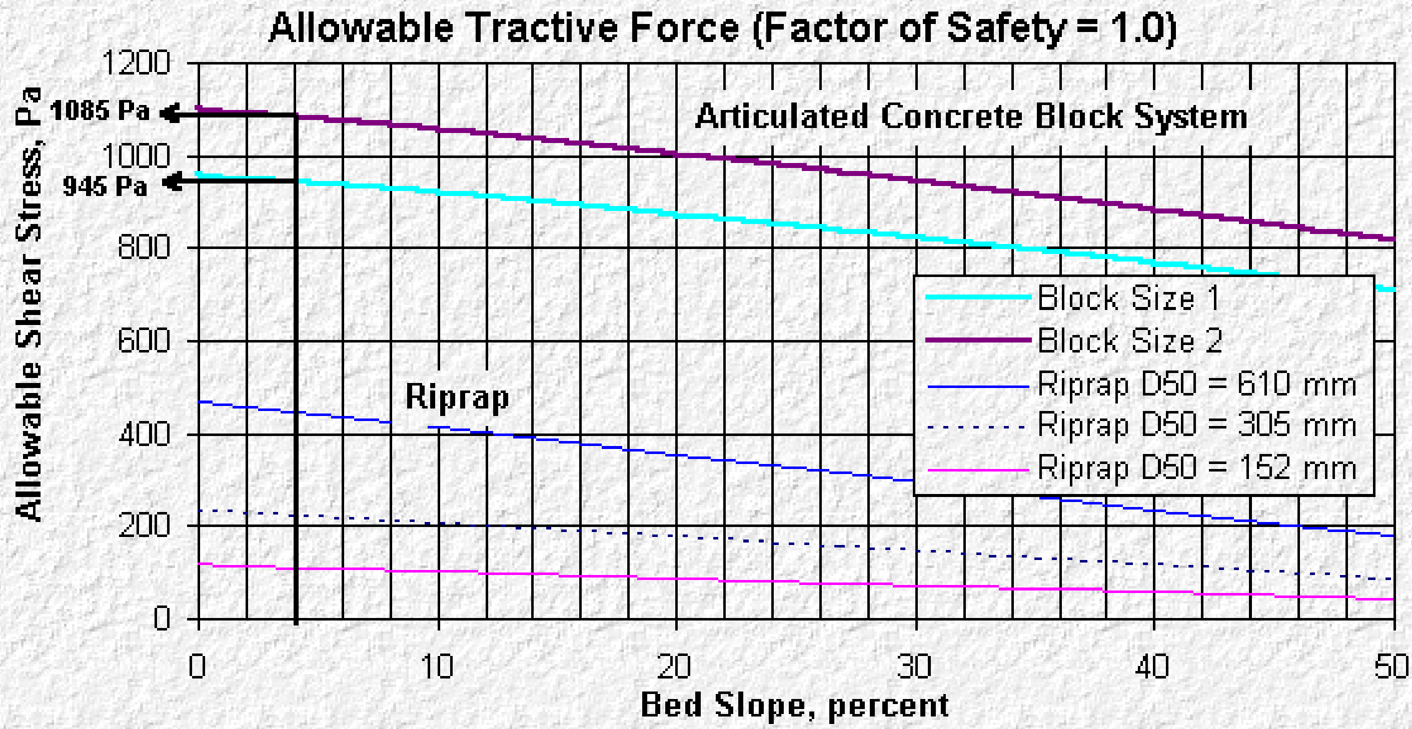

Figure 4.4 Plot of Allowable Shear Stress vs. Bed Slope

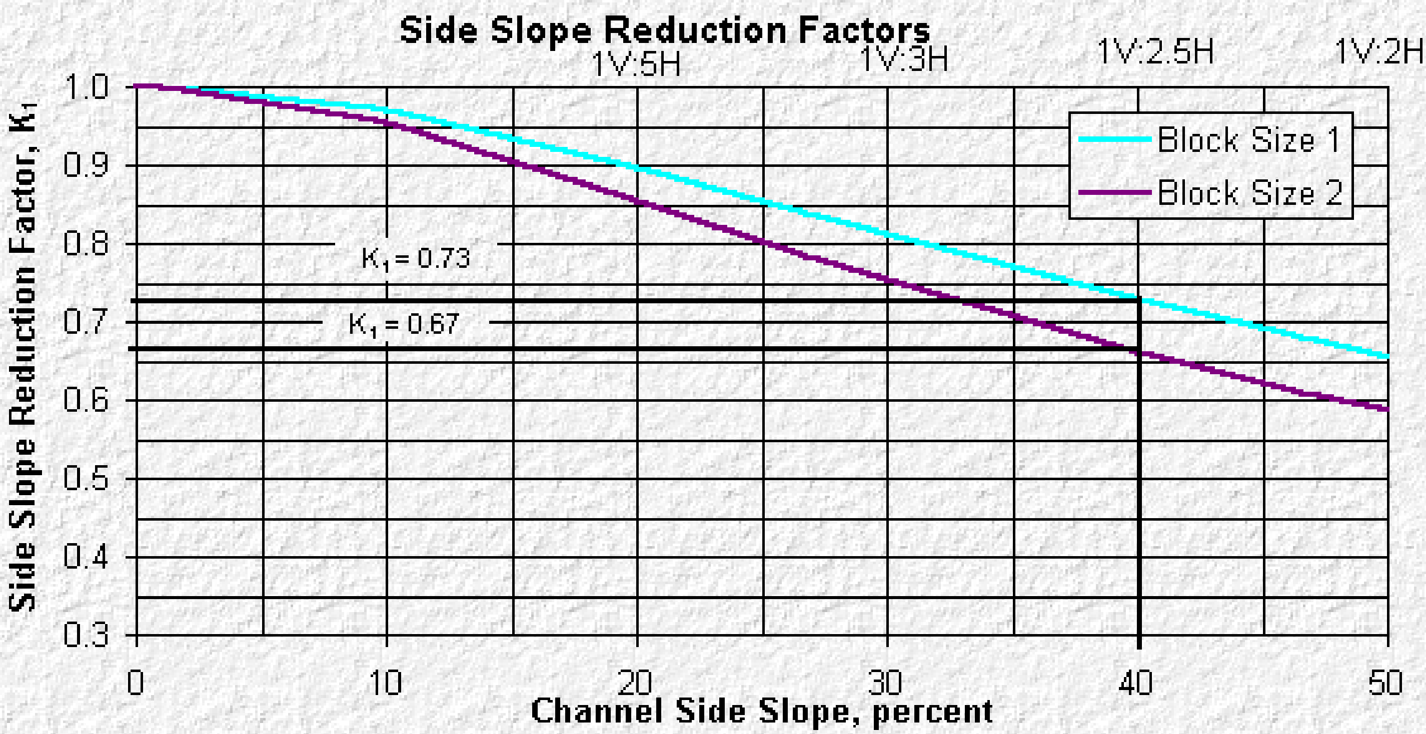

Figure 4.5 Plot of Side Slope Reduction Factors

Step 1: Determine the allowable shear stress for the hydraulic conditions.

From Figure 4.4 the allowable shear stress for the ACB's on a bed slope of 3.9% with a factor of safety of one is:

τ_a = 945 Pa (allowable shear stress for Block Size 1)

τ_a = 1085 Pa (allowable shear stress for Block Size 2)

Step 2: Determine the side slope correction factor, K_1:

From Figure 4.5 the reduction factor for a 1V:2.5H side slope is:

$K_1 = 0.73$ (for Block Size 1)

$K_1 = 0.67$ (for Block Size 2)

Step 3: Determine the factor of safety for blocks placed on the channel side slope:

$$SF = \frac{\tau_a}{\tau_o}(SF_a)K_1 = \frac{945}{235.2}(1)\,0.73 = 2.9$$ (for Block Size 1)

$$SF = \frac{\tau_a}{\tau_o}(SF_a)K_1 = \frac{1085}{209.8}(1)\,0.67 = 3.5$$ (for Block Size 2)

Factor of Safety Equations Example

Given: In addition to the hydraulic conditions given above, the following block characteristics are provided.

Block Size	**Submerged Weight (N)**	**ℓ_1 (mm)**	**ℓ_2 (mm)**	**ℓ_3 (mm)**	**ℓ_4 (mm)**	**ΔZ (mm)**	**ω (mm)**	**τ_c* Pa (N/m^2)**
1	127	76	223	122	223	12.7	329	958.0
2	148	76	223	122	223	12.7	329	1102.0

*τ_c determined from testing.

NOTE: For computations, variables have been converted to meters.

Step 1: Compute factor of safety parameters

$$\theta = \tan^{-1}\left(\frac{1}{2.5}\right) = 21.8°$$ (side slope angle)

$$\lambda = \tan^{-1}\left(\frac{0.039}{1}\right) = 2.23°$$ (bed slope angle)

$$\eta = \frac{\tau_o}{\tau_c} = \frac{235.2}{958} = 0.246$$ (stability number for Block Size 1)

$$\eta = \frac{\tau_o}{\tau_c} = \frac{209.8}{1102} = 0.190$$ (stability number for Block Size 2)

conservatively assuming that $F_L = F_D$ then:

$$\frac{M}{N} = \frac{\ell_4 F_L}{\ell_3 F_D} = \frac{223}{122} = 1.83$$ (for both block sizes)

$$\beta = \tan^{-1}\frac{\cos(2.23)}{\left(\frac{1.83+1}{\eta}\frac{76}{223}\right)\sin(21.8)+\sin(2.23)}$$

$$\beta = 33.63°$$ (for Block Size 1)

$$\beta = 27.33°$$ (for Block Size 2)

$$\delta = 90 - \beta - \lambda$$

$\delta = 54.14°$ (for Block Size 1)

$\delta = 60.44°$ (for Block Size 2)

$\eta' = 0.210$ (stability number on the side slope for Block Size 1)

$\eta' = 0.156$ (stability number on the side slope for Block Size 2)

$$SF = \frac{\cos(21.8°)\left(\frac{0.223}{0.076}\right)}{0.210\left(\frac{0.223}{0.076}\right) + \sin(21.8°)\cos(33.63°)} = \mathbf{2.94}$$ (for block size 1)

$$SF = \frac{\cos(21.8°)\left(\frac{0.223}{0.076}\right)}{0.156\left(\frac{0.223}{0.076}\right) + \sin(21.8°)\cos(27.33°)} = \mathbf{3.46}$$ (for block size 2)

Now the effect of possible vertical projections in the flow must be considered. It is assumed that an installation specification tolerance of 12.7 mm (0.5 inches) in the vertical will be maintained.

$$F_D' = 0.5\left(0.0127(0.329)(1000)(V)^2\right) = 2.089V^2$$

$$F_D' = 2.089(3.78)^2 = 29.8$$ for Block Size 1

$$F_D' = 2.089(4.36)^2 = 39.7$$ for Block Size 2

Now assuming that the additional lift due to the vertical displacement is equal to the additional drag (that is $F_D = F_L$):

$$SF = \frac{\cos(21.8°)\left(\frac{0.223}{0.076}\right)}{0.210\left(\frac{0.223}{0.076}\right) + \sin(21.8°)\cos(33.63°) + \frac{0.122(29.8)\cos(54.14°) + 0.223(29.8)}{0.076(127)}} = 1.48$$ (for Block Size 1)

$$SF = \frac{\cos(21.8°)\left(\frac{0.223}{0.076}\right)}{0.156\left(\frac{0.223}{0.076}\right) + \sin(21.8°)\cos(27.33°) + \frac{0.122(39.7)\cos(60.44°) + 0.223(39.7)}{0.076(148)}} = 1.53$$ (for Block Size 2)

Step 4:

Block Size 1 exhibits a factor of safety slightly less than the minimum value of 1.50. **Recommend Block Size 2**

It can be seen that the consideration of projecting blocks has a significant effect on the factor of safety. In this example, a projection of 12.7 mm resulted in a reduction in the factor of safety by approximately a factor of 2. If the effect of projecting blocks is not considered in the development of design charts or the factor of safety equations, then increasing the factor of safety used for final design may be appropriate.

Application 2: Design Guidelines for ACB's for Pier Scour

The hydraulic stability of articulated block systems at bridge piers can be assessed using the factor of safety method as previously discussed. However, uncertainties in the hydraulic conditions around bridge piers warrant increasing the factor of safety in lieu of a more rigorous hydraulic analysis. Experience and judgment are required when quantifying the factor of safety to be used for scour protection at an obstruction in the flow. In addition, when both contraction scour and pier scour are expected, design considerations for a pier mat become more complex. The following guidelines reflect guidance from McCorquodale (1993), Minnesota Department of Transportation (MnDOT), and the Maine Department of Transportation (MDOT) for application of ACBs as a countermeasure for pier scour.

Hydraulic model studies were conducted for cable-tied articulated block systems at the University of Windsor, Canada (McCorquodale 1993). Laboratory testing gave rise to a method of quantifying the suggested revetment extent around circular bridge piers as shown in **Figure 4.6**.

The pier scour protection dimensions shown in Figure 4.6 are defined by:

Width of the scour protection mat,	$WS = 2.5Y_s + D$
Upstream extent of scour protection,	$X1 = 1.25\ Y_s$
Downstream extent of scour protection,	$X2 = 3\ Y_s$
Estimated unprotected scour depth, (using the CSU pier scour equation)	$Y_s=(2*K_1*K_2*K_3*(Y_1/a)^{0.35}*Fr^{0.43})*a$

These dimensions are intended to reduce the amount of material required as compared to a rectangular mat. The concept is based on observations of greater pier scour occurring at the upstream end of a pier. The extent of protection at the upstream end of the pier is wider than the extent at the downstream end of the pier. Actual field applications of articulated block systems for pier protection have been installed as rectangular mats. The technique illustrated in Figure 4.6 has not been applied in the field.

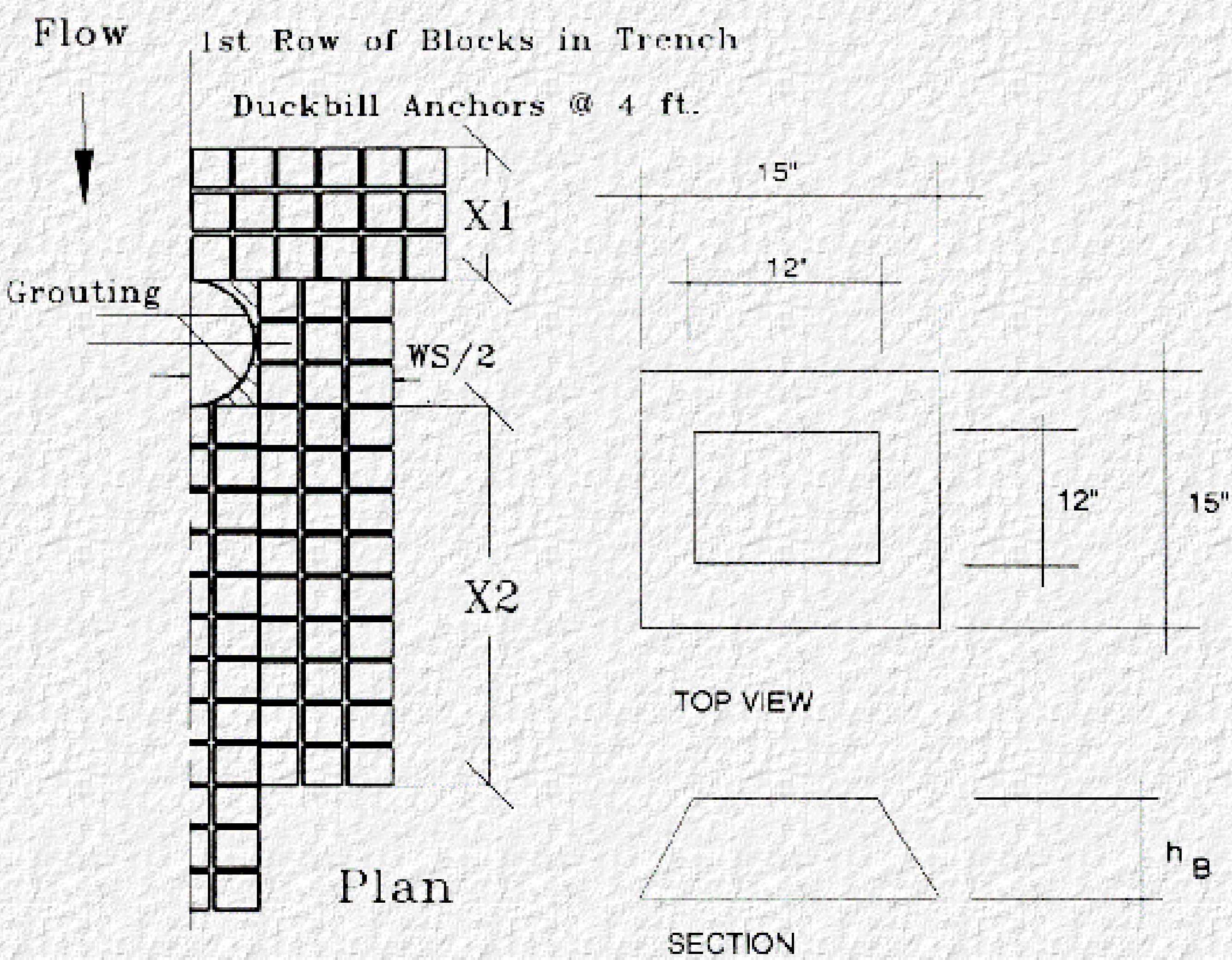

Figure 4.6 Suggested Cable-Tied Mat Dimensions for Scour Protection Around Circular Bridge Piers (McCorquodale)

Guidelines for Seal Around Pier

An observed key point of failure for articulated block systems at bridge piers occurs at the seal where the mat meets the bridge pier. During the flume studies at the University of Windsor, the mat was grouted to the pier to prevent scouring of the sediments adjacent to the pier. This procedure worked successfully in the laboratory, but there are implications which must be considered when using this technique in the field. The transfer of moments from the mat to the pier may affect the structural stability of the pier. When the mat is attached to the pier the increased loadings on the pier must be investigated.

The State of Minnesota Department of Transportation (MnDOT) has installed a cable-tied mat for a pier at TH 32 over Clearwater River at Red Lake Falls. MnDOT recommends the use of tension anchors in addition to grout around the pier seal. Anchors can provide additional support for the mat and grout at the pier seal will reduce scouring underneath the mat. MnDOT provided the following specifications:

Anchors:

Use Duckbill anchors, 0.9n1.2 m (3n4 ft) deep. Use Duckbill anchors at corners and about every 2.4 m (8 ft) around pier footings.

Seal around Pier:

Research conducted by the FHWA has indicated that the space between the pier and the cable-tied concrete blocks must be filled or scour may occur under the blocks. To provide this seal, MnDOT proposed that concrete be placed around the pier. MnDOT suggested that the river bed could be excavated around the piers to the top of the footing. The mat could be put directly on top of the footing and next to the pier with concrete placed underneath, on top, or both, to provide a seal between mat and pier.

The State of Maine Department of Transportation (MDOT) has designed an articulated block system for a pier at Tukey's Bridge over Back Cove. MDOT recommended a design in which grout bags were placed on top of the mat at the pier location to provide the necessary seal (see **Figure 4.7**).

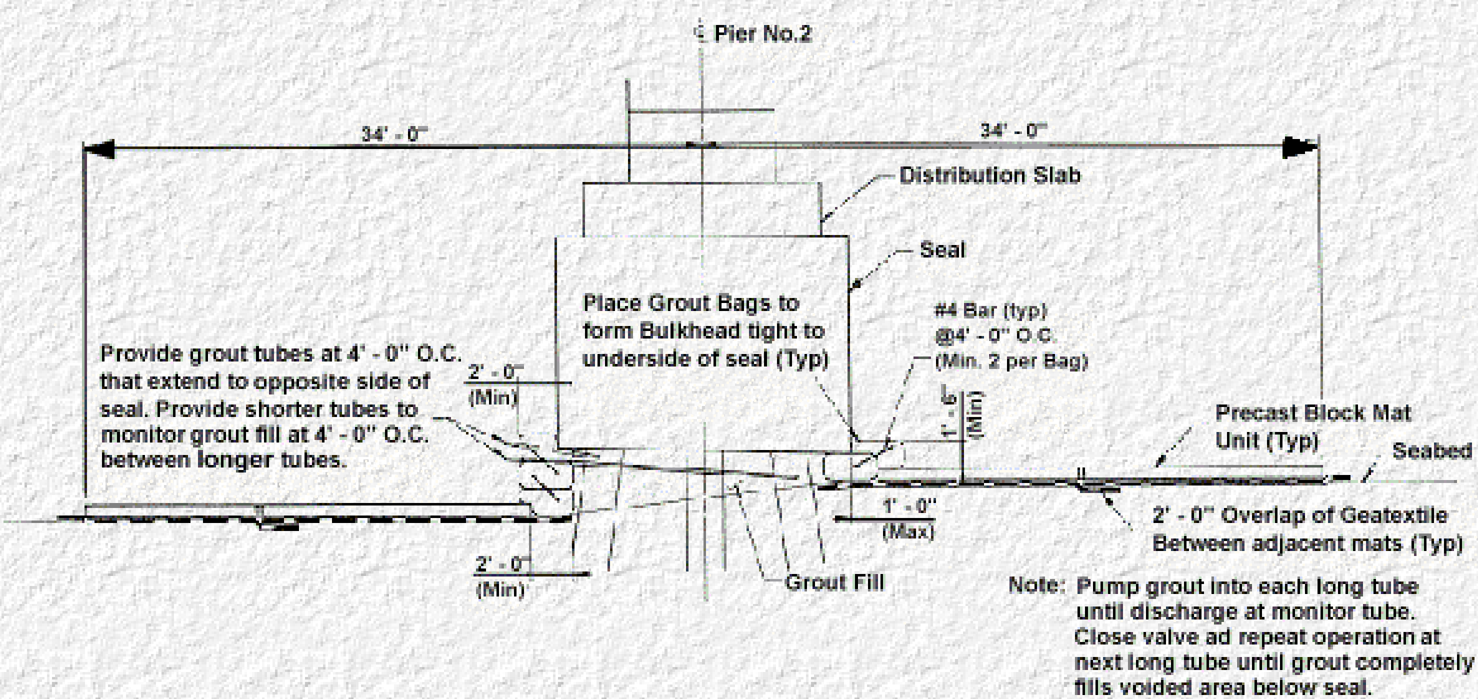

Figure 4.7 Design Plans of Cable-Tied Precast Block Mat for Tukey's Bridge, ME (MDOT)

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Go to Design Guideline 5

Design Guideline 5 : HEC 23
Articulating Grout Filled Mattress

Go to Design Guideline 6

Introduction

Articulating grout filled mattresses are a type of fabric formed concrete used as a flexible armor for slope and channel bed protection. Fabric forms for concrete come in many different designs, but all have the same general concept. A strong synthetic fabric is sewn into a series of bags that are connected internally by ducts. These bags are then filled with a cement rich concrete grout. When set, the concrete forms a mat made up of a grid of connected blocks. The individual blocks can articulate within the mat while the mat remains structurally sound. The advantage is that the mat can shift to fill voids caused by undermining and piping. In addition, the mat provides a surface which is easy to walk on for maintenance activities.

A particular design called "articulating block mat" (ABM), used by the Oregon Department of Transportation, has two features which make it distinctive among fabric formed concrete mats. First, the horizontal seams within the mat are continuous, allowing the blocks to bend downward by hinging along this seam line. Second, the individual blocks are connected internally by a series of flexible polyester cables which keep the individual blocks firmly connected while allowing them to bend (see **Figure 5.1**). Typical individual block sizes are on the order of 0.2 m^2 to 0.37 m^2 (2.25 ft^2 to 4.0 ft^2) and the mass is approximately 180 kg (400 lb) each.

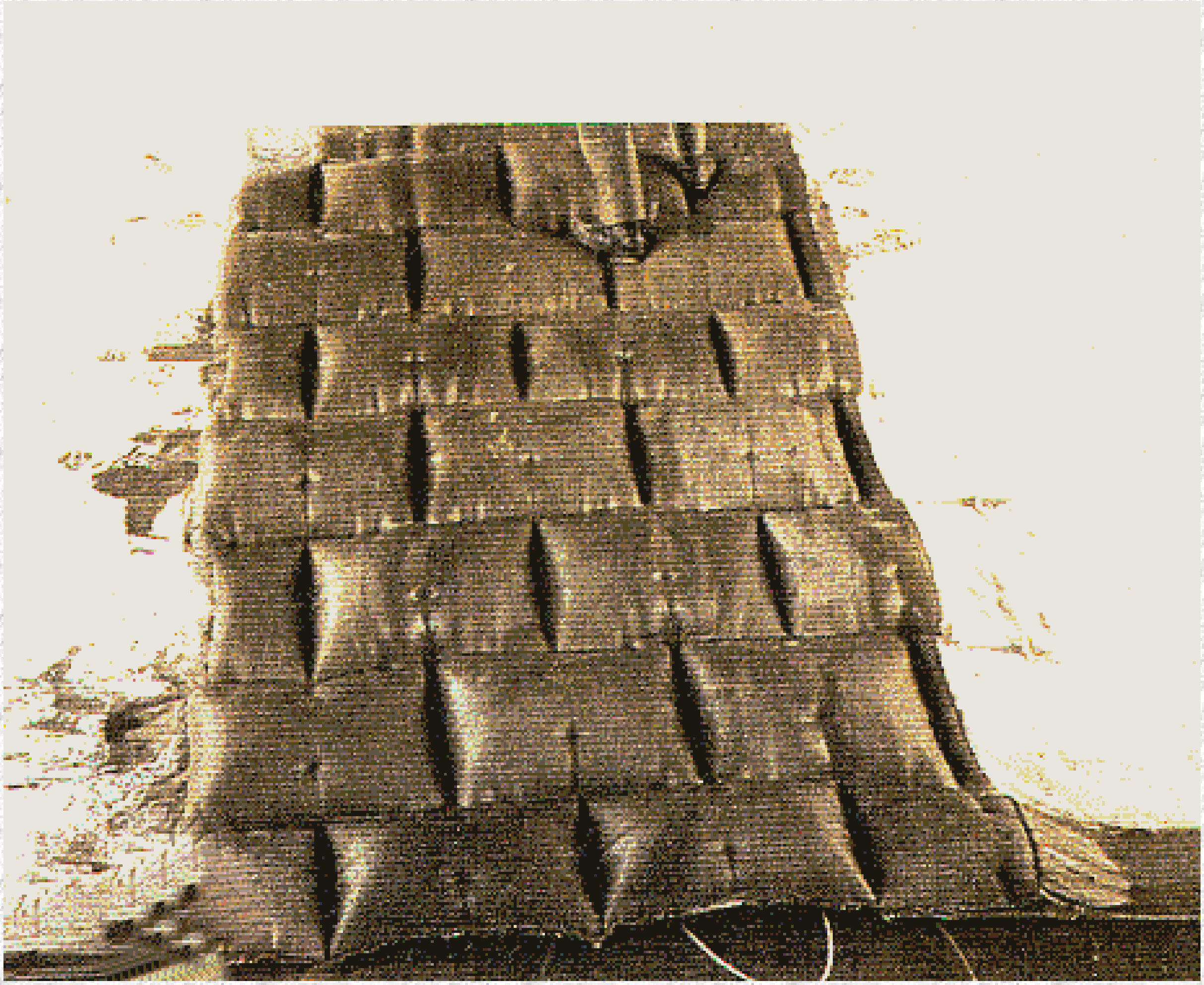


Figure 5.1 Articulating Block Mat Appearance after Filling (ODOT)

Background

Some early installations of concrete fabric mats were completed on Spring Creek and Battle Creek in South Dakota in the early 1970's (Brice and Blodgett 1978, Karim 1975). These installations used much larger sections (1.9 m^2), which made the mat more rigid and more susceptible to undermining. The simplicity of construction and durability of these mats made them an attractive erosion control alternative. Experience and technology have improved the flexibility and performance of fabric formed concrete mats since the 1970's.

Hydraulic Design Procedure

The design procedure involves quantifying the hydraulic stability of the grout filled mattress using a "continuum method" similar to the design approach presented in Chen and Cotton (HEC-15) (1986). This approach is in contrast to the "discrete particle" approach introduced by Stevens (1968) in Richardson et al. (HIRE) (1990) used for selecting riprap and for the design of articulated concrete block systems (ACB's) as presented in Design Guideline 4. The design procedure for grout filled mattresses involves computing the stability of the revetment by comparing the ratio of the tractive forces caused by the flow to the resisting forces of the protection system. The ratio of resisting to tractive forces is known as the **factor of safety** against the initiation of sliding across the subgrade surface as described by the following equation.

$$FS = \frac{\tau_r}{\tau_0}$$

where FS = factor of safety

τ_r = the resistive stresses of the mattress and anchoring system

τ_o = the design hydraulic shear stress caused by the flow

The continuum method of design is unique in that it analyzes the **frictional** forces which affect the initiation of **sliding** between the mattress and subgrade or filter material. This is in contrast to the discrete particle approach which analyzes a single unit for the initiation of overturning as with ACB's and the initiation of motion as with riprap using the Shields equation. The ratio of tractive to resisting frictional forces (the "force balance" approach) is analyzed based on the size and weight characteristics of each mattress, the angle of friction with the supporting subgrade with or without a filter, and additional resistive forces provided by anchoring systems. Forces are quantified on a per unit area basis and the design parameter is the thickness of the mattress.

Reference to manufacturer's literature is necessary for a quantification of the resistive forces supplied by proprietary grout filled mattresses and anchoring systems. A review of tractive forces caused by flowing water in straight channels, at bends, on bed slopes and side slopes, and on steep and mild gradient channels can be found in Chen and Cotton (HEC-15) (1986). The quantification of hydraulic tractive forces is a common procedure for hydraulic engineers. It is the manufacturer's responsibility to provide methods for computing the resistive forces of proprietary products.

Selection of Factor of Safety

The designer must determine what factor of safety should be used for a particular design. Some variables which should affect the selection of the factor of safety used for final design are: risks associated with a failure of the project, the uncertainty of hydraulic values used in the design, and uncertainties associated with installation practices. Typically, a minimum factor of safety of 1.5 is used for revetment design when the project hydraulic conditions are well known and variations in the installation can be accounted for. Higher factors of safety are typically used for protection at bridge piers, abutments and at channel bends due to the complexity in computing shear stress at these locations. Research is being conducted to determine appropriate values for factors of safety at bridge piers and abutments.

Design Guidelines

The selection of an appropriate mat size can be computed by applying the methodologies discussed above given the appropriate data from the manufacturer. Guidelines on the selection, design, and specifications of filter material can be found in Brown and Clyde (HEC-11) (1989) and Holtz et al. (FHWA HI-95-038) (1995). The following recommendations reflect experience from the Oregon Department of Transportation (ODOT) and Arizona Department of Transportation (ADOT). Research reports from an ODOT installation of an articulating block mat erosion control system on Salmon Creek in Oakridge, Oregon also provide experience and insight to the use of these mats.

1. Both upstream and downstream ends of the mat should be trenched (see **Figure 5.2**). The use of tension anchors can increase the stability of the mattress at the edges.

2. All edges should be keyed in and protected to prevent undermining and flow behind the mat.

3. At abutments, the mat can be wrapped around the abutment and buried to provide anchorage and to control flanking.

4. It is recommended that weep holes be cut into the fabric at the seam to allow for proper drainage.

5. The mattress should be filled with portland cement slurry consisting of a mixture of cement, fine aggregate, and water. The mix should be in such proportion of water to be able to pump the mix easily, while having a compressive strength of 17 243 kPa (2500 psi).

6. Fabric mats have been installed on slopes of 1V:1.5H or flatter.

7. Large boulders, stumps and other obstructions should be removed from slopes to be protected to provide a smooth application surface.

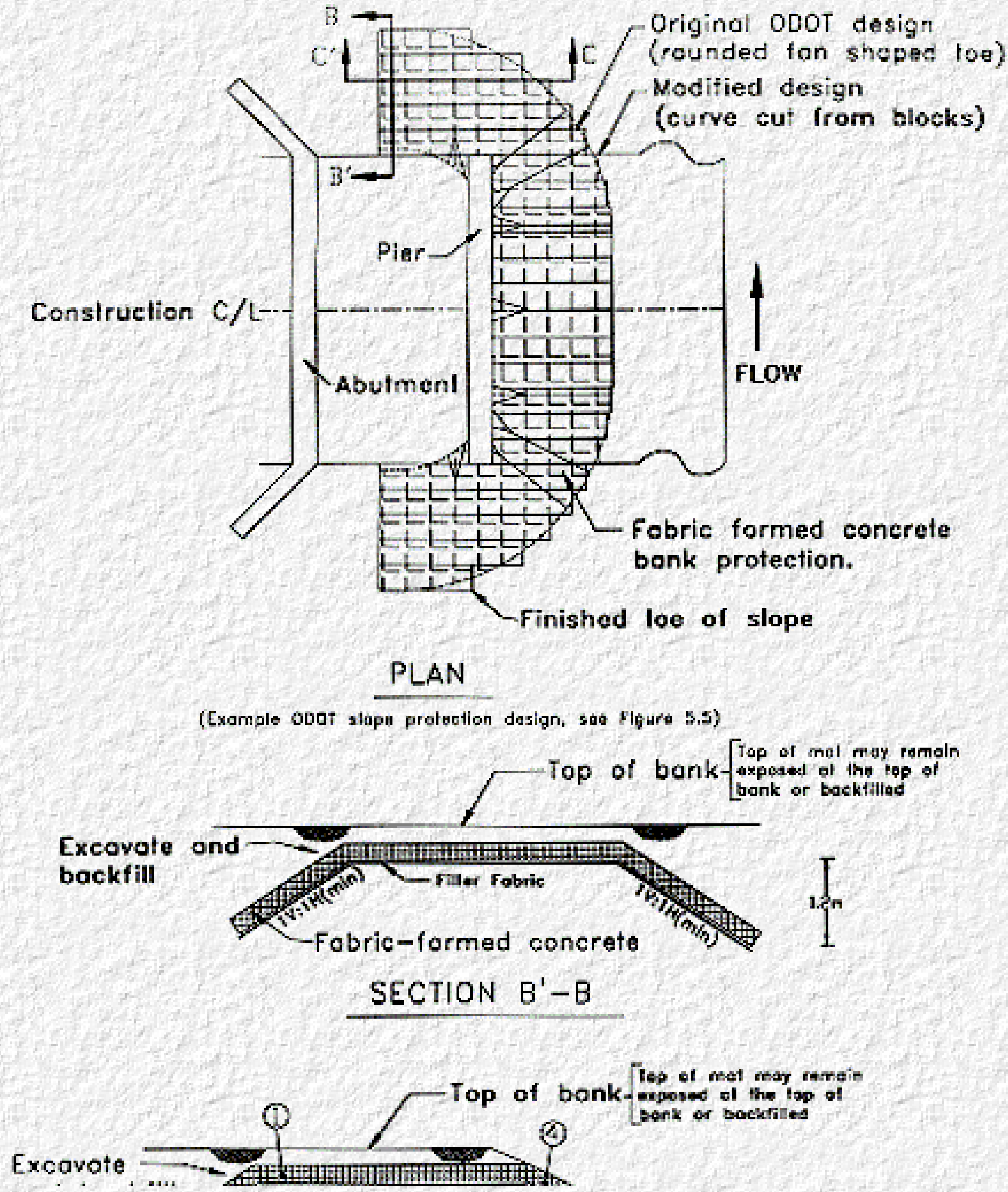
8. Use sand and gravel for any backfill required to level slopes. Silty sand is acceptable if silt content is 20% or less. Do not use fine silt, organic material or clay for backfill.

9. The injection sequence should proceed from toe of slope to top of slope, but the mat should be anchored at the top of slope first by pumping grout into the first rows of bags, by attaching the mat to a structure, or using tension anchors (see recommended injection sequence in **Figure 5.2** and **Figure 5.3**).

10. If the mat is to be permanently anchored to a pier or abutment there are implications which must be considered when using this technique. The transfer of moments from the mat to the pier may affect the structural stability of the bridge. When the mat is attached to the pier the increased loadings on the pier must be investigated.

11. Curved edge designs may require communication with the fabric manufacturer on shaping limitations and field adjustments.

12. The need for a geotextile or granular filter should be addressed. Guidelines on the selection, design, and specifications of filter material can be found in Brown and Clyde (HEC-11) (1989) and Holtz et al. (FHWA HI-95-038) (1995).



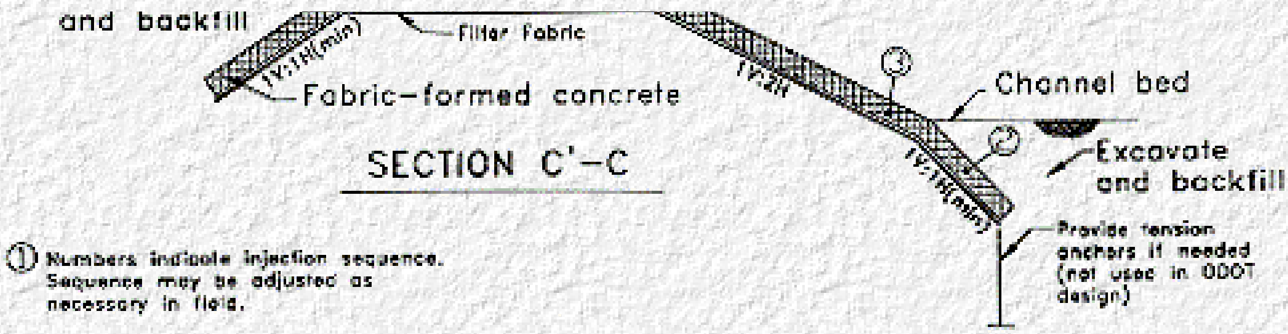


Figure 5.2 Typical Articulated Grout Filled Mat Design

Figure 5.2 illustrates some of the installation features specified by ODOT on the Salmon Creek Bridge as well as typical design features. Notice that the original ODOT design was modified by the manufacturer due to the limitations of the product. The fabric forms could not be terminated in a smooth fan shaped pattern as shown in the original ODOT design. Therefore, the mat was cut at the seams to best fit the original design. It was anticipated that this would make the system somewhat less effective than the original design because of a greater susceptibility to undermining of the edges. **Figures 5.4 and 5.5** show the final installation of the articulating block mat at Salmon Creek Bridge.

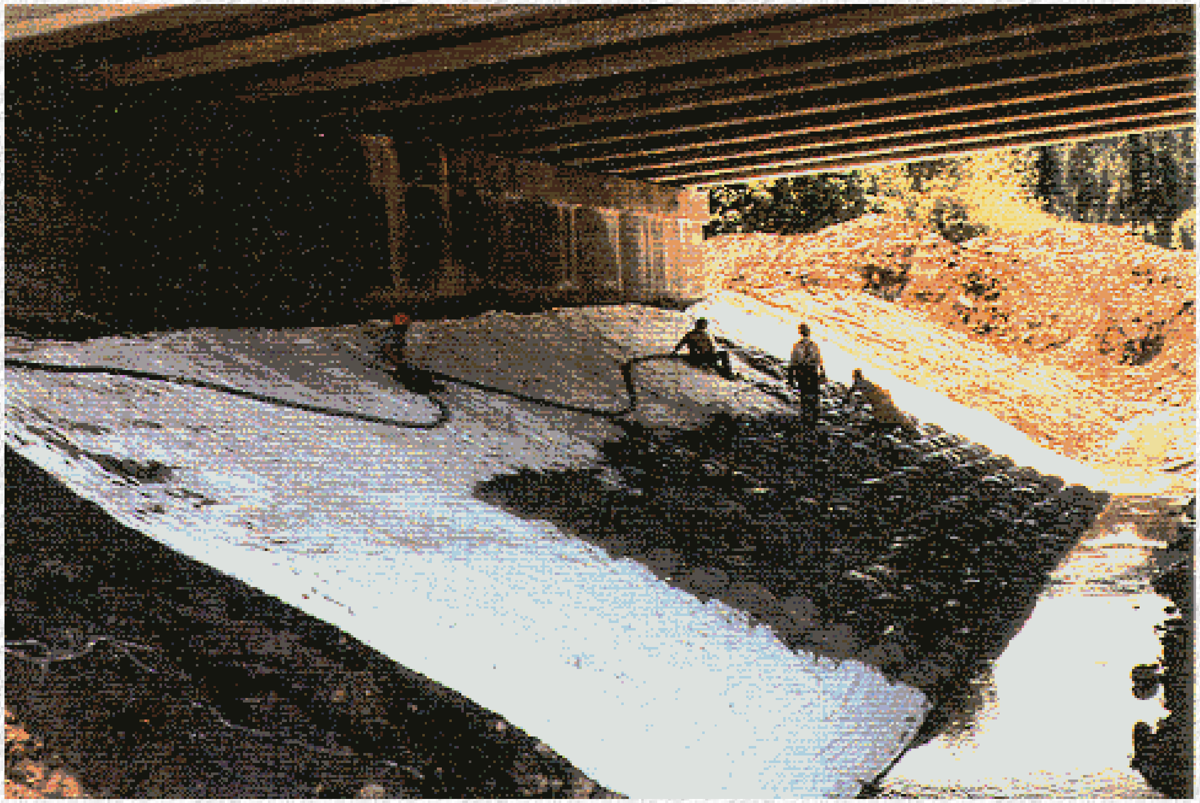


Figure 5.3 Installation of Articulating Block Mat Proceeding Upslope (ODOT)

Problems and Solutions

Some Problems and solutions identified in the construction process by ODOT are (Scholl 1991):

1. *Problem*: In the original attempt to create a smooth working surface for laying the fabric, sand was placed over the native material. This was a problem because footprints readily disturbed the surface.
   Solution: The native material (a gravelly sand) was used for the final surface by first clearing it of major rocks, then compacting it.

2. *Problem*: There was difficulty in estimating where the toe of the finished slope would be.
   Solution: Assume that the fabric contracts by 10% in length after filling with grout.

3. *Problem*: It was difficult to maintain straight lines along the horizontal seams when pumping grout.
   Solution: The fabric was kept straight by tying it to a series of #6 reinforcing bars.

4. *Problem*: Several of the bags were sewn in such a way that the grout ducts connecting them to the other bags were blocked off. This occurred mostly in areas where the bags were cut during fabrication to only ½ the original size.
   Solution: The bags were split and filled individually. This should not affect the strength or function of the system.

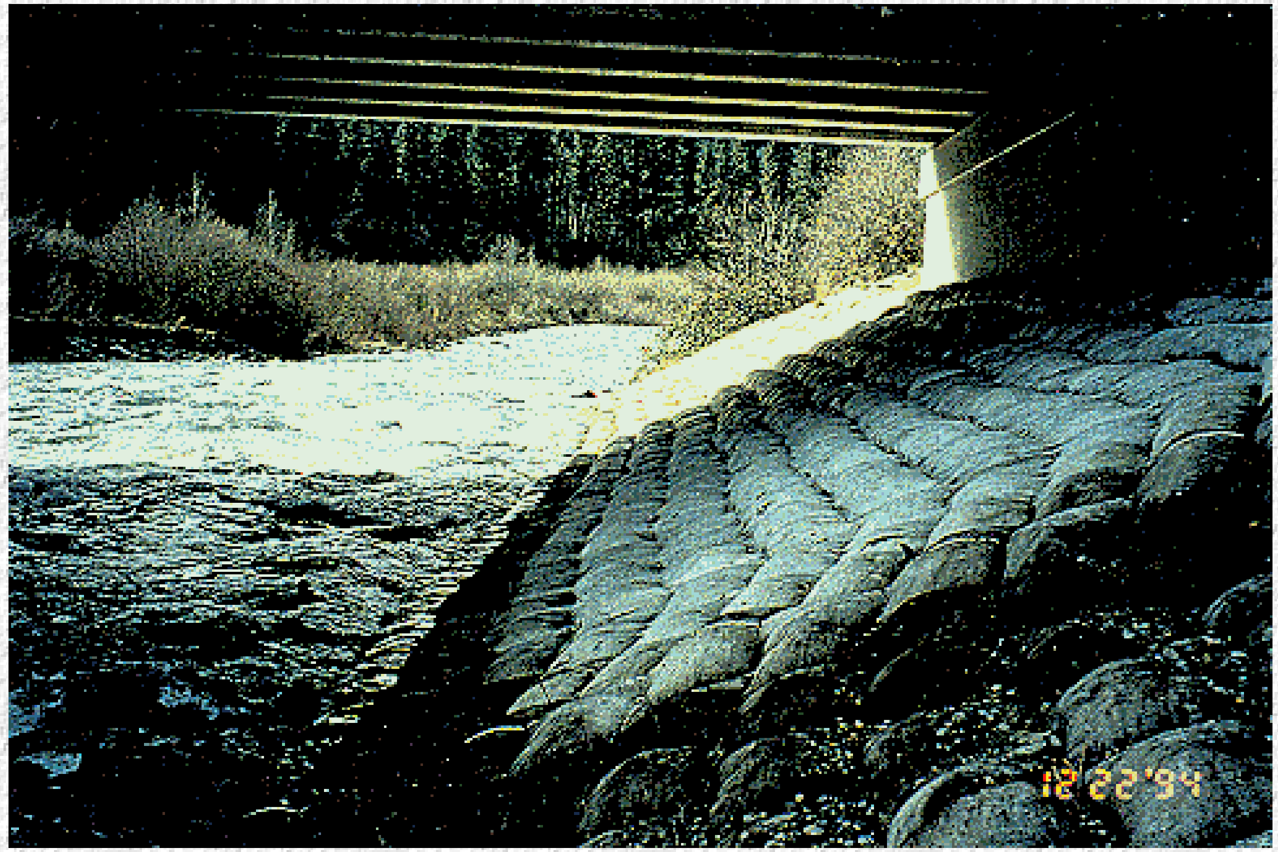


Figure 5.4 ABM Underneath Salmon Creek Bridge (ODOT)

Figure 5.5 ABM Installed on West Bank of Salmon Creek (ODOT)

Specifications

Specifications on fabric forms were not provided by the states. However specifications on the tensile and tear strength of fabric used for grout bags can be found in Design Guideline 7.

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Go to Design Guideline 6

Design Guideline 6 : HEC 23

Toskanes

Go to Design Guideline 7

Introduction

Toskanes are concrete armor units which are used as a replacement for riprap (see **Figure 6.1**). In cases where rock can't be found in suitable sizes, concrete armor units have the advantage that they can be constructed to meet the design size, mass, and number of units required to provide protection. Concrete armor units have been used in coastal applications where very large riprap would be required to resist the impact forces generated by waves.

Background

The Pennsylvania Department of Transportation (PennDOT) contracted with Colorado State University (CSU) in 1992 to investigate a concrete tetrapod as a countermeasure for local scour at bridge piers. The purpose of the research was to develop guidelines for selection and placement of cost-effective tetrapod sizes to mitigate pier scour. A literature review of concrete armor units used in coastal and river protection works led to the selection of the Toskane as the primary concrete armor unit for which guidelines where to be developed. The Toskanes were modified from those used in coastal applications by removing the pointed corners from the hammerheads, increasing the length and cross section of the beam, and including reinforcing steel in the beam.

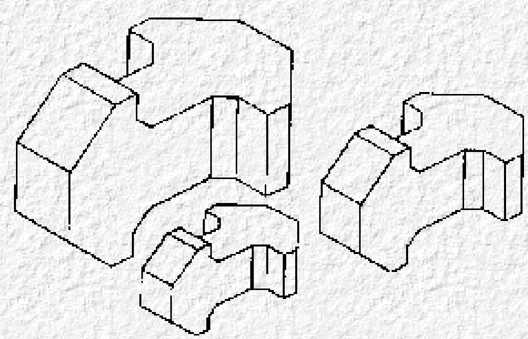


Figure 6.1 CSU Toskanes

Hydraulic tests to evaluate the performance of Toskanes were conducted in an indoor flume and two outdoor flumes at CSU. Over 400 test runs were conducted. These tests included random and pattern placement of Toskanes tested to failure around piers and abutments, determination of protective pad radius, determination of pad height (comparing installations in which the top of the pad was level with the bed and installations in which the pad protruded above the bed), comparison of gravel and geotextile filters, number of Toskanes per unit area, and effect of angle of attack on Toskanes at a round nose pier. The data were analyzed and using dimensional analysis the significant parameters were determined. The following design guidelines reflect the results of the research conducted at CSU (Fotherby and Ruff 1995):

Design Guidelines

1. Determine the velocity:

   a. Calculate the average velocity of the river directly upstream of the bridge (approximately 3 m upstream). Consider the number of substructure elements in the flow at the bridge cross section. If constriction could be significant, increase the approach flow velocity accordingly.

   V_o = average velocity directly upstream of the bridge (m/s)

   b. Select an adjustment coefficient to account for the location of the pier or abutment within the cross section. Some judgment is needed for selecting the coefficient, C_1, but generally a coefficient at 1.0 to 1.1 can be used.

   C_1 = 0.9, for a location near the bank of the river.
   C_1 = 1.0, for most applications
   C_1 = 1.1, for a structure in the main current of flow at a sharp bend.
   C_1 = 1.2, for a structure in the main current of the flow around an extreme bend, possible cross flow generated by adjacent bridge abutments or piers.

   NOTE: HEC-18 (Richardson and Davis 1995) recommends values of C_1 as large as 1.7 (see Design Guideline 8).

   Alternatively, a hydraulic computer model could be used to determine the local velocities directly upstream of bridge piers or abutments. A 1-dimensional hydraulic model (i.e., HEC-RAS, WSPRO) could be used to compute velocity distributions within in a cross section on a relatively straight reach. A 2-dimensional hydraulic model (i.e., RMA-2V, FESWMS) could be used to estimate local velocities in meandering reaches or reaches with complex flow patterns.

   c. Select an adjustment coefficient for shape of the pier or abutment. As with the CSU equation for pier scour, if angle of attack, α, is greater than 5°, set all shape coefficients to 1.0.

   For piers:
   C_s = 1.0, for a circular pier.
   C_s = 1.1, for a square nose pier.
   C_s = 0.9, for a sharp nose pier streamlined into the approach flow.

   For abutments:
   C_s = 1.1, for a vertical wall abutment.
   C_s = 0.85, for a vertical wall abutment with wingwalls.
   C_s = 0.65, for a spill through abutment.

   d. Determine if the top surface of the pad can be placed level with the channel bed and select the appropriate coefficient.

   C_h = 1.0, Level - Top of pad is flush with the channel bed.
   C_h = 1.1, Surface - Two layers of pad extend above channel bed.

   NOTE: This is not a correction for mounding. Mounding is strongly discouraged because it generates adverse side effects. The effects of mounding were not addressed in the CSU study. Pad heights were kept at 0.2 times the approach flow depth or less.

   e. Select a random or pattern installation for the protection pad. A random installation refers to the units being dumped into position. In a pattern installation, every Toskane is uniformly placed to create a geometric pattern around the pier.

   C_i = 1.0, Random Installation
   C_i = 0.9, Pattern 1 - 2 Layers with Filter
   C_i = 0.8, Pattern 2 - 4 Layers

   f. Calculate the Velocity Value:
   Multiply the average approach flow velocity and coefficients by a safety factor of 1.5.

$$V_v = 1.5 V_o C_1 C_s C_h C_i$$

2. Calculate adjusted structure width, b_a (m).

For a pier:

a. Estimate angle of attack for high flow conditions.

b. If the angle is less than 5°, use pier width b as the value b_a.

c. If the angle is greater than 5°, calculate b_a:

$$b_a = L \sin \alpha + b \cos \alpha$$

where L = length of the pier (m), b = pier width (m), b_a = adjusted structure width (m), α = angle of attack.

d. If a footing extends into the flow field a distance greater than:

0.1 * y_o (approach flow depth)

use footing width instead of pier width for b.

e. For an abutment:
Estimate the distance the abutment extends perpendicular to the flow (b) during high flow conditions.

if $b \leq 1.5$ m, then b_a = 1.5m

If 1.5 m $\leq b \leq$ 6 m, then $b_a = b$.

if $b_a \geq$ 6 m, then b_a = 6 m.

3. Select a standard Toskane size, D_u, using the design equation or nomogram on **Figure 6.2** using the calculated velocity value, V_v, and the adjusted structure width, b_a. D_u represents the equivalent spherical diameter of riprap that would be required. This parameter can be related to dimensions of the Toskane by D_u = 0.622H, where H is the length of the Toskane (see **Figure 6.3**). It should be noted that the nomogram reports results in centimeters (cm) and the equation presented in Figure 6.2 solved for D_u has the units of meters (m) when all other parameters are in meters.

Check the b_a / D_u ratio using the diameter, D_u, of the standard Toskane size. If the ratio > 21, select the next largest size of Toskane. Repeat until ratio < 21.

4. Select pad radius, ℓ (m).
1.5 b_a for most piers and 2.0 b_a for most abutments.
Use a larger pad radius if:

- uncertain about angle of attack
- channel degradation could expose footing,
- uncertain about approach flow velocity
- surface area of existing scour hole is significantly larger than pad.

If more than one Toskane pad is present in the stream cross section, check the spacing between the pads. If a distance of 1.5 m or less exists between pads, extend the width of the pads so that they join.

5. Select the number of Toskanes per unit area from the nomogram on Figure 6.2 or Toskane detail sheet Figure 6.3.

a. Determine the protection pad thickness. Pads with randomly placed units have to be a minimum of two layers thick.

b. For a two layer pad with a filter, select a pad thickness from the nomogram or Toskane detail sheet.

6. If bed material is sand, gravel, or small cobbles, add a cloth or granular filter. Toe in or anchor the filter. If the filter is granular, the d_{85} of the filter material directly below the Toskane layer can be read from the nomogram or Toskane detail. Additional layers of filter, that may be needed based on the gradation of the bed material, can be designed according to standard requirements. Additional guidelines on the selection and design of filter material can be found in Brown and Clyde (HEC-11) (1989) and Holtz et al. (FHWA HI-95-038) (1995).

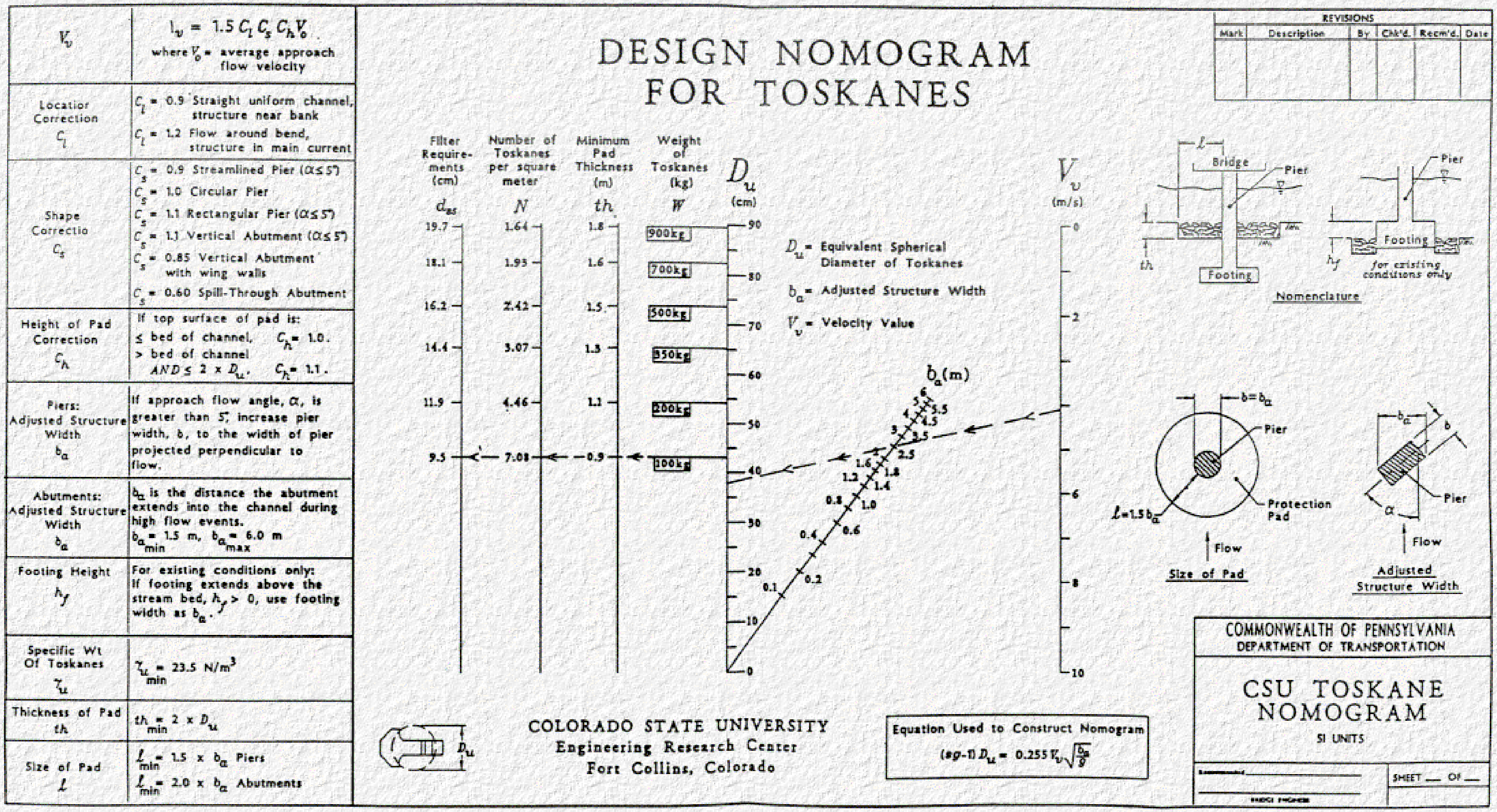
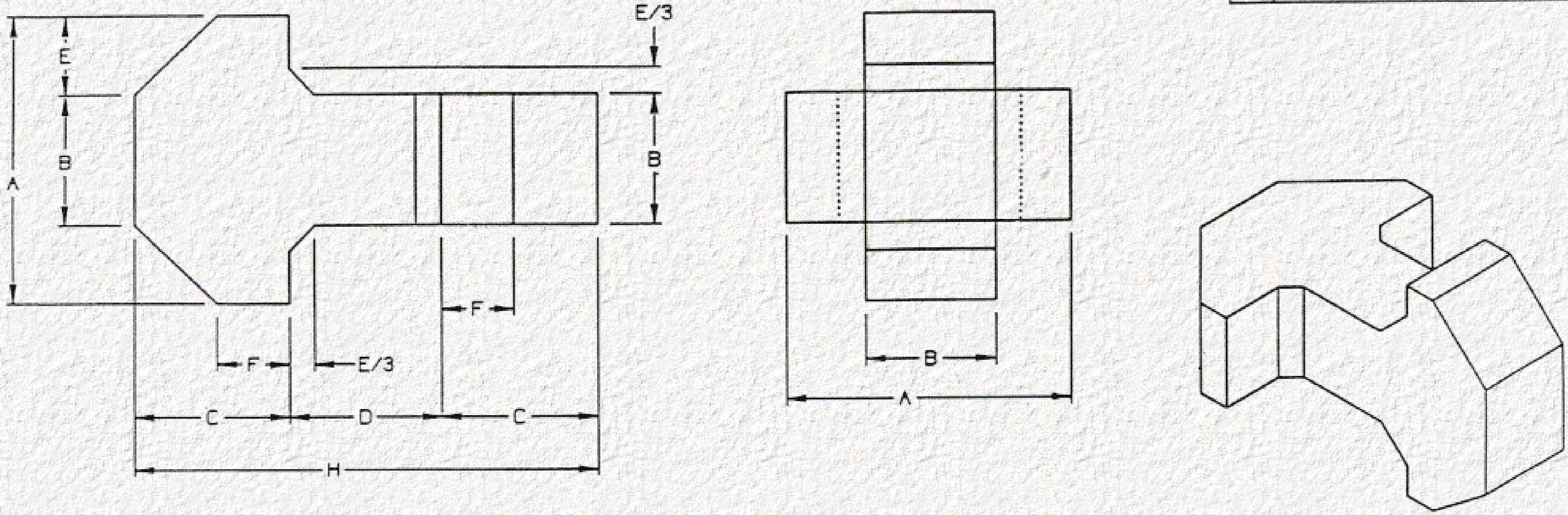


Figure 6.2 CSU Toskane Nomogram

CSU TOSKANE DESIGN PARAMETERS AND DIMENSIONS (SI UNITS)

REVISIONS

Mark	Description	By	Chk'd.	Recm'd.	Date



ISOMETRIC VIEW

DESIGN PARAMETER						DESIGN DIMENSIONS						
MASS 2408 kg/m³	VOLUME ¥ (0.1263H³)	NUMBER PER UNIT AREA $N=0.85¥^{2/3}$	D_u† (0.622H)	2 LAYER THICKNESS 1.24H	FILTER REQUIREMENTS $d_{85} = 0.22D_u$	H	A (0.616H)	B (0.280H)	C (0.335H)	D (0.330H)	E (0.168H)	F (0.156H)
kg	m³		cm	m	cm	cm	cm	cm	cm	cm	cm	cm
50	0.0208	11.24	34.2	0.7	7.5	54.8	33.8	15.4	18.4	18.1	9.2	8.6
* 100	0.0416	7.08	43.0	0.9	9.5	69.1	42.5	19.3	23.1	22.8	11.6	10.8
150	0.0624	5.40	49.3	1.0	10.8	79.1	48.7	22.1	26.5	26.1	13.3	12.3
* 200	0.0832	4.46	54.2	1.1	11.9	87.0	53.6	24.4	29.2	28.7	14.6	13.6
250	0.1040	3.84	58.4	1.2	12.8	93.7	57.7	26.2	31.4	30.9	15.7	14.6
300	0.1248	3.40	62.0	1.2	13.6	99.6	61.4	27.9	33.4	32.9	16.7	15.5
* 350	0.1457	3.07	65.3	1.3	14.4	104.9	64.6	29.4	35.1	34.6	17.6	16.4
400	0.1665	2.81	68.3	1.4	15.0	109.6	67.5	30.7	36.7	36.2	18.4	17.1
450	0.1873	2.60	71.0	1.4	15.6	114.0	70.2	31.9	38.2	37.6	19.2	17.8
* 500	0.2081	2.42	73.5	1.5	16.2	118.1	72.8	33.1	39.6	39.0	19.8	18.4
* 700	0.2913	1.93	82.3	1.6	18.1	132.1	81.4	37.0	44.3	43.6	22.2	20.6
* 900	0.3745	1.64	89.4	1.8	19.7	143.7	88.5	40.2	48.1	47.4	24.1	22.4

† D_u = EQUIVALENT SPHERICAL DIAMETER $(6¥/\pi)^{0.33}$
*RECOMMENDED SIZES

COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION

CSU TOSKANE
DETAIL
SI UNITS

SHEET ___ OF ___

Figure 6.3 CSU Toskane Detail

Design Example for a Bridge Pier (Fotherby & Ruff 1995)

A bridge over Blue Creek has a single pier located on the outside of a bend (see **Figure 6.4**). The pier is round nosed and is 1 m wide and 6 m long. The footing is not exposed and bed material is cobbles and gravel. The average velocity directly upstream of the bridge during high flow is 2.5 m/s and has an angle of attack of 15°.

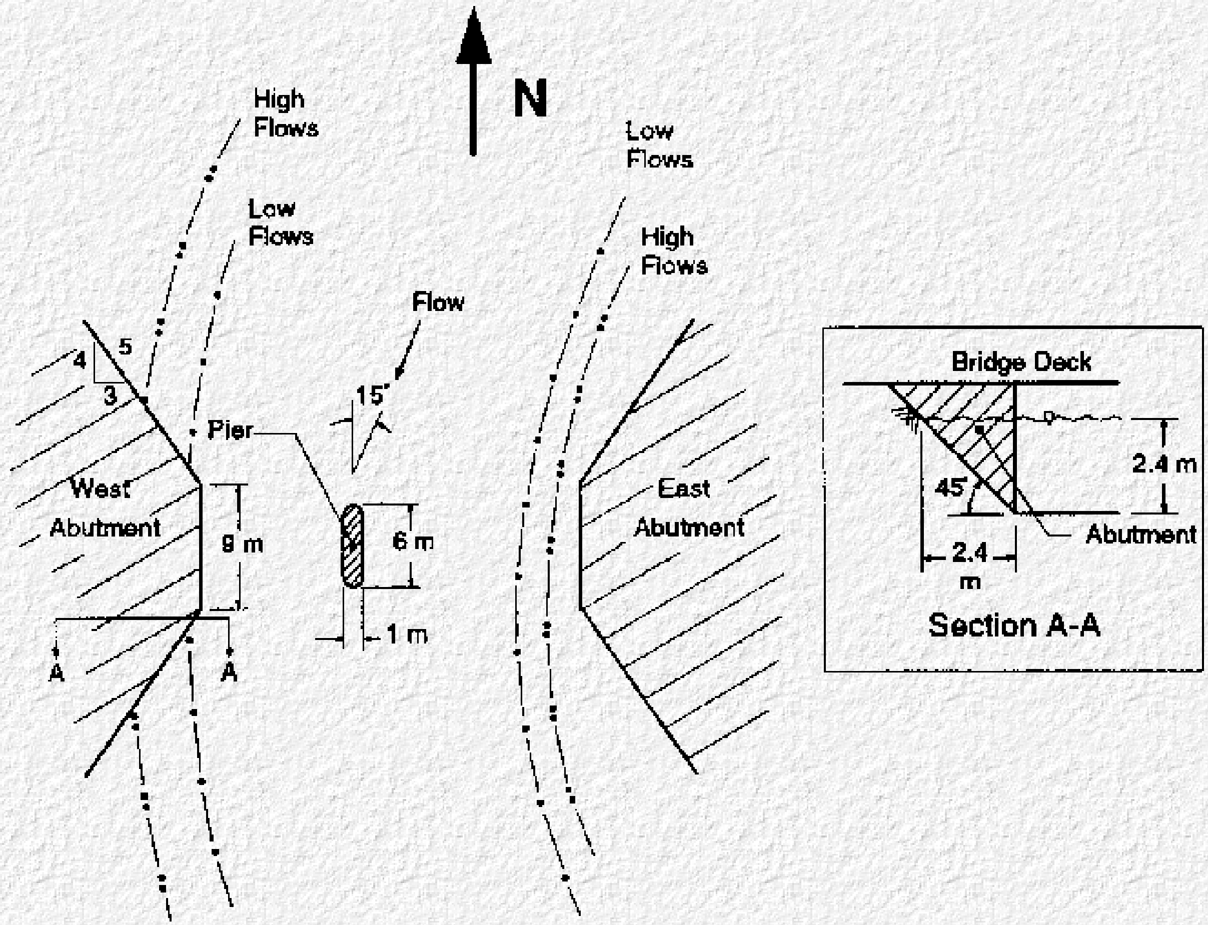


Figure 6.4 Blue Creek Site

1. Determine the velocity value, V_v (m/s).

   Because the pier is located in the thalweg of the bend, C_1 = 1.1.

   The angle of attack, α = 15° > 5°, therefore C_s = 1.0.

   The Toskane pad is installed so that the top of the pad is level with the bed, C_h = 1.0.

   A randomly installed pad of Toskanes is selected, C_i = 1.0.

   V_v = (1.5)(2.5)(1.1)(1.0)(1.0)(1.0) = **4.1 m/s**.

2. Calculate the adjusted structure width, b_a (m).

   The angle of attack, α = 15°.
   Length of pier, L = 6 m.
   Pier width, b = 1 m.

$$b_a = L\sin\alpha + b\cos\alpha = 6\sin(15^\circ) + 1\cos(15^\circ) = \mathbf{2.5\ m}$$

3. From Figure 6.2 or the design equation calculate the equivalent spherical diameter, D_u, for V_v = 4.1 m/s and b_a = 2.5 m.

$$(1.4)D_u = 0.255(4.1)\sqrt{2.5/9.806} \Rightarrow \boldsymbol{D_u} = \mathbf{0.377\ m = 377\ mm}$$

Using standard sizes, a 100 kg Toskane unit (D_u = 430 mm) is selected. The ratio b_a/D_u = 2.5/0.43 = 5.8 < 21, therefore the size is acceptable.

4. Since the engineer is confident about the flow velocity and angle of attack, and the channel is not expected to experience any vertical instability, a pad radius of ℓ = 1.5b_a was chosen.

   Pad Radius, ℓ = 1.5(2.5) = **3.75 m**.

   The Toskanes will be installed around the pier, a horizontal distance of 3.75 m from the wall of the pier.

5. From Figure 6.2 or 6.3, the number of Toskanes per unit area for the 100 kg Toskane size with a pad thickness of 2D_u is 7.08 Toskanes/m^2 Total area of the pad (see **Figure 6.5**) is:

   Area = 2(5(3.75)) + (π (4.25^2 - 0.5^2)) = **93.5 m^2**.

   #Toskanes = 7.08(93.5) = **662 Toskanes**.

   The pad thickness is 2D_u = **900 mm**.

6. Since the bed material is cobbles and gravel, a granular filter is added beneath the pad of Toskanes. The d_{85} of the filter directly beneath the pad of Toskanes is 95 mm. The cobbles and gravel are sufficiently large so no additional filter layers are required.

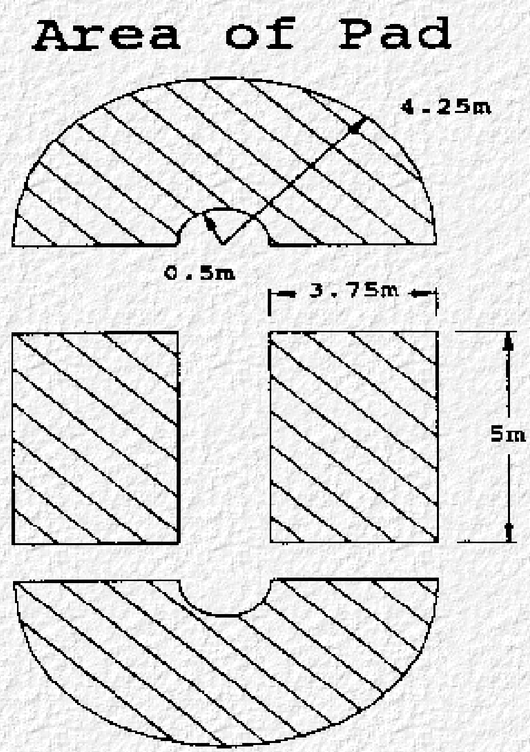


Figure 6.5 Area of Pier Pad

Design Example for a Bridge Abutment (Fotherby & Ruff 1995)

The bridge at Blue Creek in Figure 6.4 has vertical wall abutments with wing walls. During normal flows the west abutment extends 0.6 m into the flow, but during high flows it obstructs 2.4 m of the flow (normal to the flow field). The embankment slope is at 1H:1V. The east abutment does not obstruct the flow even during high flows.

1. Determine the velocity value, V_v (m/s).
   The abutment is located near the bank, outside of the thalweg, C_1 = 0.9.
   Since the abutment has wing walls, C_s = 0.85.
   The Toskane pad is installed so that the top of the pad is level with the bed, C_h = 1.0.
   A randomly installed pad of Toskanes is selected, C_i = 1.0.

   V_v = 2.5(0.9)(0.85)(1.0)(1.0)(1.5) = **2.87 m/s**.
2. Calculate the adjusted structure width, b_a (m).

   Since the west river bank has a slope of 1H:1V, an average value is used for the length of abutment that projects perpendicular to the flow. The abutment extends 2.4 m at the water surface and 0 m at the channel bed (see Figure 6.4). Therefore an average value of:
   b_a = (2.4 + 0)/2 = 1.2 m.
   This is less than the minimum, therefore b_a = **1.5 m**.
3. From Figure 6.2 or the design equation calculate the equivalent spherical diameter, D_u, for V_v = 2.87 m/s and b_a = 1.5 m.

   $$(1.4)D_u = 0.255(2.87)\sqrt{1.5/9.806} \Rightarrow D_u = \mathbf{0.204\ m} = \mathbf{204\ mm}$$

   For the west abutment, the 100 kg Toskane is selected (D_u = 430 mm). A smaller 50 kg Toskane could have been selected, but this non-standard size may not be economical to manufacture.
4. Since the engineer is confident about the flow velocity and the channel is assumed vertically stable, a pad radius of ℓ = 2.0b_a is recommended.

   Pad Radius, ℓ = 2.0(1.5) = **3.0 m**.

   The Toskanes will be installed along the abutment and wingwalls a horizontal distance of 3 m from the wall. An additional 600 mm of pad will be added at the ends of the pad for freeboard.
5. The pad thickness is 2D_u which will result in 7.08 Toskanes/m^2. The total area of the pad (see **Figure 6.6**) is:

   Area = (3)(9) + 2(3)(5) + 2(0.5)(3)(1.8) + 2(0.5)(3)(4) = **74.4 m^2**.

   #Toskanes = (74.4)(7.08) = **527 Toskanes**.
6. A filter is placed under the pad for the bed material of cobbles and gravel. The d_{85} of a granular filter is 95 mm.

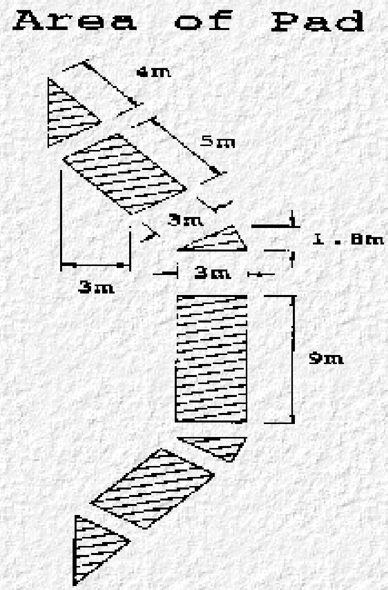


Figure 6.6 Area of Abutment Pad

The disadvantage to selecting an oversized unit is that larger units have larger voids which increase the opportunity for pumping of the bed material if a filter is not present. A well designed filter should be properly installed under the pad. More excavation is also required for the oversized Toskanes, but fewer Toskanes need to be manufactured for this design.

The distance between the pier and the west abutment is not specified in this example. If the spacing between the two protection pads is 1.5 m or less, it is recommended that the pads be joined to form a continuous pad between the abutment and the pier. **Figure 6.7** shows the recommended layout of Toskane protection pads.

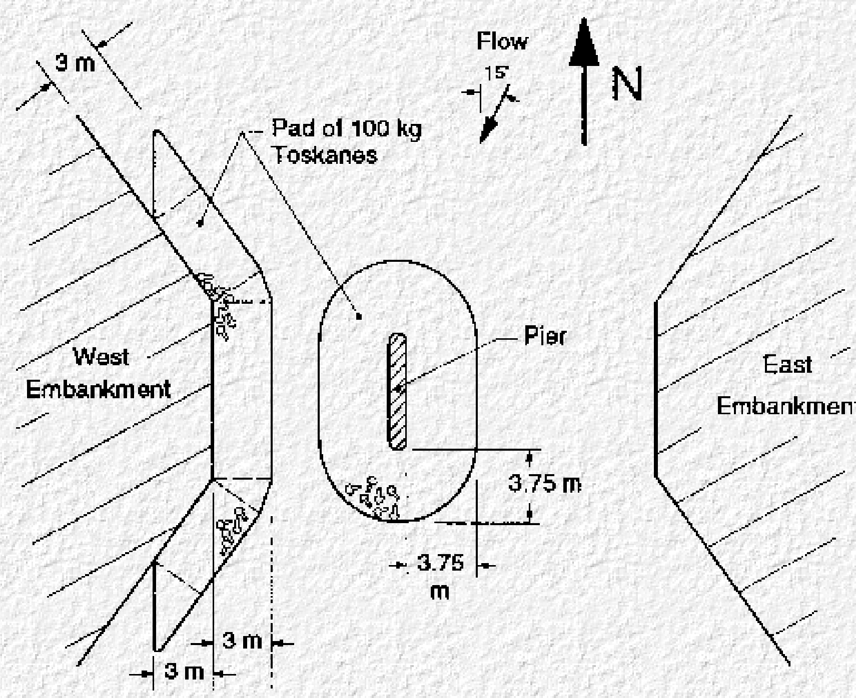


Figure 6.7 Blue Creek with Toskane Protection Pads

Information on Toskane fabrication and installation costs can be found in Fotherby and Ruff 1995 (PennDOT study).

References

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Go to Design Guideline 7

Design Guideline 7 : HEC 23
Grout/Cement Filled Bags

Go to Design Guideline 8

Introduction

Grout/cement filled bags have been used to protect stream banks in areas where riprap of suitable size and quality is not available at a reasonable cost. Guidelines for the use of bags (sacks) as a streambank revetment can be found in Richardson et al. (HIRE) (1990), Lagasse et al. (HEC-20) (1995) and Keown (1983).

Grout/cement filled bags have also been used as a countermeasure against scour at bridges. Historically they have been used to fill in undermined areas around bridge piers and abutments. As scour awareness increases, grout filled bags are being used to armor channels where scour is anticipated or where scour is detected. Whether they are implemented in a post- or pre-scour mode, grout bags are relatively easy to install and can shift to changes in the channel bed to provide effective scour protection.

Design Guidelines

A precise quantitative factor of safety design procedure is not normally completed for the design of grout filled bags. This type of design would be beneficial in determining the hydraulic stability of the bags, but historically this has not been done for grout filled bags. It would require a comparison of the hydraulic shear stress and the critical shear stress to uplift the grout bag as is done with riprap using discrete particle analysis. Information on hydraulic performance of grout bags at bridge piers can be found in Bertoldi et al. (1996) and Fotherby (1992). More often, engineering judgment is used to select a bag size that will not be removed by the flow and installation practices are more critical to the success of the system. Guidelines for the use of grout filled bags for bridge scour reflect information provided by the Maryland State Highway Administration (MDSHA).

Tips for Concrete Bag Installation (MDSHA):
(see attached Sheets 1 - 10)

1. It is preferable to place a single layer of bags instead of stacking. Filter Fabric should be placed under single layered bags that have the potential to settle away from each other. Guidelines on the selection and design of filter material can be found in Brown and Clyde (HEC-11) (1989) and Holtz et al. (FHWA HI-95-038) (1995).

2. If bags are stacked, overlap the joints of the preceding layer.

3. If possible, bags should be buried so that the top of the bag is at or below the stream bottom (see Sheet 3 of 10). When filling a scour hole, keep the top of the bag at or below the stream bottom, if possible (see Sheet 5 of 10).

4. Do not tie bags together with reinforcing steel or by any other means. Allow bags to settle to a state of equilibrium individually. (This differs from specifications recommended by the State of Maine where stitching bags together is a recommended procedure for protection of undermined areas at piers)

5. Excessively large bags, one side greater than 4.6 m (15 ft), are more susceptible to undermining because they do not tend to settle or shift into place as scour develops.

6. Small bags, no side greater than 1.5 m (5 ft) , tend to settle and conform to the bottom.

7. The bag placed directly in front of the nose of the pier should be the width of the exposed portion of the pier (See Sheets 8 and 10 of 10). This is the area with the greatest turbulence. Overlapping of bags is important here. Any open gaps between bags can allow sediment under the bags to be eroded causing undermining of the bags. Geotextile fabric at this location would also help eliminate the possibility of undermining. Similarly, no gaps should be allowed to form between the bags and the front face of the footing.

8. The concrete bags should cover the stream bottom around the pier for a distance of 1.5 times the width of the exposed portion of the pier or a minimum of 1.8 m (6 ft) whichever is greater.

9. Use a cutoff wall along the entrance and trail end of the concrete bags that extend across the entire stream channel, if possible.

10. Where there is a potential for continued scour along newly installed concrete bags in a wide stream channel, use a cutoff wall or a fabric hinge to protect the bags against undermining.

Concrete Bag Installation and Grouting of Undermined Area at Piers and Abutments (MDSHA):

(see Sheets 2 and 6 to 10 of 10)

1. Depending on the depth of the undermining, place one concrete bag or stack several layers of concrete bags along the face of the abutment or pier in front of the undermined area.

2. Once the vent/fill pipes have been installed and the bags are filled, pump the grout into the undermined area. Cut the vent/fill pipes flush with the top of the bags after the pumping operation is complete. Debris could get caught up on these pipes and cause additional scour if left exposed.

3. Adequate venting of the water to be displaced in the undermined area is important. The water must be able to escape as it is displaced by the grout pumped into the cavity. A 1.2 m (4 ft) maximum spacing of the vent/fill pipes is recommended.

4. It is important to keep the nozzle buried in the grout during pumping. This is to reduce the amount of mixing of the grout and the water to be displaced.

5. Debonding jackets should be placed around piles to prevent the grout from adhering to the piles if the added weight from the grout would cause a significant reduction in the pile capacity.

6. If possible, clean out unstable material along the bottom of the undermined area prior to filling with grout. This would reduce the amount of loose sediment discharged through the vent pipes.

7. Pumping grout in the undermined area under a footing is not an underpinning for the footing. This is done only to fill the void area and stop the fill material located behind the footing from settling into the void area resulting in settlement of the roadway behind the structure.

Specifications (MDSHA)

Grout:

Portland cement concrete shall consist of nine bags, 55.8 kg/m^3 (94 lb per cubic yard) Type II Portland cement, air entrainment, 6 ± 1 percent mortar sand aggregate, and water so proportioned to provide a pumpable mixture. The 28 day minimum day strength shall be 24 140 kPa (3500 psi).

Bags:

Fabric bags shall be made of high strength water permeable fabric of nylon or cordura. Each bag shall be provided with a self closing inlet valve, to accommodate insertion of the concrete hose. A minimum of two valves shall be provided for bags more than 6.1 m (20 ft) long. Seams shall be folded and double stitched.

Dowels:

Reinforcing steel dowels, if specified on the plans, shall conform to ASTM A 615, Grade 60 and shall be epoxy coated.

Fabric:

Fabric shall exhibit the following properties in both warp and fill directions:

Tensile Strength, min.	70 kN/m	(400lb/in)	ASTM D 1682, Grab Method
Tear Strength, min	400 N	(90 lb)	ASTM D 2262, Tongue Method

Construction:

The bags shall be positioned and filled so that they abut tightly to each other and to the substructure units. Joints between bags in successive tiers shall be staggered.

Fabric porosity is essential to the successful execution of this work. Suitability of fabric design shall be demonstrated by injecting the proposed mortar mix into three 610 mm (2 ft) long by approximately 150 mm (6 in) diameter fabric sleeves under a pressure of not more than 103 kPa (15 psi) which shall be maintained for not more than 10 minutes. A 300 mm (12 in) long test cylinder shall be cut from the middle of each cured test specimen and tested in accordance with ASTM C 39. The average seven day test compressive strength of the fabric form shall be at least higher than that of companion test cylinders made in accordance with ASTM C 31.

Standoffs to provide a uniform cross section shall be used.

Ready mixed high strength mortar may be permitted by written permission of the Engineer. The ready mixed high strength mortar shall be furnished by a manufacturer approved by the Laboratory and the plan, equipment, etc., shall be subject to inspection and approval.

The concrete pump shall be capable of delivering up to 19 m^3/hr (25 yd^3/hr).

Supplemental Observations on Grout Bags (MDSHA)

Design of Bags

Bags should be designed and constructed as flat mats, 0.9 m to 1.2 m (3 to 4 feet) wide and about 0.3 m (1 foot) thick. The bag lengths should be on the order of 1.2 m to 2.4 m (4 to 8 feet). Bags should not be filled to the point that they look like stuffed sausages, since they will be much more vulnerable to undermining and movement, and will not fit properly into the mat.

Both the designer and the installer should understand how the mat is expected to perform. Each bag should be independent of other bags so that it is free to move; however, the bag should be snugly butted against adjoining bags to minimize gaps in the mat. This concept will result in a semi-flexible mat that will be able to adjust to a degree to changes in the channel bed. The mat should not be constructed as a rigid monolithic structure. It would be helpful to have a pre-construction conference with the designer, contractor and the State inspector.

The bags should be sized and located in accordance with the SHA Standards for the particular type of foundation and condition of scour. It is recommended that the type of grout bag installation and its design be reviewed by an engineer with experience in evaluating scour at bridges.

Installation

Careful attention should be given to preparation of the bed on which the bags are to be placed. **Where the bed is uneven, such as might occur in scour holes, best results will be obtained by planning for a sequence of placement of the bags so that each bag adds to the support of the other bags**. This is particularly important in locations where several layers of bags are to be placed. It is unlikely that detailed plans will be developed for such locations, and the integrity of the installation will depend on the skill of the persons placing the mat. If the bed is highly irregular, appropriate modification of the bed and removal of obstacles should be accomplished prior to placement of the bags.

Each bag should butt up firmly against its neighbor to provide a tight seal and to minimize the occurrence of gaps between bags. Particular attention should be given to obtaining this tight seal between the foundation and the first row of bags.

For piers, the bags should extend to a distance of 1.5 to 2 times the pier width on both sides as well as upstream of the pier nose and downstream of the pier end.

For abutments, the best results are obtained for most locations by placing the bags the full length along the upstream wingwall, abutment backwall and downstream wingwall to form a solid mat. As an interim guide, the mat width for abutments is recommended to be on the order of 1.8 m to 2.4 m (6 to 8 feet), depending upon the particular site conditions. This arrangement provides for a smooth streamlined design that locates the ends of the mat away from the main stream current or thalweg. Of course, there are a wide variation of conditions at abutments and each location needs to be designed for the site conditions.

In some cases, it may be necessary to provide for both grout bags and rock riprap to provide the desired degree of scour protection. As a general rule, however, it is preferable to provide either riprap or grout bags but not both at any one pier or abutment.

For small structures such as bridges or "bottomless" culverts with spans in the range of 4.6 m to 7.6 m (15 to 25 feet), there are essentially two choices for the design of the bags:

- Place the bags full width under the structure
- Place the bags along each abutment/ wingwall, leaving the center of the channel unprotected.

If the center channel is unprotected, it can be expected to scour. This may result in undermining and displacement of the bags next to the channel or possibly of the whole installation. As an interim guide, it is suggested that consideration be given to lining the entire channel if more than half of the channel would be covered by grout bags placed along the abutments. **If the bags extend across the entire channel, attention needs to be given to the treatment of the upstream and downstream ends of the bag to avoid undermining and displacement.**

Filter Cloth

The following interim guidance is provided with regard to use of filter cloth:

Filter cloth should generally be used at locations where the bags are placed in a single layer along a level plane on the channel bed or flood plain. The filter cloth provides for additional support and stability in the event that the bags are subjected to undermining or movement as a result of scouring and hydraulic forces.

Where grout bags are placed in layers in a trenched condition, such as might occur in a scour hole, there is probably less need to provide for the filter cloth. At this point, however, it is recommended that the decision to eliminate filter cloth be made on a case by case basis. The general rule should be to place filter cloth under the grout bags.

Undermined Foundations

Grout bags provide for an efficient, cost effective means of underpinning foundations that have been scoured down below the bottom of the footing. General guidance on placement of bags and procedures for grouting the voids under the footing has been developed by MSHA in standard drawings.

Appearance

If grout bags are placed underwater, they are barely noticeable. A well designed and installed grout bag mat exposed to view under a bridge can be expected to have a streamlined and pleasing appearance. At some sites, the mats become covered with silt and are barely distinguishable from the channel banks or bed. Grout bags placed along wingwalls are usually exposed to the sun. Bags in these locations are likely to be covered by vegetation, especially when they have been covered by silt during high water events.

There were a few sites visited where the bags had an ungainly appearance. In most cases, these were bags that were pumped so full that they looked like sausages. Other reasons for a poor appearance include inadequate attention to design, installation, preparation of the bed on which the mat is placed, or a combination of these factors.

Early installations included bags with lengths of 4.6 m (15 feet) or more. In some cases, the bags were too long to fit properly into a compact mat. Use of shorter bags should help to minimize this problem in future installations.

Specifications for grout bags for undermined areas at piers were also provided by the State of Maine Department of Transportation as follows:

The underwater grout bags shall be fabricated based on the dimensions of the existing voids to be filled. Bags should be on the order of 900 mm to 1.2 m (3 to 4 ft) wide and 1.8 to 2.4 m (6 to 8 ft) long. Bags shall be securely placed to form a perimeter bulkhead to partially fill and enclose the substructure void. Grout shall be pumped to uniformly fill the secured bag with sufficient restraint so as to not rupture the bag. Consecutive bag placement shall be in accordance with the manufacturer's requirements. At a minimum this will require: placement of reinforcing bar between successive layers, stitching together adjacent bags with an overlapping splice (where accessible), and covering holes left by grout and other inserts.

NOTE: The State of Maine recommends stitching bags together for protection of undermined areas at piers. This procedure conflicts with the guideline provided by the State of Maryland in Tips for Concrete Bag Installation number 4.

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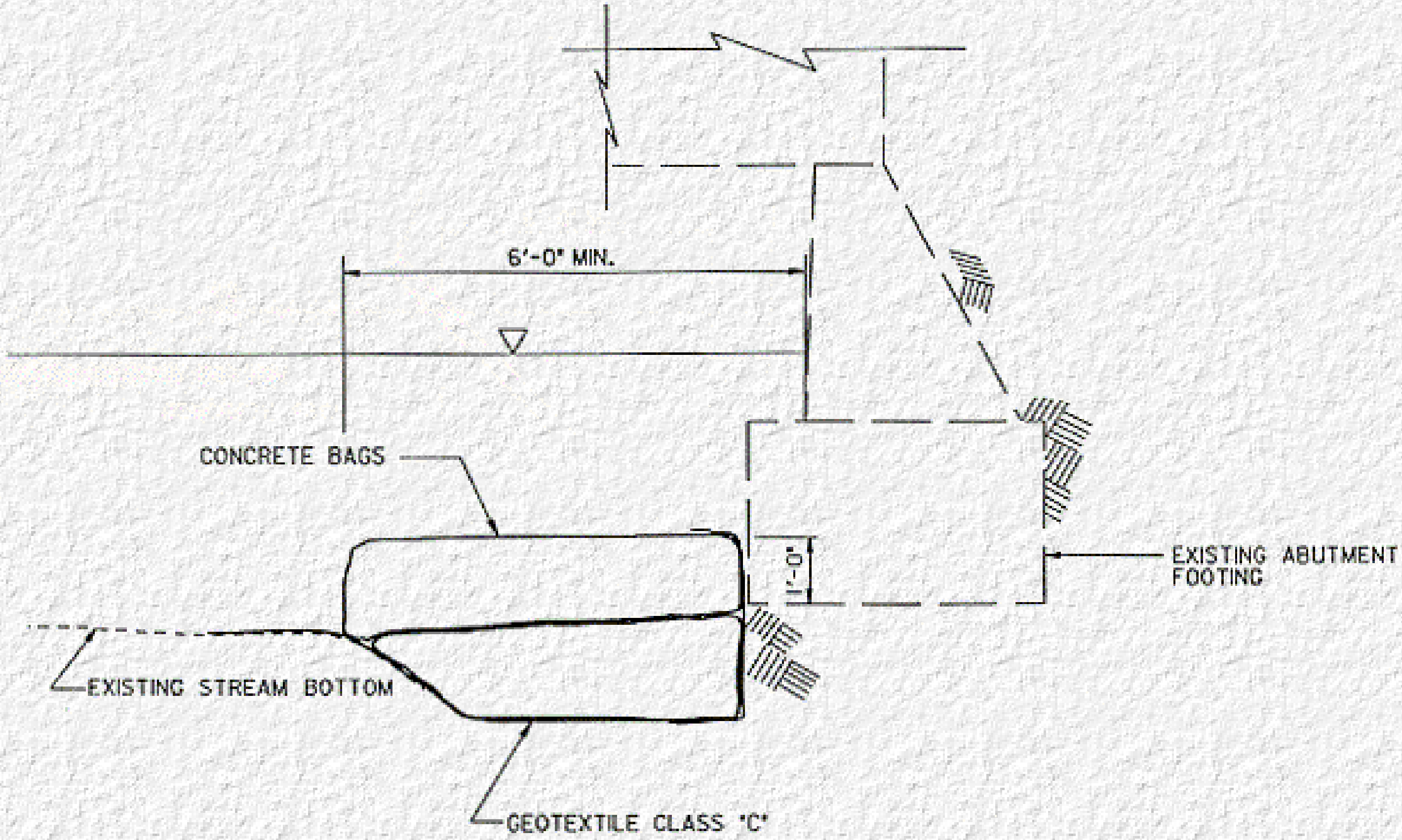
Go to Design Guideline 8

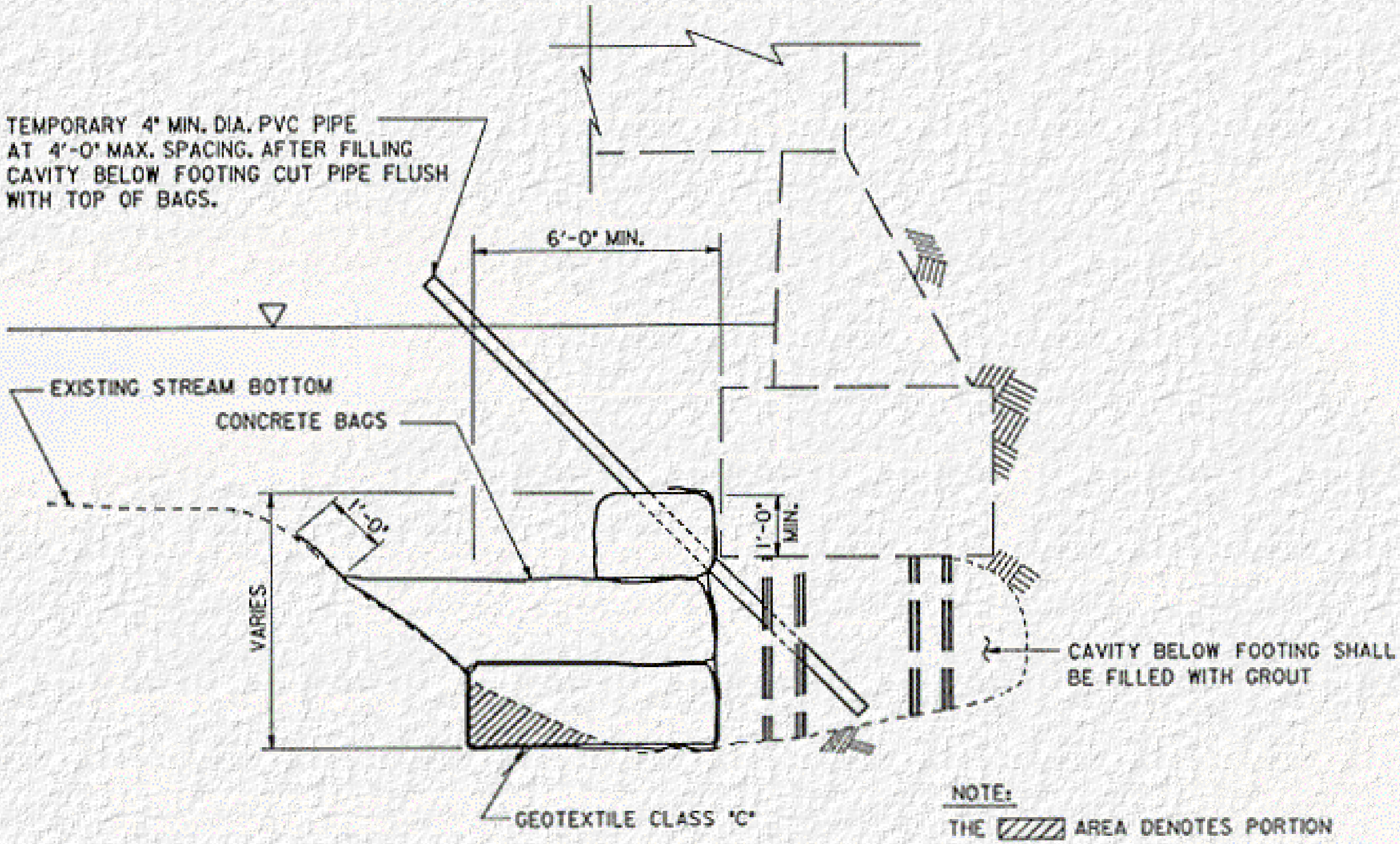


SECTION THRU ABUTMENT – NO PILES

SHEET NO. 1 OF 10

REVISIONS	
	MARYLAND DEPARTMENT OF TRANSPORTATION STATE HIGHWAY ADMINISTRATION BRIDGE INSPECTION AND REMEDIAL ENGINEERING CASE WHERE SCOUR HAS OCCURRED AT ABUTMENT
	PROVED:
	CHIEF, BRIDGE INSPECTION AND REMEDIAL ENGINEERING DIVISION

Attachment 1

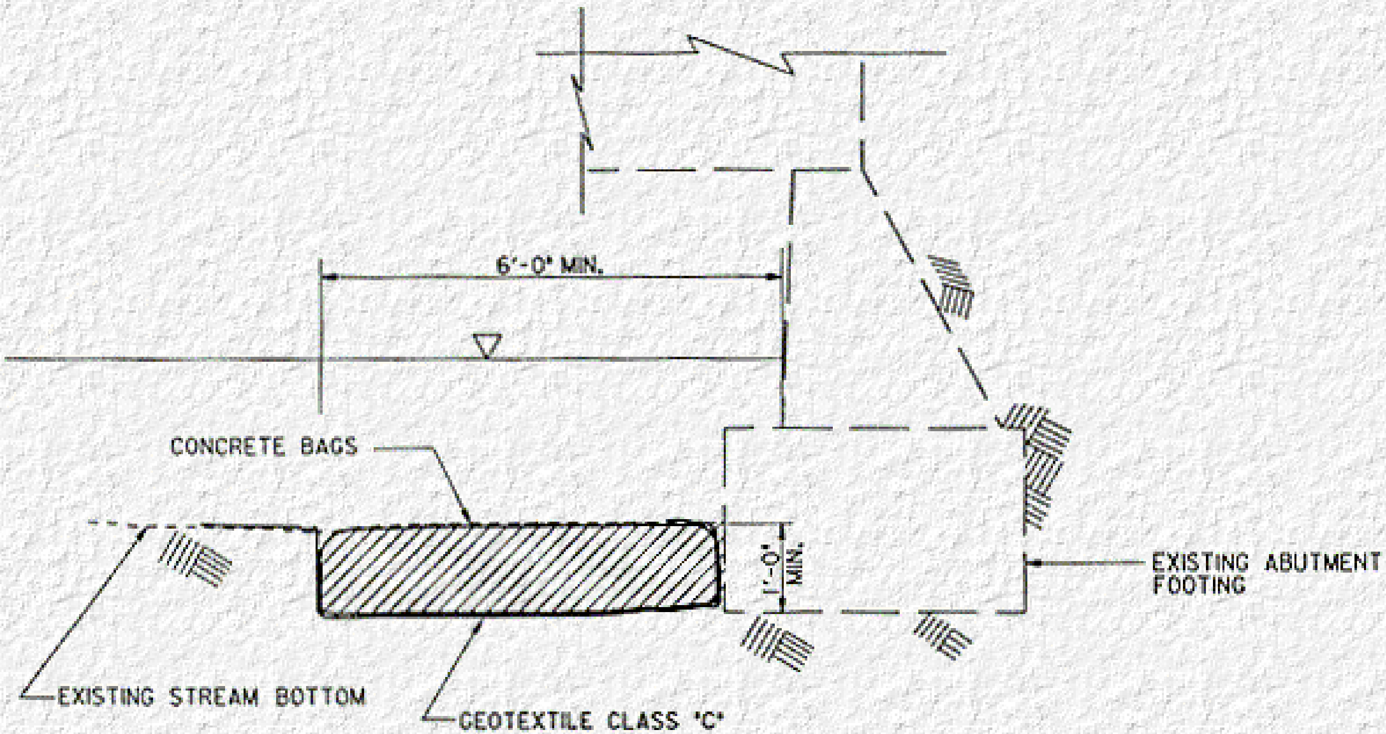


SECTION THRU ABUTMENT – ON PILES

SHEET NO. 2 OF 10

REVISIONS	MARYLAND DEPARTMENT OF TRANSPORTATION STATE HIGHWAY ADMINISTRATION BRIDGE INSPECTION AND REMEDIAL ENGINEERING CASE WHERE SCOUR AND UNDERMINING HAS OCCURRED AT ABUTMENT
	'PROVED:
	CHIEF, BRIDGE INSPECTION AND REMEDIAL ENGINEERING DIVISION

Attachment 2



NOTE:
DO NOT EXCAVATE BELOW BOTTOM OF FOOTING
TO ACCOMMODATE CONCRETE BAGS.

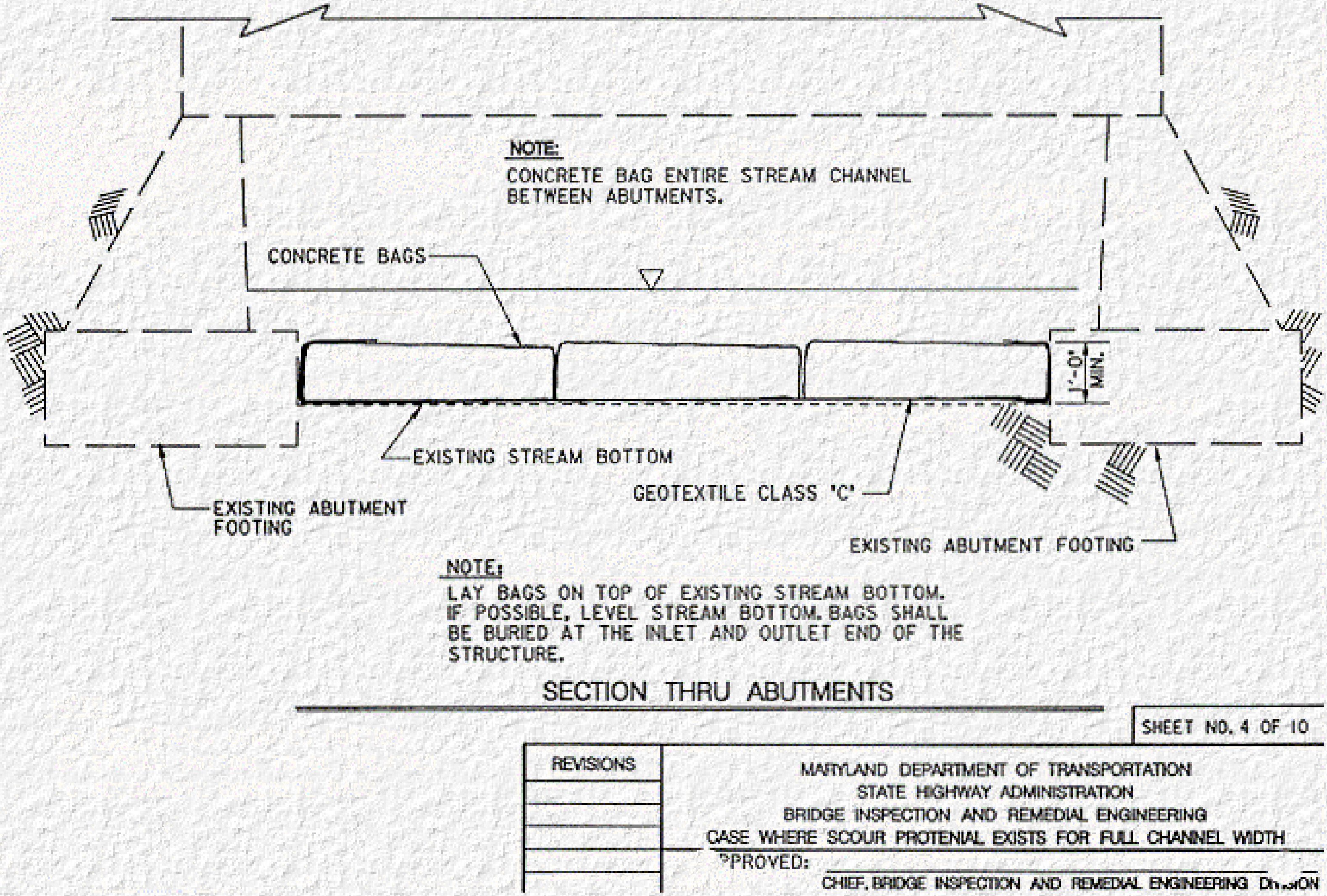
SECTION THRU ABUTMENT – NO PILES

NOTE:
THE ▨ AREA DENOTES PORTION
OF STREAM BOTTOM TO BE AIRLIFTED
TO ACCOMMODATE CONCRETE BAGS

SHEET NO. 3 OF 10

REVISIONS	MARYLAND DEPARTMENT OF TRANSPORTATION STATE HIGHWAY ADMINISTRATION BRIDGE INSPECTION AND REMEDIAL ENGINEERING CASE WHERE SCOUR PROTENIAL EXISTS AT ABUTMENT
	PROVED:
	CHIEF, BRIDGE INSPECTION AND REMEDIAL ENGINEERING DIVISION

Attachment 3



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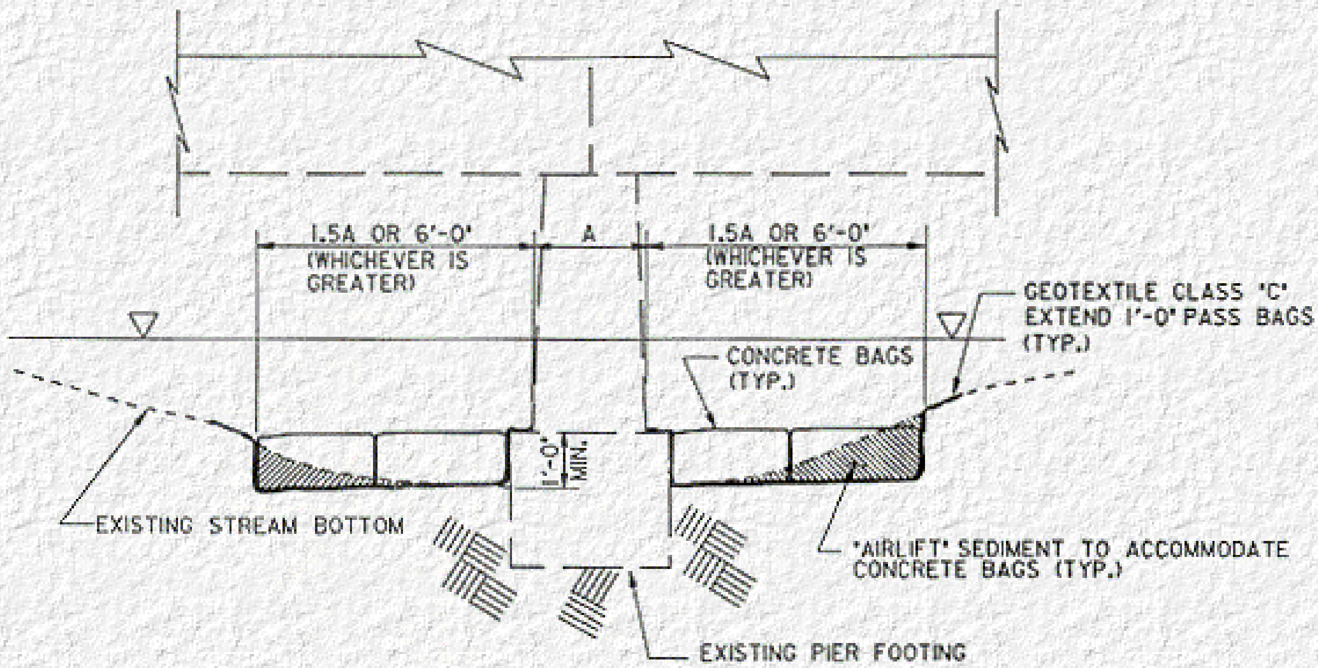


SECTION THRU PIER – NO PILES

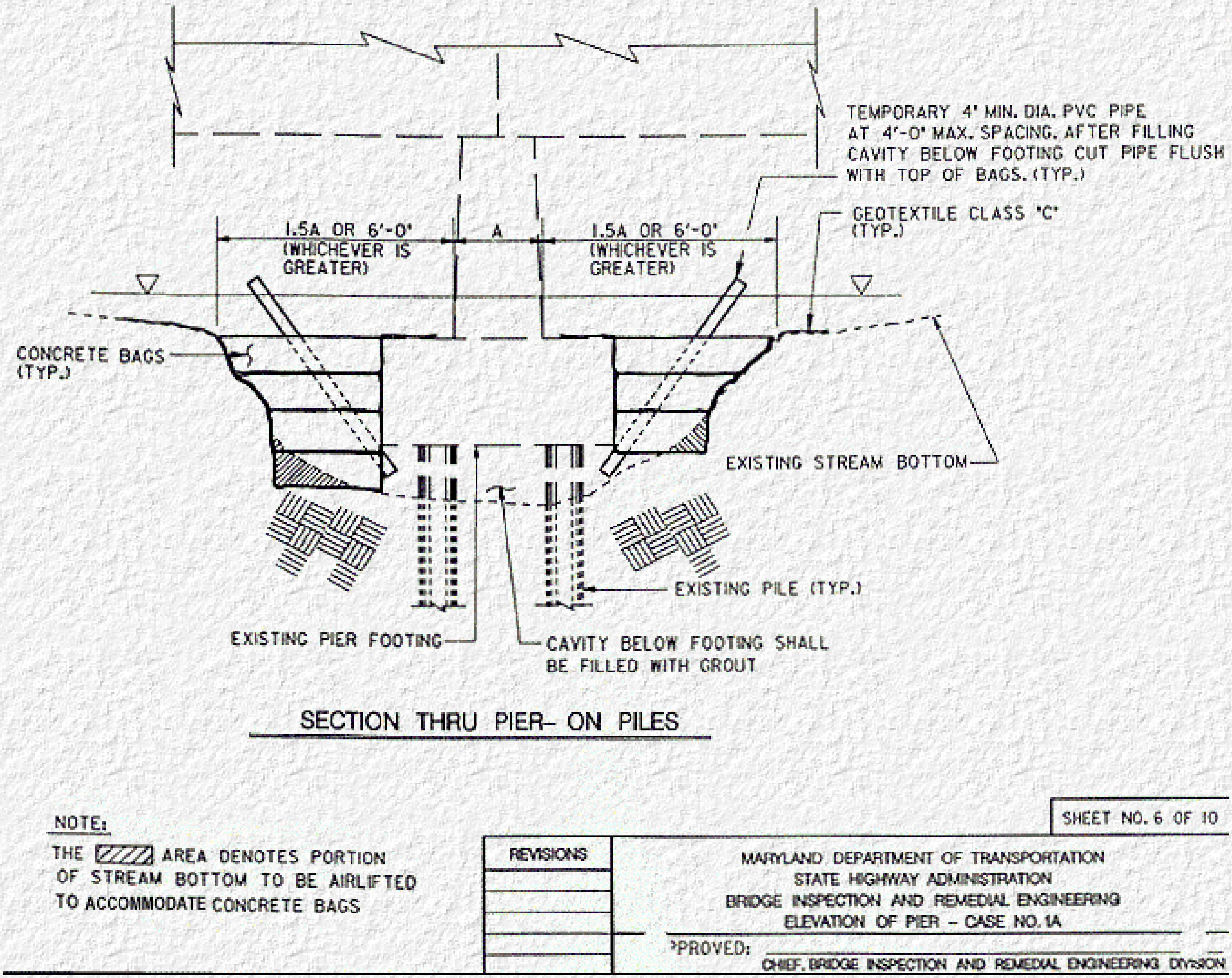
NOTE:

THE ▨ AREA DENOTES PORTION OF STREAM BOTTOM TO BE AIRLIFTED TO ACCOMMODATE CONCRETE BAGS

SHEET NO. 5 OF 10

REVISIONS	MARYLAND DEPARTMENT OF TRANSPORTATION STATE HIGHWAY ADMINISTRATION BRIDGE INSPECTION AND REMEDIAL ENGINEERING CASE WHERE SCOUR HAS OCCURRED AT PIER
	.PROVED: ______
	CHIEF, BRIDGE INSPECTION AND REMEDIAL ENGINEERING DIVISION

Attachment 5



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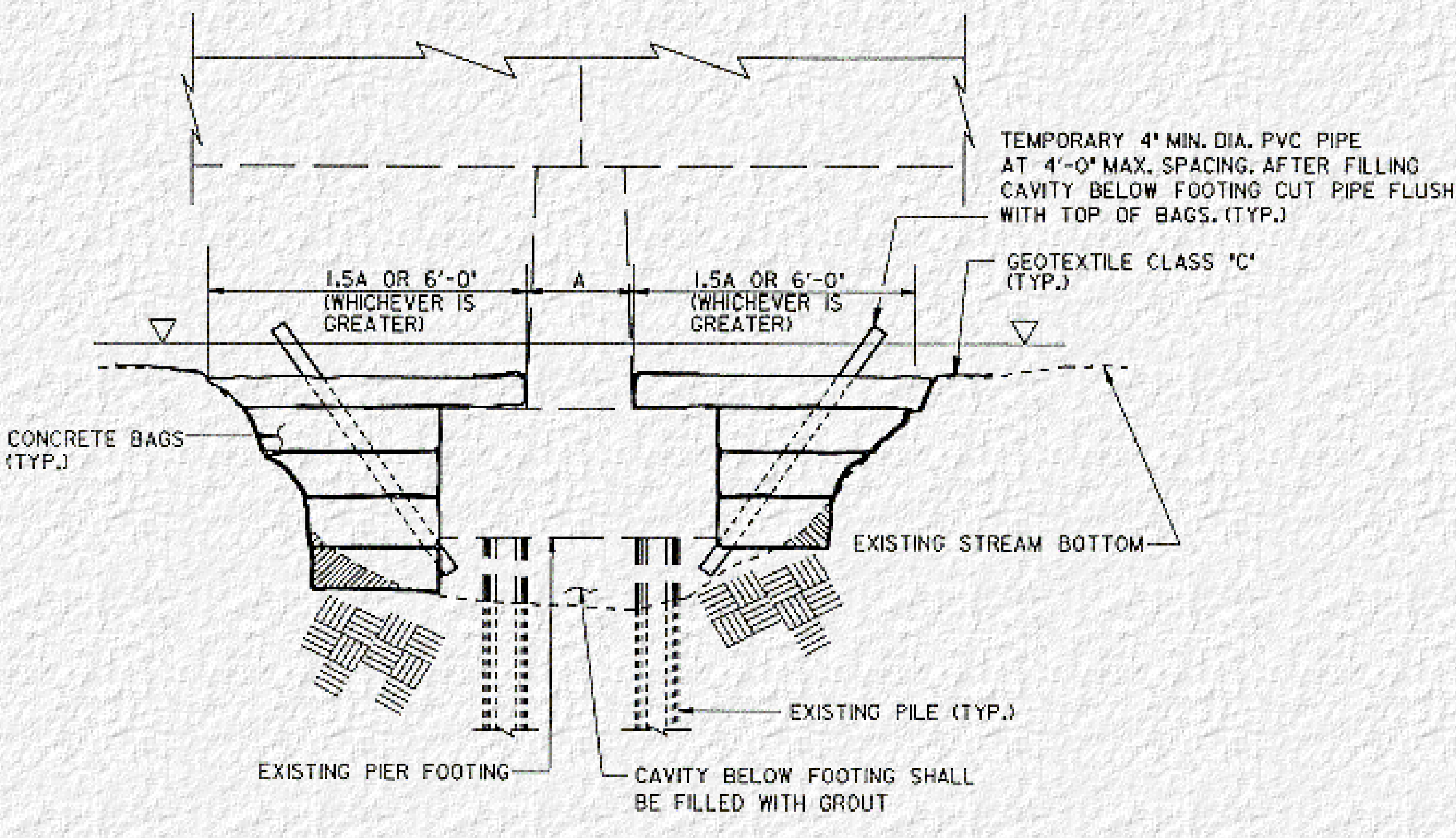


SECTION THRU PIER– ON PILES

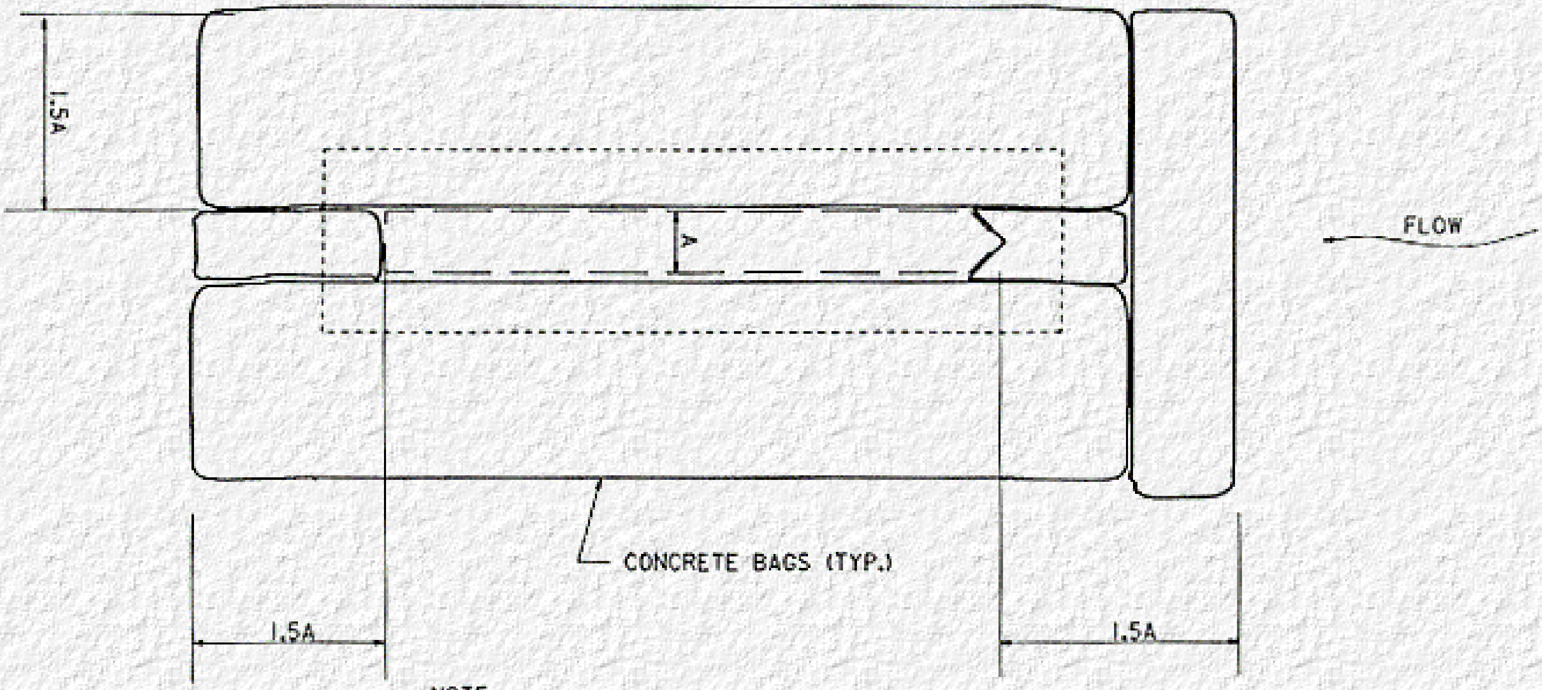
NOTE:

THE ▨ AREA DENOTES PORTION OF STREAM BOTTOM TO BE AIRLIFTED TO ACCOMMODATE CONCRETE BAGS

SHEET NO. 7 OF 10

REVISIONS	MARYLAND DEPARTMENT OF TRANSPORTATION STATE HIGHWAY ADMINISTRATION BRIDGE INSPECTION AND REMEDIAL ENGINEERING ELEVATION OF PIER – CASE NO. 1B
	?PROVED: ______ CHIEF, BRIDGE INSPECTION AND REMEDIAL ENGINEERING DIVISION

Attachment 7



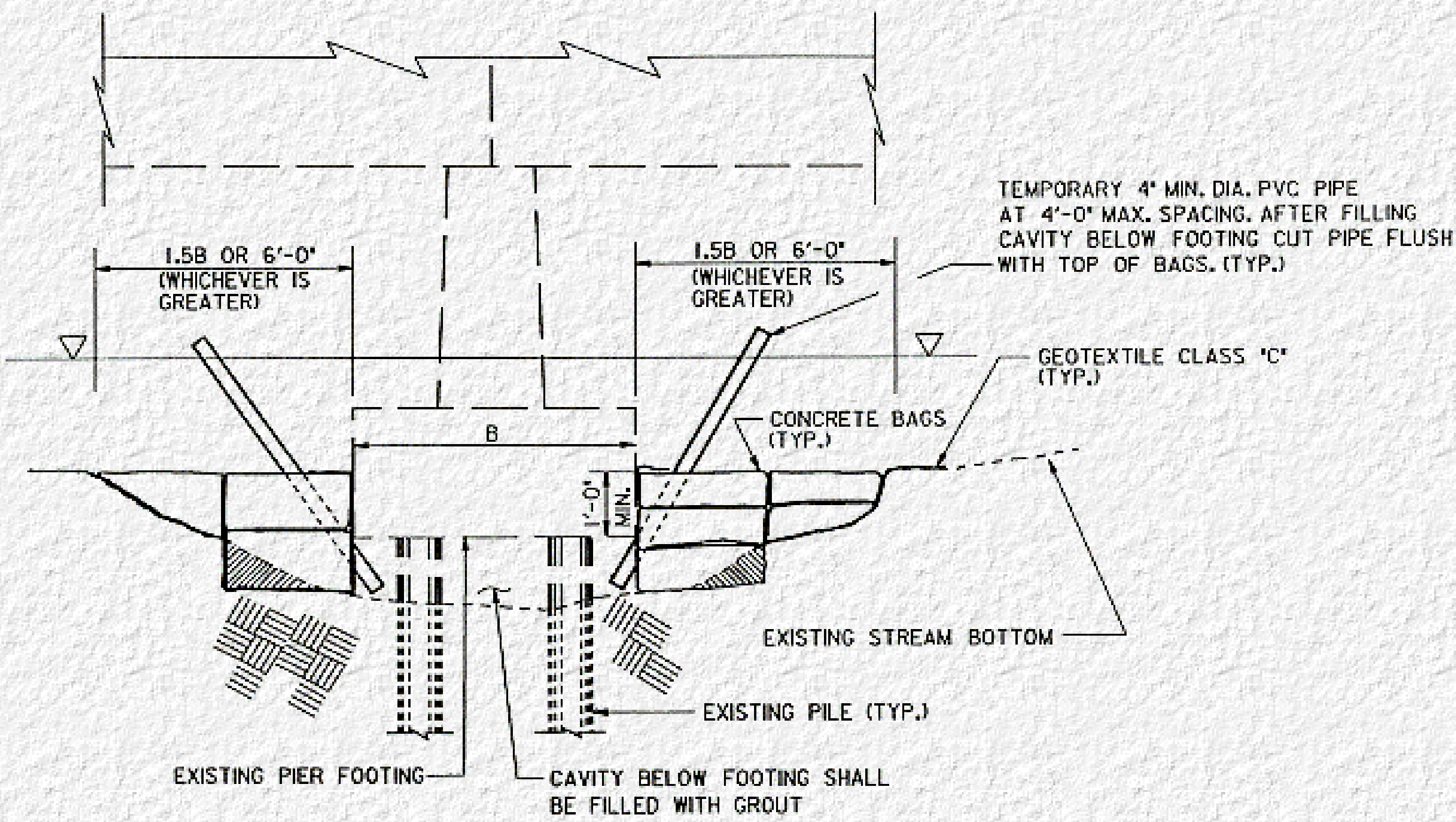
NOTE:
PLACE GROUT BAGS ON TOP OF PIER FOOTING
OR AT LEAST FLUSH WITH TOP OF FOOTING.

PLAN OF PIER – CASE NO. 1B
SCALE: 3/8" = 1'-0"

SHEET NO. 8 OF 10

REVISIONS	MARYLAND DEPARTMENT OF TRANSPORTATION STATE HIGHWAY ADMINISTRATION BRIDGE INSPECTION AND REMEDIAL ENGINEERING PLAN OF PIER – CASE NO. 1B
	APPROVED: ______ CHIEF, BRIDGE INSPECTION AND REMEDIAL ENGINEERING DIVISION

Attachment 8



SECTION THRU PIER– ON PILES

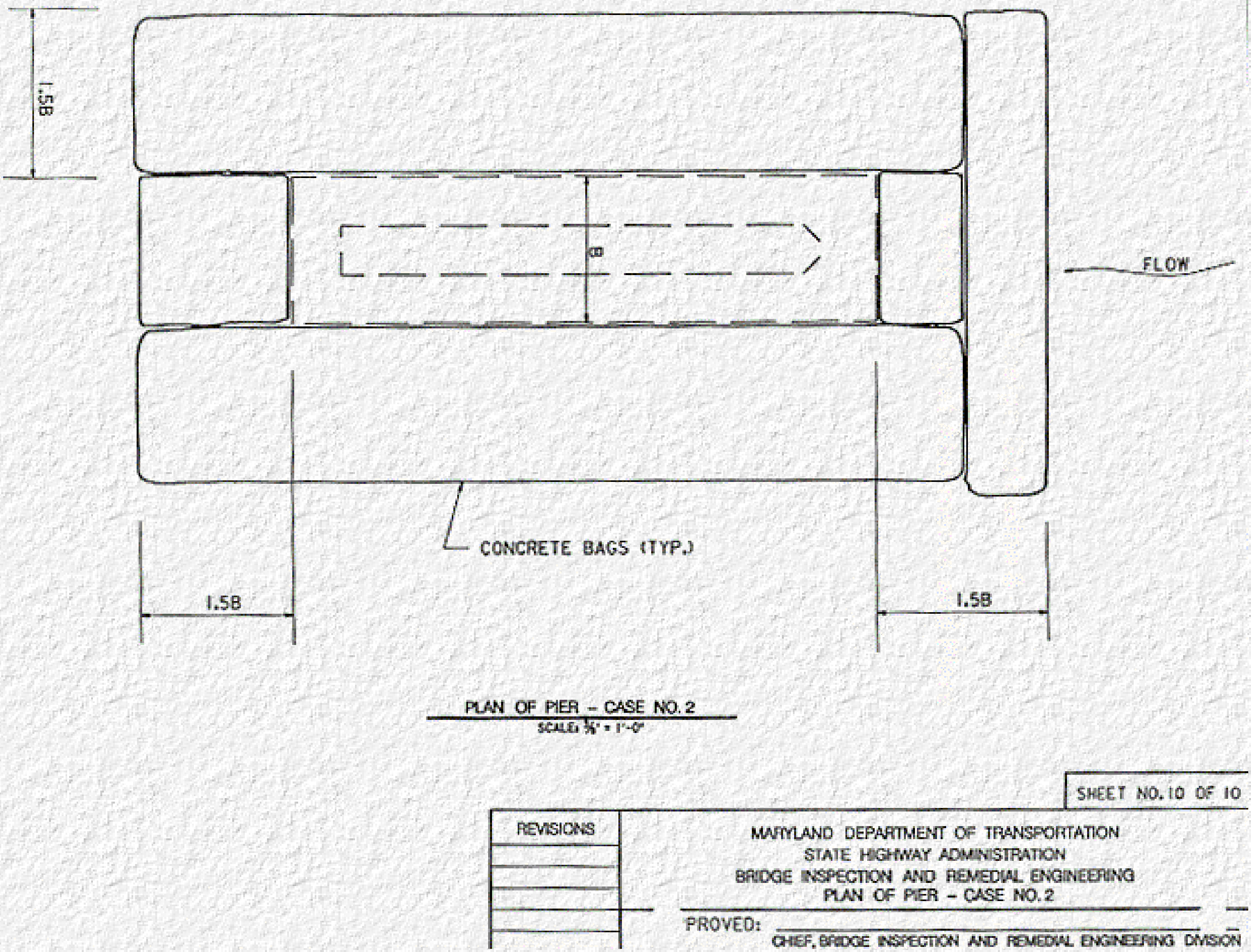
NOTE:

THE [hatched symbol] AREA DENOTES PORTION OF STREAM BOTTOM TO BE AIRLIFTED TO ACCOMMODATE CONCRETE BAGS

SHEET NO. 9 OF 10

REVISIONS	MARYLAND DEPARTMENT OF TRANSPORTATION STATE HIGHWAY ADMINISTRATION BRIDGE INSPECTION AND REMEDIAL ENGINEERING ELEVATION OF PIER – CASE NO. 2
	PPROVED: ______ CHIEF, BRIDGE INSPECTION AND REMEDIAL ENGINEERING D[illegible]ION

Attachment 9



Attachment 10

Go to Design Guideline 8

Design Guideline 8 : HEC 23

Rock Riprap at Abutments and Piers

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Introduction

The FHWA continues to evaluate how best to design rock riprap at bridge piers and abutments. Present knowledge is based on research conducted under laboratory conditions with little field verification, particularly for piers. Flow turbulence and velocities around a pier are of sufficient magnitude that large rocks move over time. Bridges have been lost (Schoharie Creek bridge for example) due to the removal of riprap at piers resulting from turbulence and high velocity flow. Usually this does not happen during one storm, but is the result of the cumulative effect of a sequence of high flows. **Therefore, if rock riprap is placed as scour protection around a pier, the bridge should be monitored and inspected during and after each high flow event to ensure that the riprap is stable**.

Sizing Rock Riprap at Abutments

The FHWA conducted two research studies in a hydraulic flume to determine equations for sizing rock riprap for protecting abutments from scour (Pagán-Ortiz 1991 and Atayee 1993). The first study investigated vertical wall and spill-through abutments which encroached 28 and 56 percent on the floodplain, respectively. The second study investigated spill-through abutments which encroached on a floodplain with an adjacent main channel (see **Figure 8.1**). 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 (see **Figure 8.2**). 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 reported in Highways in the River Environment (HIRE) indicate that with large overbank flow or large drawdown through a bridge opening that scour holes develop on the side slopes of spill-through abutments and the scour can be at the upstream corner of the abutment (Richardson et al. 1990). 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 which erode the approach embankment and the downstream corner of the abutment.

For Froude Numbers $V/(gy)^{1/2} \leq 0.80$, the recommended design equation for sizing rock riprap for spill-through and vertical wall abutments is in the form of the Isbash relationship:

$$\frac{D_{50}}{y} = \frac{K}{(S_s - 1)} \left[\frac{V^2}{gy}\right] \tag{8.1}$$

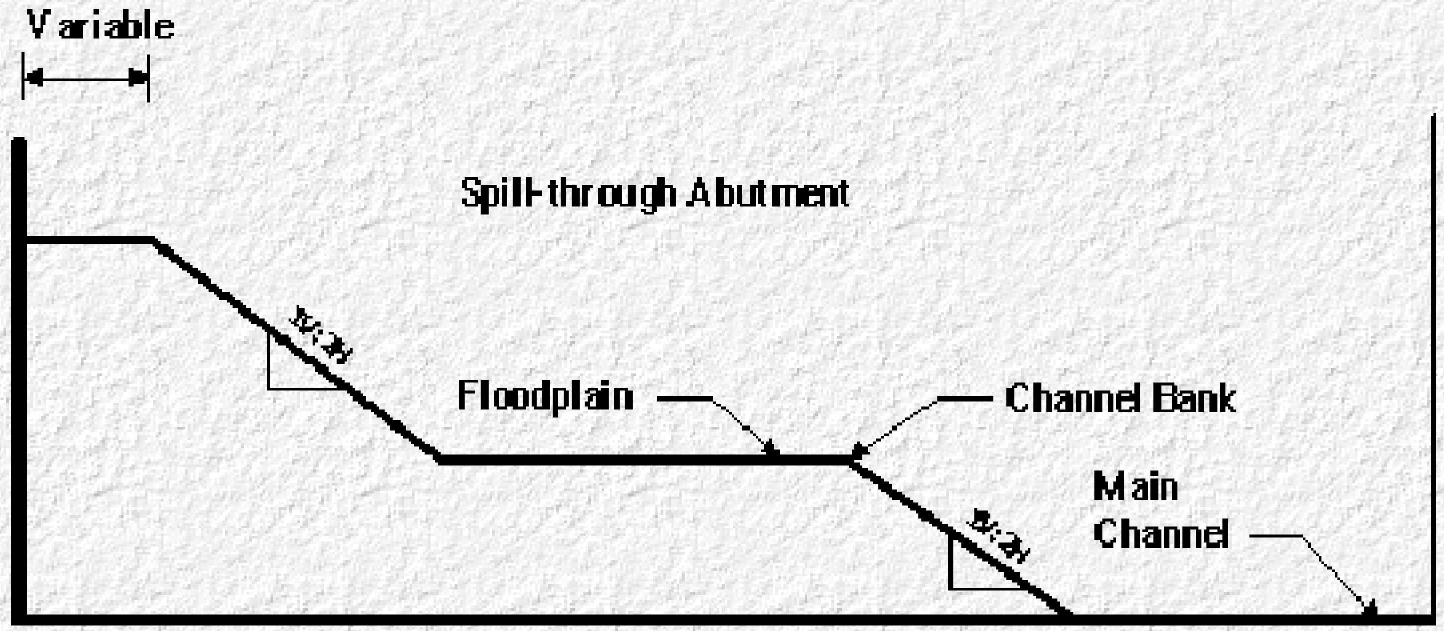

Figure 8.1 Section View of a Typical Setup of Spill-Through Abutment on a Floodplain with Adjacent Main Channel

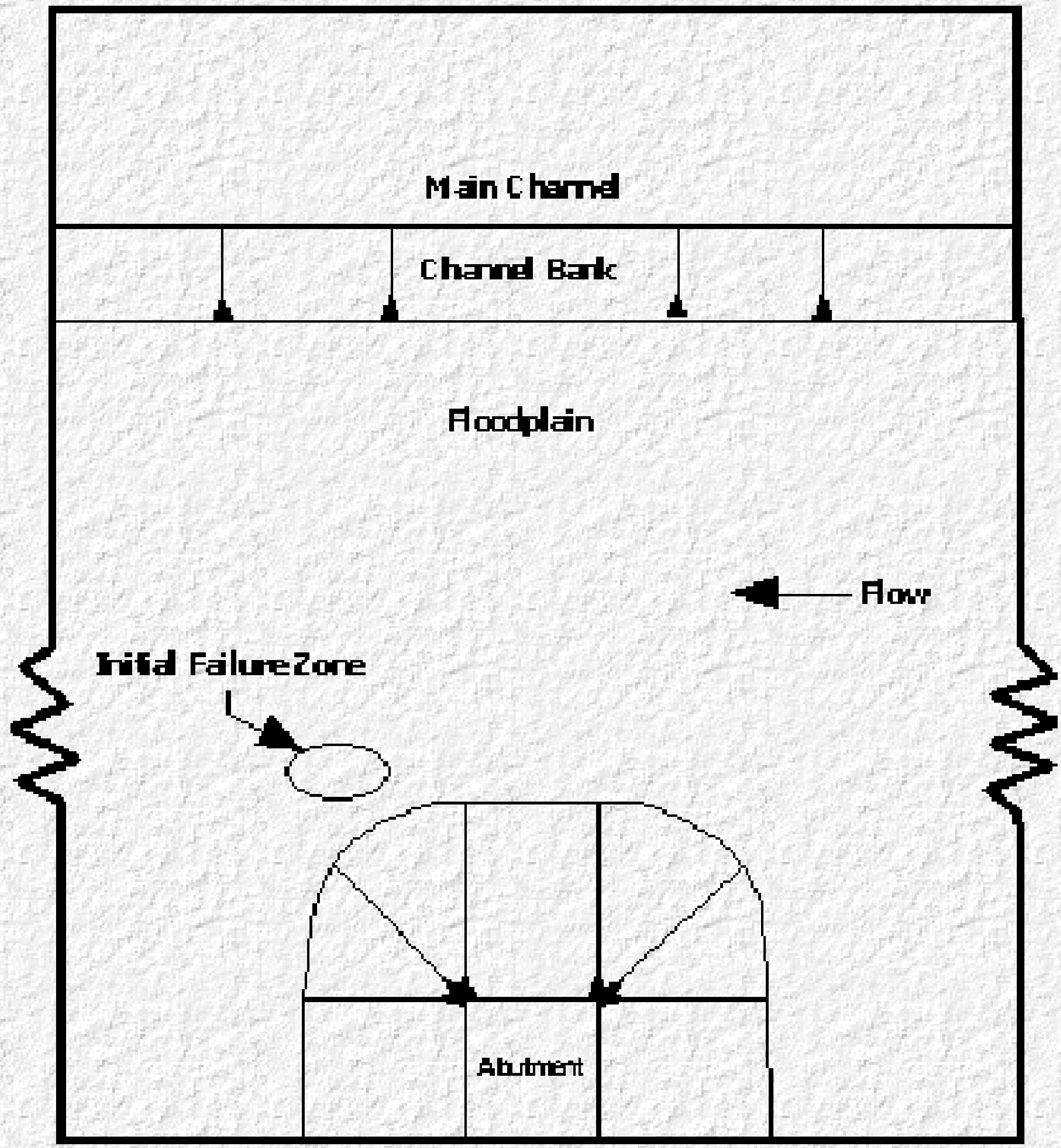

Figure 8.2 Plan View of the Location of Initial Failure Zone of Rock Riprap for Spill-Through Abutment

where:

D_{50} = Median stone diameter, m

V = Characteristic average velocity in the contracted section (explained below), m/s

S_s = Specific gravity of rock riprap

g = Gravitational acceleration, 9.81 m/s^2

y = Depth of flow in the contracted bridge opening, m

K = 0.89 for a spill-through abutment
1.02 for a vertical wall abutment

For Froude Numbers >0.80, equation 8.2 is recommended (Kilgore 1993):

$$\frac{D_{50}}{y} = \frac{K}{(S_s - 1)}\left[\frac{V^2}{gy}\right]^{0.14} \tag{8.2}$$

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 forced to overpredict 90 percent of the laboratory data.

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

1. 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 = Set-back length/average channel flow depth

   a. If SBR is less than 5 for both abutments, 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. The WSPRO average velocity through the bridge opening is also appropriate for this step.

   b. If SBR is greater than 5 for an abutment, 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. This velocity can be approximated by a hand calculation using the cumulative flow areas in the overbank section from WSPRO, or from a special WSPRO run using an imaginary wall along the bank line.

   c. If SBR for an abutment is less than 5 and SBR for the other abutment at the same site is more than 5, a characteristic average velocity determined from Step 1a for the abutment with SBR less than 5 may be unrealistically low. This 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.
2. Compute rock riprap size from equations 8.1 or 8.2, based on the Froude Number limitation for these equations.

3. Determine extent of rock riprap.

a. The apron at the toe of the abutment slope 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.

b. 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 7.5 m (see **Figure 8.3**) (Atayee et al. 1993).

c. Spill-through abutment slopes should be protected with rock riprap size computed from equations 8.1 or 8.2 to an elevation 0.15 m above expected high water elevation for the design flood. Upstream and downstream coverage should agree with step 3a except that the downstream riprap should extend back from the abutment 2 flow depths or 7.5 m whichever is larger to protect the approach embankment. Several States in the southeast use a guide bank 15 m long at the downstream end of the abutment to protect the downstream side of the abutment.

d. The 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 percent when it is placed under water to provide for the uncertainties associated with this type of placement.

e. The rock riprap gradation and the potential need for underlying filter material must be considered.

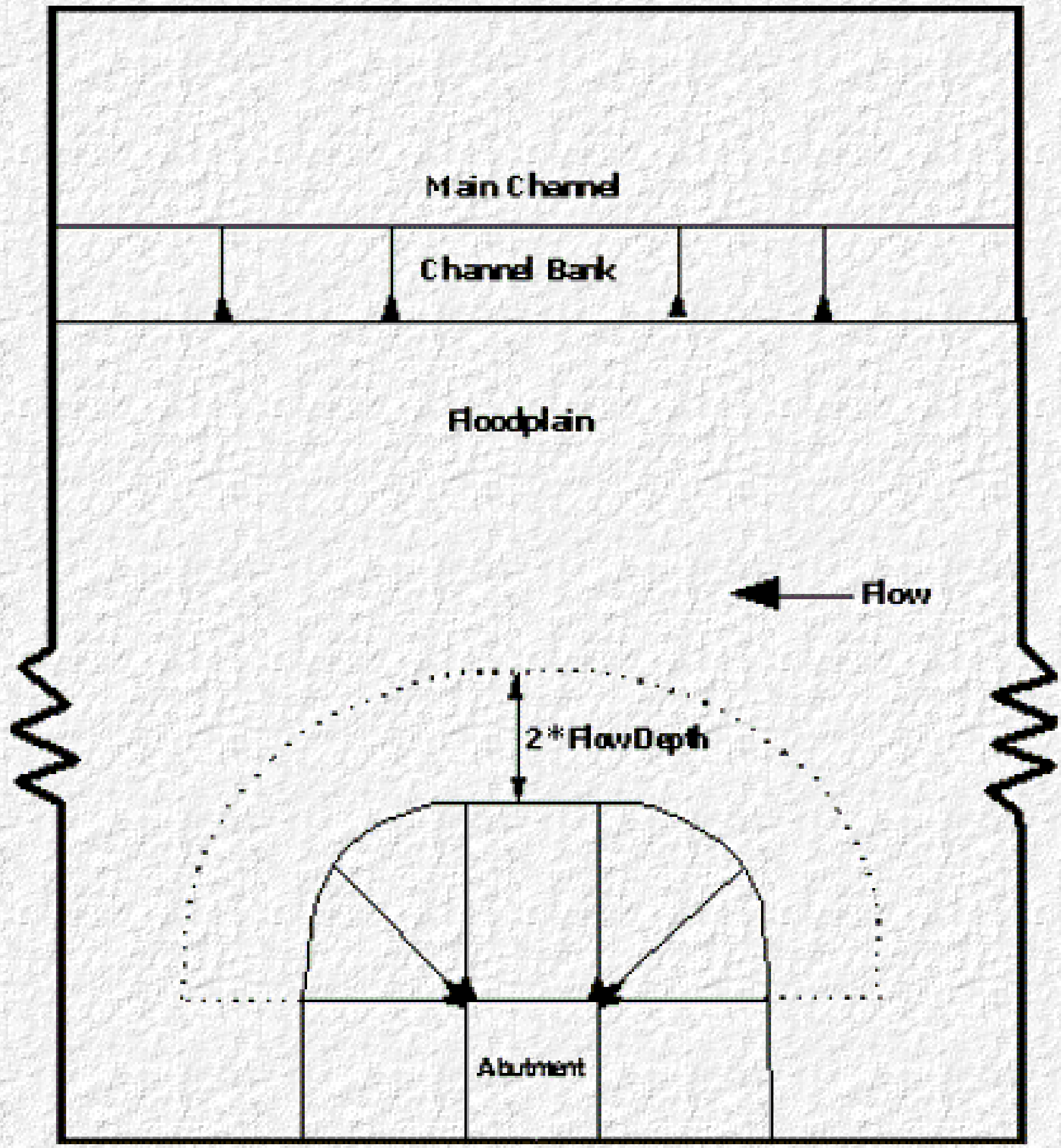

Figure 8.3 Plan View of the Extension of Rock Riprap Apron

Sizing Rock Riprap at Piers

Riprap is not a permanent countermeasure for scour at piers for existing bridges and is not to be used for new bridges. Determine the D_{50} size of the riprap using the rearranged Isbash equation (Richardson et al. 1990) to solve for stone diameter (in meters, for fresh water):

$$D_{50} = \frac{0.692(KV)^2}{(S_s - 1)2g} \qquad (8.3)$$

where:

D_{50}	=	Median stone diameter, m
K	=	Coefficient for pier shape
V	=	Velocity on pier, m/s
S_s	=	Specific gravity of riprap (normally 2.65)
g	=	9.81 m/s^2
K	=	1.5 for round-nose pier
K	=	1.7 for rectangular pier

To determine V multiply the average channel velocity (Q/A) 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 bend.

1. Provide a riprap mat width which extends horizontally at least two times the pier width, measured from the pier face.
2. Place the top of a riprap mat at the same elevation as the streambed. The deeper the riprap is placed into the streambed, the less likely it will be moved. Placing the bottom of a riprap mat on top of the streambed is discouraged. In all cases where riprap is used for scour control, the bridge must be monitored during and inspected after high flows.

   It is important to note that it is a disadvantage to bury riprap so that the top of the mat is below the streambed because inspectors have difficulty determining if some or all of the riprap has been removed. Therefore, it is recommended to place the top of a riprap mat at the same elevation as the streambed.

   a. The thickness of the riprap mat should be three stone diameters (D_{50}) or more. In general, the bottom of the riprap blanket should be placed at or below the computed contraction scour depth.

   b. In some conditions, place the riprap on a geotextile or a gravel filter. However, if a well-graded riprap is used, a filter may not be needed. In some flow conditions it may not be possible to place a filter or if the riprap is buried in the bed a filter may not be needed.

   c. The maximum size rock should be no greater than twice the D_{50} size.

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16. Abstract

This document provides guidelines for the selection and design of appropriate countermeasures to mitigate potential damage to bridges and other highway components at stream crossings. A countermeasure matrix is presented as an aid to identify most types of countermeasures which have been used by State highway agencies (SHAs) for bridge scour and stream instability problems. The matrix supports the selection of appropriate countermeasures considering such characteristics as the functional application, suitable river environment, and estimated allocation of maintenance resources. In addition, SHAs with installation experience and design guideline references are included for each type of countermeasure. Design guidelines for the following seven countermeasures are provided based on information obtained from SHAs: bendway weirs/stream barbs, soil cement, wire enclosed riprap, articulated concrete block systems, articulating grout filled mattresses, Toskanes, and grout filled bags. Design Guideline 8 presents guidance for pier and abutment riprap protection from Hydraulic Engineering Circular 18.

17. Key Words		18. Distribution Statement	
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Introduction : HEC 23

Experience, Selection and Design Guidance

Go to Design Guideline 1

1.0 Introduction

Bridge scour and stream instability problems have always threatened the safety of our nation's highways. Countermeasures for these problems are defined as measures incorporated into a highway-stream crossing system to monitor, control, inhibit, change, delay, or minimize stream instability and bridge scour problems. An action plan for monitoring structures during and/or after flood events can also be considered a countermeasure.

Countermeasures include river stabilizing works over a reach of the river up- and downstream of the crossing. Countermeasures may be installed at the time of highway construction or be retrofitted to resolve stability problems as they develop at existing crossings.

While considerable research has been dedicated to design of countermeasures for scour and stream instability, many countermeasures have evolved through a trial and error process. In addition, some countermeasures have been applied successfully in one locale, state or region, but have failed when installations were attempted under different geomorphic or hydraulic conditions. In many cases, a countermeasure that has been used with success in one state or region is virtually unknown to highway design and maintenance personnel in another state or region. Thus, there is a significant need for information transfer regarding stream instability and bridge scour countermeasure design, installation, and maintenance.

This document represents **an initial step** toward sharing countermeasure experience, selection, and design guidelines among Federal, State, and local highway agency personnel. This information may facilitate the selection and design of countermeasures as State highway agencies (SHAs) develop Plans of Action for bridges identified as scour critical.

2.0 Purpose and Sources

The purpose of this document is to identify bridge scour and stream instability countermeasures that have been implemented by various SHAs to protect bridges in the United States. The approach was to supplement information gathered from the SHAs with guidelines reported, primarily, in Federal Highway Administration (FHWA) publications, and to develop a matrix which summarizes countermeasure application and use throughout the United States. In addition, design guidelines are provided for several countermeasures which have been applied successfully on a state or regional basis, but for which only limited design references are available.

Primary information sources are:

- Response to questionnaires distributed to SHAs and others under NCHRP Project 24-7 "Effectiveness of Countermeasures to Protect Bridge Piers from Scour" in September 1995.
- Follow-up telephone conversations with selected SHA personnel who reported unique or successful countermeasures on the NCHRP Project 24-7 questionnaires.
- Review of selection, design, and case study information in several key FHWA publications including:

  - Highways in the River Environment (HIRE, 1990)
  - Evaluating Scour at Bridges (HEC-18, 1995)
  - Stream Stability at Highway Structures (HEC-20, 1995)
  - Hydraulics of Bridge Waterways (HDS-1, 1978)
  - Design of Riprap Revetment (HEC-11,1989)
  - Brice et al. (1984)
  - Brice and Blodgett (1978) Volumes 1 and 2
  - Brown et al. (1980)
  - Clopper and Chen (1988)
  - Clopper (1989)
- Review of selection and design information on countermeasures from other agencies, including:

  - State highway agencies
  - U.S. Army Corps of Engineers (COE)
  - Transportation Research Board (TRB)
  - Manufacturers' literature
- Personal experience of the authors and FHWA reviewers.

3.0 The Countermeasures Matrix

3.1 Overview

A wide variety of countermeasures have been used to control scour and stream instability at bridges. The countermeasure matrix, presented in Table 1, is organized to highlight the various groups of countermeasures and to identify their individual characteristics. The left column of the matrix lists types of countermeasures in groups. In each row of the matrix, distinctive characteristics of a particular countermeasure are identified. The matrix identifies most countermeasures used by SHAs and lists information on their functional applicability to a particular problem, their suitability to specific river environments, the general level of maintenance resources required, and which states have experience with specific countermeasures. Finally, a reference source for design guidelines is noted, where available.

Countermeasures were organized into groups based on their functionality with respect to scour and stream instability. The three main groups of countermeasures are: **hydraulic countermeasures, structural countermeasures and monitoring**. The following outline identifies the countermeasure groups in the matrix:

Group 1. Hydraulic Countermeasures

- Group 1.A: River training structures

  - Transverse structures
  - Longitudinal structures
  - Areal structures

- Group 1.B: Armoring countermeasures

  - Revetment and Bed Armor

    + Rigid
    + Flexible/articulating

  - Local armoring

Group 2. Structural Countermeasures

- Foundation strengthening
- Pier geometry modification

Group 3. Monitoring

- Fixed Instrumentation
- Portable instrumentation
- Visual Monitoring

3.2 Countermeasure Groups

3.2.1 Group 1. Hydraulic Countermeasures

Hydraulic countermeasures are those which are primarily designed either to modify the flow or resist erosive forces caused by the flow. Hydraulic countermeasures are organized into two groups: **river training structures** and **armoring countermeasures**. The performance of hydraulic countermeasures is dependent on design considerations such as filter requirements and edge treatment, which are discussed in Sections 4.4.1 and 4.4.2, respectively.

3.2.1.1 Group 1.A River Training Structures

River training structures are those which modify the flow. River training structures are distinctive in that they alter hydraulics to mitigate undesirable erosional and/or depositional conditions at a particular location or in a river reach. River training structures can be constructed of various material types and are not distinguished by their construction material, but rather, by their orientation to flow. River training structures are described as **transverse, longitudinal or areal** depending on their orientation to the stream flow.

> **Transverse river training structures** are countermeasures which project into the flow field at an angle or perpendicular to the direction of flow.
>
> **Longitudinal river training structures** are countermeasures which are oriented parallel to the flow field or along a bankline.
>
> **Areal river training structures** are countermeasures which cannot be described as transverse or longitudinal when acting as a system. This group also includes countermeasure "treatments" which have areal characteristics such as channelization, flow relief, and sediment detention.

3.2.1.2 Group 1.B Armoring Countermeasures

Armoring countermeasures are distinctive because they resist the erosive forces caused by a hydraulic condition. Armoring countermeasures do not necessarily alter the hydraulics of a reach, but act as a resistant layer to hydraulic shear stresses providing protection to the more erodible materials underneath. Armoring countermeasures generally do not vary by function, but vary more in material type. Armoring countermeasures are classified by two functional groups: **revetments and bed armoring or local armoring.**

> **Revetments and bed armoring** are used to protect the channel bank and/or bed from erosive/hydraulic forces. They are usually applied in a blanket type fashion for areal coverage. Revetments and bed armoring can be classified as either **rigid** or **flexible/articulating**. **Rigid** revetments and bed armoring are typically impermeable and do not have the ability to conform to changes in the supporting surface. These countermeasures often fail due to undermining. **Flexible/articulating** revetments and bed armoring can conform to changes in the supporting surface and adjust to settlement. These countermeasures often fail by removal and displacement of the armor material.
>
> **Local scour armoring** is used specifically to protect individual substructure elements of a bridge from local scour. Generally, the same material used for revetments and bed armoring is used for local armoring, but these countermeasures are designed and placed to resist local vortices created by obstructions to the flow.

3.2.2 Group 2. Structural Countermeasures

Structural countermeasures involve modification of the bridge structure (foundation) to prevent failure from scour. Typically, the substructure is modified to increase bridge stability after scour has occurred or when a bridge is assessed as scour critical. These modifications are classified as either **foundation strengthening** or **pier geometry modifications**.

> **Foundation strengthening** includes additions to the original structure which will reinforce and/or extend the foundations of the bridge. These countermeasures are designed to prevent failure when the channel bed is lowered to an expected scour elevation, or to restore structural integrity after scour has occurred. Design and construction of bridges with continuous spans provide redundancy against catastrophic failure due to substructure displacement as a result of scour. Retrofitting a simple span bridge with continuous spans could also serve as a countermeasure after scour has occurred or when a bridge is assessed as scour critical.
>
> **Pier geometry modifications** are used to either reduce local scour at bridge piers or to transfer scour to another location. These modifications are used primarily to minimize local scour.

3.2.3 Group 3. Monitoring

Monitoring describes activities used to facilitate early identification of potential scour problems. Monitoring could also serve as a continuous survey of the scour progress around the bridge foundations. Monitoring allows for action to be taken before the safety of the public is threatened by the potential failure of a bridge. Monitoring can be accomplished with instrumentation or visual inspection. Two types of instrumentation are used to monitor bridge scour: **fixed instruments and portable instruments**.

> **Fixed instrumentation** describes monitoring devices which are attached to the bridge structure to detect scour at a particular location. Typically, fixed monitors are located at piers and abutments. The number and location of piers to be instrumented should be defined, as it may be impractical to place a fixed instrument at every pier and abutment on a bridge. Instruments such as sonar monitors can be used to provide a timeline of scour, whereas instruments such as magnetic sliding collars can only be used to monitor the maximum scour depth. Data from fixed instruments can be downloaded manually at the site or it can be telemetered to another location.
>
> **Portable instrumentation** describes monitoring devices that can be

manually carried and used along a bridge and transported from one bridge to another. Portable instruments are more cost effective in monitoring an entire bridge than fixed instruments; however, they do not offer a continuous watch over the structure. The allowable level of risk will affect the frequency of data collection using portable instruments.

Visual inspection describes standard monitoring practices of inspecting the bridge on a regular interval and increasing monitoring efforts during high flow events (flood watch). Typically, bridges are inspected on a biennial schedule where channel bed elevations at each pier location are taken. The channel bed elevations should be compared with historical cross sections to identify changes due to scour. Channel elevations should also be taken during and after high flow events. If measurements cannot be safely collected during a high flow event, the bridge owner should determine if the bridge is at risk and if closure is necessary. Underwater inspections of the foundations could be used as part of the visual inspection after a flood.

A well designed monitoring program can be a very cost-effective countermeasure. It should be noted that a Plan of Action for a scour-critical bridge should include:

- Timely installation of temporary scour countermeasures, such as monitoring or riprap with monitoring.
- Development of a monitoring program which includes both scour measurements and detailed bridge closure instructions.
- A schedule for the timely design and construction of permanent scour countermeasures or immediate bridge replacement depending on risk involved. Monitoring can be an effective countermeasure to enhance public safety; however, the use of monitoring **does not "fix"** the scour problem and the bridge would still be considered scour critical until such time as permanent countermeasures are installed.

3.3 Biological Countermeasures

A countermeasure group not included in the matrix is biological countermeasures such as biotechnical/bioengineering stabilization. This group was not listed because it is not as well accepted as the classical engineering approaches to bridge stability. Bioengineering is a relatively new field with respect to scour and stream instability at highway bridges. There is research being conducted in this field, but bioengineering techniques have generally not been tested specifically as a countermeasure to protect bridges in the riverine environment.

3.4 Countermeasure Characteristics

The countermeasure matrix (Table 1) was developed to identify distinctive characteristics for each type of countermeasure. Five categories of countermeasure characteristics were defined to aid in the selection and implementation of countermeasures:

- Functional Applications
- Suitable River Environment
- Maintenance
- Installation/Experience by State
- Design Guidelines Reference

These categories were used to answer the following questions:

- For what type of problem is the countermeasure applicable?
- In what type of river environment is the countermeasure best suited or, are there river environments where the countermeasure will not perform well?
- What level of resources will need to be allocated for maintenance of the countermeasure?
- What states or regions in the U.S. have experience with this countermeasure?
- Where do I obtain design guidance reference material?

3.4.1 Functional Applications

The functional applications category describes the type of scour or stream instability problem for which the countermeasure is prescribed. The five main categories of functional applications are local scour at abutments and piers, contraction scour, and vertical and lateral instability. Vertical instability implies the long-term processes of aggradation or degradation over relatively long river reaches, and lateral instability involves a long-term process of channel migration and bankline erosion problems. To associate the appropriate countermeasure type with a particular problem, filled circles, half circles and open circle are used in the matrix as described below:

l **well suited/primary use** - the countermeasure is well suited for the application; the countermeasure has a good record of success for the application; the countermeasure was implemented primarily for this application.

◗ **possible application/secondary use** - the countermeasure can be used for the application; the countermeasure has been used with limited success for the application; the countermeasure was implemented primarily for another application but also can be designed to function for this application.

In addition, this symbol can identify an application for which the countermeasure has performed successfully and was implemented primarily for that application, but there is only a limited amount of data on its performance and therefore the application cannot be rated as well suited.

¡ **unsuitable/rarely used** - the countermeasure is not well suited for the application; the countermeasure has a poor record of success for the application; the countermeasure was not intended for this application.

N/A not applicable - the countermeasure is not applicable to this functional application.

3.4.2 Suitable River Environment

This category describes the characteristics of the river environment for which a given countermeasure is best suited or under which there would be a reasonable expectation of success. Conversely, this category could indicate conditions under which experience has shown a countermeasure may not perform well. The river environment characteristics that can have a significant effect on countermeasure selection or performance are:

- River type
- Stream size (width)
- Bend radius
- Flow velocity
- Bed material
- Ice/debris load
- Bank condition
- Floodplain (width)

For each environmental characteristic, a qualitative range is established (e.g., stream size: **W**ide, **M**oderate, or **S**mall) to serve as a suitability discriminator. While most characteristics are self explanatory, both HEC-20 ("Stream Stability at Highway Structures" - Figure 1 and Figure 12) and "Highways in the River Environment" (Chapter V) provide guidance on the range and definitions of these characteristics of the river environment. In the context of this matrix, the bank condition characteristic (**V**ertical, **S**teep, or **F**lat) considers the effectiveness of a given countermeasure to **protect** a bank with that configuration, **not** the suitability for installation of the countermeasure **on** a bank with that configuration.

√ Where a block is **checked** for a given countermeasure under an environmental characteristic, the countermeasure is considered suitable or has been applied successfully for the full range of that environmental characteristic.

The checked block means that the characteristic **does not influence** the selection of the countermeasure, i.e., the countermeasure is suitable for the full range of that characteristic. For example, **guide banks** have been applied successfully in braided, meandering, and straight streams; however, **bendway weirs/stream barbs** are most suitable for installation on meandering streams.

3.4.3 Maintenance

The maintenance category identifies the estimated level of maintenance that may need to be allocated to service the countermeasure. The ratings in this category range from "Low" to "High" and are subjective. The ratings represent the relative amount of resources required for maintenance with respect to other countermeasures within the matrix shown in Table 1. A low rating indicates that the countermeasure is relatively maintenance free, a moderate rating indicates that some maintenance is required, and a high rating indicates that the countermeasure requires more maintenance than most of the countermeasures in the matrix.

3.4.4 Installation/Experience by State Highway Agencies

This category identifies SHAs for which information on the use of a particular countermeasures was available. These listings may not include all of the states which have used a particular countermeasure. Information for state use was obtained from three sources: NCHRP 24-7 Questionnaire (University of Minnesota survey); Brice and Blodgett, "Countermeasures for Hydraulic Problems at Bridges, Volumes 1 and 2," (1978); and personal correspondence with SHA staff. **It is expected that additional information on state use will be obtained as this matrix is distributed and revised.** Certain countermeasures are used by many states. These countermeasures have a listing of "Widely Used" in this category. Both successful, and unsuccessful experiences are reflected by the listing.

3.4.5 Design Guideline Reference

Reference manuals which provide guidance in countermeasure design have been developed by government agencies through research programs. The FHWA has produced a wealth of information through the federally coordinated program of highway research and development. The design guideline reference column identifies reference manuals where guidance on design of the countermeasures can be obtained. The references are symbolized by numbers in this column. **The numbers correspond to the numbers of the references listed on the second page of the matrix (see also Section 6.0 References).** Countermeasures for which design guidelines are provided within this document are referenced using **DG#**, where **#** represents a number assigned to the design guideline (see Section 5.0 Design Guidelines).

4.0 Countermeasure Design Philosophy

4.1 Investment in Countermeasures

While it is sometimes possible to predict that bank erosion will occur at or near a given location in an alluvial stream, one can frequently be in error about the exact location or magnitude of potential erosion. At some locations, unexpected lateral erosion occurs because of a large flood, a shifting thalweg, or from other actions of the stream or human activities. Where the investment in a highway crossing is not in imminent danger of being lost, it is often prudent to delay the installation of countermeasures until the magnitude and location of the problem becomes obvious.

Thus, for stream instability countermeasures, a "wait and see" attitude may constitute the most economical approach. Retrofitting can be considered sound engineering practice in many locations because the magnitude, location, and nature of potential instability problems are not always discernible at the design stage, and indeed, may take a period of several years to develop.

4.2 Countermeasure Design Approach

The bridge scour and stream instability countermeasures matrix (Table 1) helps define the set of specific countermeasures that are best suited to specific site conditions. The countermeasures matrix is intended, primarily, to assist with the selection of an appropriate countermeasure. Consideration of potential environment impacts, maintenance, construction-related activities, and legal aspects can be used to refine the selection. The final selection criteria, and perhaps the most important, are the initial and long-term costs. The countermeasure that provides the desired level of protection at the lowest total cost may be the "best" for a particular application.

The following principles should be followed in designing and constructing stream instability and bridge scour countermeasures:

- The initial and long-term cost should not exceed the benefits to be derived. Permanent countermeasures should be used for important bridges on main roads and where the results of failure would be intolerable. Expendable works may be used where traffic volumes are light, alternative routes are available, and the risk of failure is acceptable.
- Designs should be based on studies of channel trends and processes and on experience with comparable situations. The environmental effects of the countermeasures on the natural channel both up- and downstream should be considered.
- Field reconnaissance by the designer is highly desirable and should include the watershed and river system up- and downstream from the bridge.
- Evaluation of time-sequenced aerial photography is a useful tool to detect long-term trends.
- The possibility of using physical model studies as a design aid should receive consideration at an early stage.
- Countermeasures must be inspected periodically after floods to check performance and modify the design, if necessary. The first design may require modification. Continuity in treatment, as opposed to sporadic attention, is advisable. The condition of the countermeasure should be documented with photographs to enable comparison of its condition from one inspection to another.
- In most cases, the countermeasure does not "cure" the instability or scour problem, and planning (funding) for continued maintenance of the countermeasure will be required.

4.3 Environmental Considerations

The environmental permitting process can have a significant effect on the planning, design and implementation of river engineering works. Often, permitting can become a lengthy process for the implementation of bridge scour and stream instability countermeasures. To expedite this process, a memorandum dated February 11, 1997, was prepared jointly by the U.S. Army Corps of Engineers (USACE) Directorate of Civil Works and the Federal Highway Administration (FHWA). The purpose of the memorandum is to facilitate timely decisions on permit applications for work associated with measures to protect bridges determined to be at risk as the result of scouring around their foundations. The USACE and FHWA consider this agreement essential to ensure the safety of the traveling public while protecting the environment. Since installing protective armoring is usually determined to be the most feasible and economical method to protect bridge foundations, it is expected that USACE Districts may experience a significant increase in requests, from bridge owners, for permits for the installation of this type of scour countermeasure.

Recognizing the importance of protecting the foundations of our Nation's scour critical bridges with properly designed scour countermeasures and the need for environmentally sound projects, the FHWA and the USACE agree to work together with the bridge owners, in a cooperative effort, to plan ahead for managing projects that will need a USACE permit. A strong cooperative effort will aid in advanced planning to avoid and minimize environmental impacts, and in identifying locations where mitigation may be appropriate. If the bridge foundation has been determined to be scour critical as part of the bridge owner's scour evaluation program, the USACE will give priority to the bridge owner's request for authorization for the installation of scour countermeasures. Bridge owners must provide the FHWA and USACE Districts advance notice of the proposed countermeasure design and construction schedule. The notice must include an evaluation of the environmental impacts of the proposed scour countermeasure and appropriate mitigation of unavoidable impacts to aquatic resources, including fisheries and wetlands. This will allow appropriate and timely cooperation on project reviews. The USACE will make the maximum use possible of forms of expedited authorization, such as nationwide permits and regional permits, and Letters of Permission and the use of FHWA's Categorical Exclusion when the condition of the bridge foundation meets the criteria for codes O through 4 for Item 113.

4.4 Other Design Considerations

4.4.1 Filter Requirements

Granular or geosynthetic filters are essential to the performance of hydraulic countermeasures, especially armoring countermeasures. Filters prevent soil erosion beneath the armoring material, prevent migration of fine soil particles through voids in the armoring material, distribute the weight of the armor units to provide a more uniform settlement, and permit relief of hydrostatic pressure within the soils. Experience has indicated that the proper design of filters is critical to the stability of revetments. If openings in the filter material are too large, excessive piping through the filter can result in erosion of the subgrade beneath the armor. Conversely, if openings in the filter are too small, hydrostatic pressures can build up in the underlying soil and result in failure of the countermeasure. Guidelines for the selection, design, and specifications of filter material can be found in Brown and Clyde (HEC-11) (1989), and detailed information on the use of geosynthetic filters can be found in Holtz et al. (FHWA HI-95-038) (1995) (see Supplemental References). The State of California Department of Transportation also provides guidance on the use of geotextile filters with slope protection measures (see Supplemental References).

4.4.2 Edge Treatment

Undermining of the edges of armoring countermeasures is one of the primary mechanisms of failure. The edges of the armoring material (head, toe, and flanks) should be designed so that undermining will not occur. For channel bed armoring, this is accomplished by keying the edges into the subgrade to a depth which extends below the combined expected contraction scour and long-term degradation depth. For side slope protection, this is achieved by trenching the toe of the revetment below the channel bed to a depth which extends below the combined expected contraction scour and long-term degradation depth. When excavation to the contraction scour and degradation depth is impractical, a launching apron can be used to provide enough volume of rock to launch into the channel while maintaining sufficient protection of the exposed portion of the bank. Continuous systems, such articulating concrete block systems and grout filled mattresses applied on side slopes, should be designed with an apron or toe trench so that the system provides protection below the combined expected contraction scour and long-term degradation depth. Tension anchors may be used to increase stability at the edges of these continuous systems. Additional guidelines on edge treatment for armoring countermeasures can be found in Brown and Clyde (HEC-11) (1989).

5.0 Design Guidelines

5.1 Overview

Following the countermeasures matrix, design guidelines are provided for several countermeasures which have been applied successfully on a state or regional basis, but for which only limited design references are available in published handbooks, manuals, or reports. No attempt has been made to include in this document design guidelines for all the countermeasures listed in the matrix. There are, however, references in the matrix to publications that contain at least a sketch or photograph of a particular countermeasure, and in many cases contain more detailed design guidelines. FHWA currently has four publications dealing with stream instability and bridge scour countermeasures. HEC-18 ("Evaluating Scour at Bridges"), HEC-20 ("Stream Stability at Highway Structures"), "Highways in the River Environment" (HIRE), and HEC-11 "Design of Riprap Revetment" contain debited design procedures for the following countermeasures:

Impermeable and permeable spurs	- HEC-20, HIRE
Drop structures (hydraulic design only)	- HEC-20
Guide Banks	- HEC-20
Riprap stability factor design	- HIRE
Sizing rock riprap at abutments	- HEC-18
Sizing rock riprap at piers	- HEC-18
General revetment design	- HEC-11

Reference to these documents is suggested for design guidelines on these countermeasures. The HEC-18 procedures for sizing rock riprap at bridge piers and abutments are presented in this document as Design Guideline 8. For guidelines on the use of geotextile for filters for countermeasures see Holtz, *et. al.*, Supplemental References.

A number of highway agencies provided specifications, procedures, or design guidelines for bridge scour and stream instability countermeasures that have been used successfully locally, but for which only limited design guidance is available outside the agency. Several of these are presented following the matrix for the consideration of and possible adaptation to the needs of other highway agencies. These specifications, procedures, or guidelines have not been evaluated, tested, or endorsed by the authors of this document or by the Federal Highway Administration. They are presented here in the interests of information transfer countermeasures that **may** have application in another state or region.

5.2 Countermeasure Design Guidelines

The following specifications, procedures, or design guidelines are included following the countermeasures matrix. The application of the countermeasure and the contributing source(s) of information are also indicated below.

Design Guideline 1

- **Bendway Weirs / Stream Barbs**

- Source(s): Colorado Department of Transportation
  Washington State Department of Transportation
  SCS
  U.S. Army Corps of Engineers
- Application: Bankline protection and flow alignment in
  meandering channel bends

[Design Guideline 2]

- **Soil Cement**
  - Source(s): Portland Cement Association
    Pima County Arizona
    Maricopa County Arizona
  - Application: Revetment for banklines and sloping abutments

[Design Guideline 3]

- · **Wire Enclosed Riprap Mattress (Railbank or Rock Sausage)**
  - Source(s): New Mexico State Highway and Transportation Department
  - Application: Revetment for banklines, guide banks, and sloping abutments

[Design Guideline 4]

- **Articulated Concrete Block System**
  - Source(s): Hydro Review
    ASCE Hydraulic Engineering
    Federal Highway Administration
    Maine Department of Transportation
    Minnesota Department of Transportation
  - Application 1: Bankline and abutment revetment and bed armor
  - Application 2: Pier scour protection

Design Guideline 5

- **Articulating Grout Filled Mattress**
  - **Source(s):** Oregon Department of Transportation
    Arizona Department of Transportation
  - **Application:** Bankline and abutment revetment and bed armor

Design Guideline 6

- **Toskanes**
  - **Source(s):** Pennsylvania Department of Transportation
    Tested at Colorado State University
  - **Application:** Pier scour protection

Design Guideline 7

- **Grout/Cement Filled Bags**
  - **Source(s):** Maryland State Highway Administration
    Maine Department of Transportation
  - **Application:** Protection of undermined areas at pier and abutments

Design Guideline 8

- **Abutment and Pier Riprap**
  - **Source(s):** HEC-18 Scour at Bridges
  - **Application:** Abutment and Pier Scour Protection

5.3 Case Histories

Section 5.7 of HEC-20 (1995) summarizes case histories of stream instability problems at bridge sites and provides information on the success (or failure) of various countermeasures used to stabilize streams. All case histories are taken from Brice and Blodgett (1978), Brice et al. (1984), and Brown et al. (1980). Site data are from Brice and Blodgett (1978). This compilation of case histories at 224 bridge sites is recommended reference material for those responsible for selecting countermeasures for scour and stream instability. Additional case histories are given in Highways in the River Environment (HIRE) (1990).

Click here to go to Table 1

Go to Design Guideline 1

TABLE 1. BRIDGE SCOUR AND STREAM INSTABILITY COUNTERMEASURES MATRIX

Countermeasure Group	Countermeasure Characteristics															
	FUNCTIONAL APPLICATIONS					SUITABLE RIVER ENVIRONMENT								MAINTENANCE	INSTALLATION EXPERIENCE BY STATE	DESIGN GUIDELINE REFERENCE
	Local Scour		Contraction Scour	Stream Instability		River Type	Stream Size	Bend Radius	Velocity	Bed Material	Ice/Debris Load	Bank Condition	Floodplain	Estimated Allocation of Resources		
	Abutments	Piers	Floodplain and Channel	Vertical	Lateral	B=braided M=meandering S=straight	W=wide M=moderate S=small	L=long M=moderate S=short	F=fast M=moderate S=slow	C=coarse bed S=sand bed F=fine bed	H=high M=moderate L=low	V=vertical S=steep F=flat	W=wide M=moderate N=narrow/none	H=high M=moderate L=low		
Group 1. HYDRAULIC COUNTERMEASURES																
Group 1.a. RIVER TRAINING STRUCTURES																
TRANSVERSE STRUCTURES																
Impermeable spurs (jetties, groins, wing dams)	◗	◗	¡	¡	l	B, M	W, M	L, M	ü	ü	ü	ü	ü	M - L	Widely Used	1, 4, 5
Permeable spurs (fences, netting)	◗	◗	¡	¡	l	B, M	W, M	L, M	M, S	S, F	L	ü	ü	H - M	AZ, CA, IA, MS, NE, OK, SD, TX	1, 2, 4
Transverse dikes	¡	¡	¡	¡	l	B, M	W, M	ü	ü	ü	ü	ü	ü	M - L	NE	
Bendway weirs/Stream barbs	◗	◗	¡	¡	l	M	ü	M, S	ü	ü	ü	ü	ü	L	CO, ID, IL, MO, MT, OR, WA	DG1
Hardpoints	¡	¡	¡	¡	l	ü	ü	ü	ü	ü	ü	ü	ü	L	CA, ND, NE, OR, SD	1, 4, 5, 7
Drop structures (check dams, grade control)	◗	◗	◗	l	¡	ü	ü	ü	ü	ü	ü	ü	ü	M	Widely Used	4, 5
Embankment spurs	◗	¡	◗	¡	¡	ü	ü	ü	ü	ü	ü	ü	W	L	AK, OK	14
LONGITUDINAL STRUCTURES																
Longitudinal dikes (crib/rock toe/embankments)	◗	¡	¡	¡	l	ü	ü	L, M	ü	ü	M, L	ü	ü	M - L	AK, AZ, CA, OK, OR, MS	2, 4, 5
Retards	◗	¡	¡	¡	l	ü	ü	L, M	ü	S, F	L	ü	ü	H - M	Widely Used	2, 4, 5
Bulkheads	l	¡	¡	¡	l	ü	ü	ü	ü	ü	ü	V, S	ü	M	Widely Used	2, 4, 5
Guide banks	l	◗	◗	¡	◗	ü	L, M	ü	ü	ü	ü	ü	W, M	M - L	Widely Used	6, 4, 5
AREAL STRUCTURES/TREATMENTS																
Jacks/tetrahedron jetty fields	¡	¡	¡	¡	l	B, M	W, M	L	M, S	S, F	M, L	ü	W, M	M	IL, KS, MS, NM, OK, SD, TX	1, 2, 4, 5
Vanes	¡	◗	¡	¡	l	B, M	W, M	L, M	M, S	S, F	L	ü	ü	H - M	IA	7, 15
Channelization	◗	◗	¡	¡	l	B, M	ü	ü	ü	ü	ü	ü	ü	M	MS, MO, MT, TX	
Flow relief (overflow, relief bridge)	◗	◗	l	¡	¡	ü	ü	ü	ü	ü	ü	ü	W	M	AZ, MD	
Sediment detention basin	¡	¡	¡	l	¡	ü	ü	ü	ü	C, S	ü	ü	ü	H - M	Widely Used	
GROUP 1.B. ARMORING COUNTERMEASURES																
REVETMENTS AND BED ARMOR																
RIGID																
Soil cement	l	◗	◗	◗	l	ü	ü	ü	ü	S, F	ü	S, F	ü	L	AZ, CO, NM, TX	DG2, 4, 5
Concrete pavement	l	◗	l	◗	l	ü	ü	ü	ü	ü	ü	S, F	ü	M	Widely Used	2, 3
Rigid grout filled mattress/concrete fabric mat	l	◗	◗	◗	l	ü	ü	ü	ü	ü	ü	S, F	ü	M	GA, MA, MD, ME, SD, WS	17
Grouted riprap	◗	¡	¡	¡	◗	ü	ü	ü	ü	ü	ü	S, F	ü	M	AZ, CA, CT, ME, MI, TN	2, 3
FLEXIBLE/ARTICULATING																
Riprap	l	◗	◗	◗	l	ü	ü	ü	ü	ü	ü	S, F	ü	M	Widely Used	2, 3, 5
Self launching riprap (windrow)	¡	¡	¡	¡	¡	ü	ü	ü	ü	C, S	ü	ü	ü	H - M	GA, CA, IL, PA	2, 4, 5, 7
Riprap fill-trench	◗	¡	¡	¡	l	ü	ü	ü	ü	ü	ü	ü	ü	M	Widely Used	2, 3, 4, 5
Gabions/gabion mattress	l	◗	◗	◗	l	ü	ü	ü	ü	S, F	M, L	ü	ü	M	Widely Used	2, 3, 5, 7, 20
Wire enclosed riprap mattress (rail bank/sausage)	l	¡	¡	¡	l	ü	ü	ü	M, S	S, F	M, L	S, F	ü	M	AZ, CO, NM	DG3, 8
Articulated concrete blocks (interlocking/cable tied)	l	◗	◗	◗	l	ü	ü	ü	ü	ü	ü	S, F	ü	M - L	AZ, CA, FL, MI, MN, OH, ME, TX	DG4, 2, 3, 10, 11, 12, 19
Articulating grout filled mattress	l	◗	◗	◗	l	ü	ü	ü	M, S	ü	ü	S, F	ü	M - L	AZ, CA, CT, ME, MI, TN	DG5, 17
LOCAL SCOUR ARMORING																
Riprap (fill/apron)	l	◗	N/A	N/A	N/A	ü	ü	ü	ü	ü	ü	S, F	ü	H - M	Widely Used	DG8, 5, 9
Grouted riprap	◗	¡	N/A	N/A	N/A	ü	ü	ü	ü	ü	ü	S, F	ü	H - M	Widely Used	3
Concrete armor units (Toskanes, tetrapods, etc.)	◗	◗	N/A	N/A	N/A	ü	ü	ü	ü	ü	ü	S, F	ü	M - L	AZ, CA, IA, IL, OR	DG6, 21, 23
Grout filled bags/sand cement bags	l	◗	N/A	N/A	N/A	ü	ü	ü	ü	ü	M, L	S, F	ü	H - M	Widely Used	DG7, 2, 4, 5, 8, 23
Gabions	l	◗	N/A	N/A	N/A	ü	ü	ü	ü	S, F	M, L	ü	ü	M	FL, OR, TN, WA	
Articulated concrete blocks (cable tied)	¡	◗	N/A	N/A	N/A	ü	ü	ü	ü	ü	ü	S, F	ü	M - L	AZ, CA, FL, MI, MN, OH, ME, TX	DG4, 13, 23
Sheet pile/cofferdam	◗	◗	N/A	N/A	N/A	ü	ü	ü	ü	ü	ü	ü	ü	M - L	CA, CT, FL, NH, WA	2, 3, 5, 7
GROUP 2. STRUCTURAL COUNTERMEASURES																
FOUNDATION STRENGTHENING																
Crutch bents/Underpinning	¡	l	l	l	◗	ü	ü	ü	ü	ü	ü	ü	ü	L	FL, NC, OR, TX	
Cross bracing	¡	l	l	l	¡	ü	ü	ü	ü	ü	ü	ü	ü	L	NC, FL, LA	

Continuous spans	¡	l	l	l	¡	ü	ü	ü	ü	ü	ü	ü	ü	L	NC	
Pumped concrete/grout under footing	l	l	◗	◗	◗	ü	ü	ü	ü	ü	ü	ü	ü	M	GA, MD, ME, MN, NC, NY, OR	
Lower foundation	l	l	l	l	l	ü	ü	ü	ü	ü	ü	ü	ü	L	CA, OR, TX	
PIER GEOMETRY MODIFICATION																
Extended footings	N/A	l	N/A	N/A	N/A	ü	ü	ü	ü	ü	ü	ü	ü	L	AR, AZ, NY, PA, TN, TX, WA	23
Pier shape modifications	N/A	l	N/A	N/A	N/A	ü	ü	ü	ü	ü	ü	ü	ü	M	FL	
Debris deflectors	N/A	l	N/A	N/A	N/A	ü	ü	ü	ü	ü	ü	ü	ü	H - M	CA, FL, NM, OR	
Sacrifical piles/dolphins	N/A	l	N/A	N/A	N/A	ü	ü	ü	ü	ü	ü	ü	ü	H - M		22, 24
GROUP 3. MONITORING																
FIXED INSTRUMENTATION																
Sonar scour monitor	◗	l	l	l	◗	ü	ü	ü	ü	ü	L	ü	ü	M	CO, FL, IN, NY, NA, TX	15, 18
Magnetic sliding collar	l	l	l	l	◗	ü	ü	ü	ü	S, F	ü	ü	ü	M	CO, FL, IN, MI, MN, NM, NY, TX	15, 18
Sounding rods	◗	l	l	l	◗	ü	ü	ü	M, S	C	M, L	ü	ü	H	AR, IA, NY	15, 18
PORTABLE INSTRUMENTATION																
Physical probes	l	l	l	l	l	ü	ü	ü	M, S	ü	M, L	ü	ü	L	Widely Used	15, 18
Sonar probes	l	l	l	l	l	ü	ü	ü	M, S		L	ü	ü	L	Widely Used	15, 18
VISUAL MONITORING																
Periodic Inspection	l	l	l	l	l	ü	ü	ü	ü	ü	M, L	ü	ü	H	Widely Used	15, 18
Flood watch	l	l	l	l	l	ü	ü	ü	ü	ü	M, L	ü	ü	H	Widely Used	15, 18

l well suited/primary use
◗ possible application/secondary use
¡ unsuitable/rarely used
N/A not applicable

ü suitable for the full range of the characteristic
DG# design guideline attached

NOTES:

1. There is limited but successful field experience using bendway weirs/stream barbs as stream instability countermeasures
2. Performance of welded versus twisted wire and PVC coated versus uncoated wire gabions is not distinguished in the matrix.
3. There is limied field experience using concrete armor units as protection for bridge piers.

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