FHWA NJ

Handbook of Scour Countermeasures Designs

FINAL REPORT
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16. Abstract

Bridge engineers in New Jersey are presented with numerous tidal flow and non-tidal scour countermeasure designs for application to scour critical bridges. This handbook presents recommended guidelines for scour countermeasure design and planning for bridges in New Jersey. The handbook has been prepared on the basis on an extensive review of all available literature on scour countermeasure for bridge foundations. The presented guidelines are based on recommendation in HEC-23, HEC-18, NCHRP 24-07 and CIRIA Manual.

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C	=	Coefficient
D_{50}	=	Median stone diameter, ft
F_r	=	upstream Froude Number
K	=	Velocity magnification factor
	=	Coefficient for pier shape
O_{95}	=	Opening size where 95% of pores are smaller, ft
Q	=	Discharge, ft^3/s
S_s	=	Specific gravity of riprap
V	=	Flow velocity, ft/s
W	=	width of pier at base, ft
	=	width of abutment at the base, ft
WR	=	the width of a riprap layer adjacent to a footing, ft
X	=	the width of abutment footing, ft
Y	=	Design depth of Riprap, ft
d	=	Water depth,
	=	thickness of riprap
y	=	Scour depth, ft
y_0	=	Flow depth, ft
α	=	Location factor
β	=	Flood factor
ζ	=	Scaling Factor for application to the geology of soil
η	=	Scaling Factor for application to width of the bridge opening
ξ	=	Scaling Factor for application to river training measures
χ	=	Scaling Factor for application to the remaining bridge life assessed
ψ	=	Scaling Factor for application to underwater inspection
Ψ_{CR}	=	Stability Factor
θ	=	Side slope angle with respect to the horizontal plane
	=	angle between the impinging flow direction and the vertical wall
ρ	=	density of water, lb/ft^3
ρ_{cb}	=	density of block material, lb/ft^3

FOREWARD

This handbook has been prepared to provide comprehensive and detailed guidelines on bridge scour countermeasures in New Jersey. The handbook can be used by bride engineers and designers for designing countermeasures for protecting foundations of abutments and piers from scour. It can also be used for planning of a new bridge site to reduce scour risk. The handbook is based on Hydraulic Engineering Circulars (HEC-18, HEC-23), recent NCHRP research reports and CIRIA Manual on Scour used extensively in Britain. The handbook is prepared with a goal to supplement HEC-18 and HEC-23 so that a bridge engineer/consultant can address different aspects of countermeasure design in New Jersey effectively. The guidelines presented in this handbook are based on the following major resources:

1. Hydraulic Engineering Circular – 18 (HEC-18)
2. Hydraulic Engineering Circular – 23 (HEC-23)
3. Countermeasures to Protect Bridge Piers from Scour (NCHRP 24-07)
4. Manual on Scour at Bridges and Other Hydraulic Structures (2000).
5. New Jersey Department of Transportation. Bridges and Structures Design Manual, Fourth Edition. Trenton, New Jersey, 2002.
6. New Jersey Department of Transportation. NJDOT Soil Erosion and Sediment Control Standards. NJDOT, Trenton, New Jersey.

CHAPTER 1

INTRODUCTION TO COUNTERMEASURES

1.1 GENERAL

1. A countermeasure is defined by HEC-23 as a measure incorporated at a stream/bridge crossing system to monitor, control, inhibit, change, delay, or minimize stream and bridge stability problems and scour.

2. Scour is the result of erosive action of running water, excavating and carrying away material from bed and banks of stream. Over thirty six thousand bridges in USA are either scour critical or scour susceptible. According to AASHTO LRFD Specifications (Section C3.7.5) "Scour is the most common reason for the failure of highway bridges in the United States".

3. For scour resistant design of existing and proposed bridges, the currently used AASHTO Codes, NJDOT Bridge Structures Design Manual, NJDOT Soil Erosion and Sediment Control Standards, FHWA Hydraulic Engineering Circulars Nos. 11, 18, 20 and 23 have outlined the scope of work and procedures that must be followed by bridge engineers.

4. NJDOT Structural Evaluation unit has identified scour critical bridges located in New Jersey. Results from hydraulic analysis, scour analysis and countermeasures design of over 200 such bridges was used as reference for this handbook.

5. Scour Critical Rivers in New Jersey: Figure 1.1 shows the locations of rivers in New Jersey. An in-depth scour review of over two hundred bridge located on these rivers was carried out by NJDOT.

1.2 OBJECTIVES OF THE HANDBOOK

The contents of the handbook are based on the review of a vast amount of literature on the subject of bridge scour and countermeasures. The objectives of the Handbook may be summarized as follows:

1. Detailing guidelines for design of countermeasures appropriate for New Jersey based on current research and knowledge.

2. Understanding erosion science and failure types

3. Identifying variables for a parametric study of erosion and scour depths

4. Applying the correct methodology and understanding limitations of scour theory

5. Identifying and using the correct software.

6. Making it easy for selecting appropriate countermeasures.

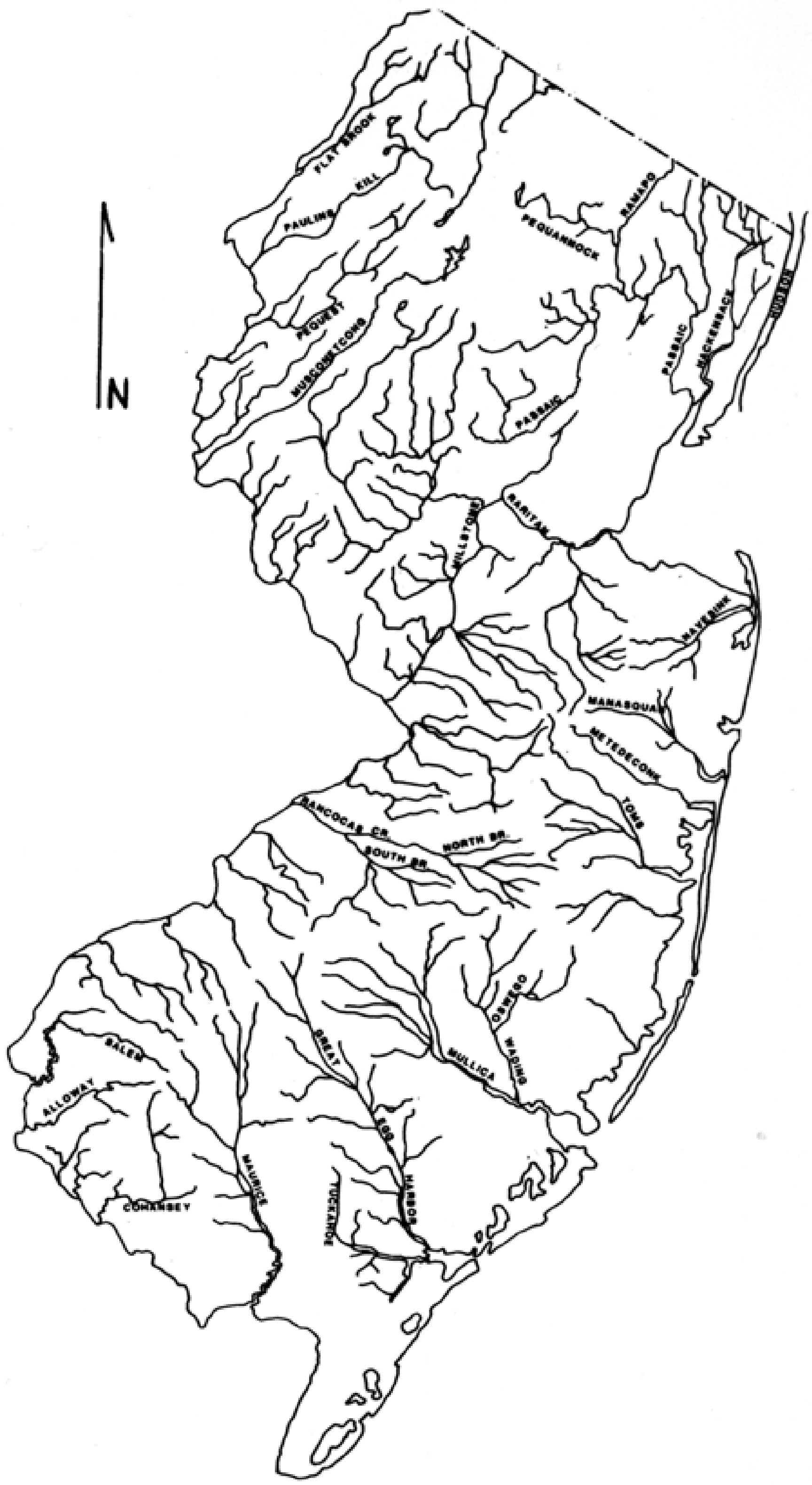

Figure 1.1. Drainage Map of New Jersey Containing Bridges on Scour Critical Rivers
N.T.S.

1.3 SCOPE OF HANDBOOK

1. The handbook is restricted to scour of bridges and culverts resulting from river flow.

2. Tidal flow is not covered by this handbook.

3. Scour of embankments and riverbeds is not covered by this handbook.

1.4 EXAMPLES OF FAILURES

Figure 1.2. Recent Scour Failure of a Ovilla Road Bridge in Ellis County, TX.

Ovilla Road Bridge (Figure 1.2) in Ellis County, Texas washed away due to heavy flooding at 8:57 a.m. CDT July 30, 2004. At least one person was killed and one was missing. Figure 1.3 shows scour damage to Peckman's bridge in New Jersey. Figure 1.4 shows scour Failures of Abutments and Approaches of US Route 9 Bridge Over Middle Branch of Forked River in New Jersey.

Figure 1.3. Failure of Approach at Peckman's River Bridge after Hurricane Floyd Struck.

Figure 1.3 (b). Abutment Settlement at Peckman's River Bridge.

Figure 1.4 (a). Collapsed northwest approach of the Peckman's River Bridge roadway showing utility pipe.

Figure 1.4 (b). Collapsed northwest approach roadway of the Peckman's River Bridge.

Figure 1.4 (c). Collapsed original north abutment built in 1890 founded on Spread footings on soil and undamaged widened abutment founded on piles, constructed in 1925.

Figure 1.4 (d). Settlement of the north abutment of the Peckman's River Bridge (severe damage at the section not founded on pile).

1.5 DESCRIPTION OF COUNTERMEASURES

Table 1.1 presents a description of commonly used countermeasures. Table 1.2 shows a description of other types of countermeasures used.

1.6 REVIEW OF COUNTERMEASURES LISTED IN HEC-18 AND HEC-23

A review of FHWA publications (e.g., HEC-18, HEC-23) on scour countermeasures has shown that the following issues need to be further addressed:

1. Foundations of new bridges should be designed to resist scour.

2. The importance of inspection and monitoring.

3. The lack of structural countermeasures and selection criteria.

4. The lack of repair details for concrete and masonry footings for existing bridges.

5. The lack of countermeasures design for unknown foundations through applications of NDT techniques.

6. The limitation to countermeasures selection based on environmental considerations; such as, meeting the requirements of a stream encroachment permit application.

7. The fact that HEC-23 does not list all types of available countermeasures.

8. The need for regular coordination between structural, hydraulic and geotechnical engineers.

9. The importance of report writing for the design and selection of countermeasures.

Table 1.1. Description of commonly used countermeasures [Mellville & Coleman (2000)]

Item No.	Countermeasure	DESCRIPTION
1	Monitoring	Under water inspection by divers or using remote sensors
2	Rock riprap	Dumped or broken rock
3	Extended footing	Structure to support the slope or protect it from erosion
4	Gabions /Reno mattress	Wire mesh baskets, mattress filled with loose stones
5	Guide Banks (spurs/dyke)	Structure to support the slope or protect it from erosion
6	Pavement/Channel lining	R.C./ bituminous concrete pavement to channel bed/banks
7	Bridge closure	Temporary detour of traffic until the bridge is repaired or replaced
8	Sacked concrete	Fabric bags filled with concrete and stacked to produce a protective layer
9	Check dams	Installing sills or drop structures
10	Artificial riprap	Alternatives to riprap such as tetrapods / toskanes
11	Concrete filled mat	Porous fabric bags placed on surface and filled with high strength mortar
12	Jetties	Walls to support the slope or protect bank from erosion
13	Flexible Revetment	Artificial Armoring
14	Precast concrete blocks	Concrete blocks of a cellular shape placed as revetment
15	Retard (timber & sheet piles/Trees)	Wall to support the slope or protect it from erosion
16	Concrete grouted riprap	Standard riprap with concrete grout
17	Sacrificial piles	Steel, timber or concrete piles driven upstream to reduce velocity
18	Soil cement	In-place soil stabilized with cement
19	Flow deflecting plates	Plates connected to piers to deflect flow
20	Cable-tied blocks	Concrete blocks /slabs interconnected with steel cables
21	Braced Piles	Piles braced together in transverse direction
22	Increase Span/ Relief Bridge	Increase the opening by reconstructing abutment/ Provide a new opening by adding span at approaches
23	Vanes	Obstructions placed upstream to redirect or reduce flow
24	Tetrapods	Artificial concrete blocks

Table 1.2. Description of other types of countermeasures (Mellville & Coleman)

Item No.	Countermeasure	DESCRIPTION
1	Hardpoint	In-place soil stabilized with cement
2	Bulkhead	Wall to support the slope or protect it from erosion
3	Channel improvements (channelization)	Dredging to increase channel width
4	Debris basin	Depressions formed to collect debris
5	Underpinning	Foundation strengthening by columns
6	Collar	Plates attached to pier to deflect flow
7	Vegetable planting	Trees planted to prevent bank erosion

1.7 HEC-23 MATRIX OF COUNTERMEASURES

The second edition of HEC-23 was published in 2001. It developed a countermeasures matrix, based on the latest research. (See HEC-23 for matrix). The matrix facilitates preliminary selection of feasible alternatives, prior to a more detailed investigation. The matrix lists the countermeasure type placed in rows, against countermeasure characteristics placed in columns. The HEC-23 Countermeasures are broad based and are applicable to a wide range of scour problems.

1. The matrix is based on engineering factors, environmental factors and cost.

2. Countermeasures have been organized into groups based on their functionality with respect to scour and stream instability. Types are classified into three groups:

   Group 1. Hydraulic Countermeasures

   - Group 1A: River training structures
     - Transverse structures
     - Longitudinal structures
     - Arial structures
   - Group 1B: 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. Each countermeasure must be selected on the basis of scour analysis for each specific site. Not all of the listed matrix countermeasures are applicable due to unique site conditions of New Jersey.

4. Countermeasure characteristics are classified into three groups:

   a. Functional Applications: Functional applications are the computed or observed scour conditions; such as local, contraction and stream instability conditions. All the listed types are applicable to New Jersey conditions.

   b. Suitable River Environment: The Suitable River Environment grouping lists a wide range of physical data for hydraulic and geotechnical conditions related to the river.

   c. Maintenance.

1.8 LIST OF HEC-23 DESIGN GUIDELINES FOR SELECTED COUNTERMEASURES

HEC-23 contains general design guidelines for several countermeasures listed below. The design guideline numbering in Table 1.3 indicates the HEC-23 design guideline chapter.

Table 1.3. Description of types of countermeasure in HEC-23

Description	Design Guidelines
Bend way Weirs	1
Soil Cement	2
Wire Enclosed Riprap Mattress	3
Articulated Concrete Block System	4
Grout Filled Mattresses	5
Concrete Armor Units	6
Grout/Cement Filled Bags	7
Rock Riprap at Piers and Abutments	8
Spurs	9
Guide Banks	10
Check Dams/Drop Structures	11
Revetments	12

HEC-23 Guidelines provide broad applications based on a wide range of hydraulic and scour conditions. In this handbook, however, a focus is made on the scour conditions prevalent in New Jersey bridges. Selected countermeasures are recommended based on experience and review of in-depth scour investigation reports.

1.9 COUNTERMEASURES BASED ON SCOUR TYPE

Scour countermeasures recommended in HEC-23 are based on the type of scour, such as contraction and local scour, aggradation or degradation. The design and selection of scour countermeasures depends on the type of scour problem at a specific site. Scour problems at a specific bridge site can be classified to meet the following objectives:

Make existing bridges safe against scour.

To identify common scour problems at bridge locations.

To group together a family of site conditions causing scour.

To recognize modes, common traits and physical characteristics causing scour.

To perform qualitative and quantitative appraisals of the variety of scour problems at bridge sites.

To find suitable solutions for the scour problems.

To rate the bridges for the degree and magnitude of scour.

To implement a safe bridge program, by preventing any scour failures.

The effectiveness of the countermeasure depends on the type of scour. When the magnitude of total scour is based on several types of scour acting simultaneously, the countermeasure shall be selected based on the governing or predominant component of scour using guideline in Chapter 3 of HEC-23.

The Table 1.4 was developed based on the recommendations given by Melville & Coleman (2000).

1.10 COUNTERMEASURES IN ASCENDING ORDER OF COST (HEC-23)

1. Development of bridge inspection and scour monitoring programs; closing bridges when necessary
2. Providing riprap at piers and monitoring
3. Providing riprap at abutments and monitoring
4. Constructing guide banks (spurs/dikes)

5. Constructing river training countermeasures and channel improvements
6. Strengthening the bridge foundations
7. Constructing sills or drop structures (check dams)
8. Constructing relief bridges or lengthening existing bridges

Table 1.4. Bridge scour countermeasures: categorized by scour type

Scour Type	**Counter-Measures**	**Examples**	**Purpose**
Lateral erosion	Armoring devices (revetment)	Riprap, gabions, cable-tied blocks, tetrapods, precast, concrete blocks, used tire, etc. Vegetation planting	Prevention of erosion to the channel bank in the vicinity of the bridge; stabilization of the channel alignment.
	Retardation	Timber piles, sheet piles Jack or tetrahedron fields Vegetation planting	Reduction of flow velocity near channel bank and inducement of deposition of sediment
	Groynes, Hardpoints	Groynes, spurs, dykes	Reduction of flow velocity near channel bank and inducement of deposition of sediment; stabilization of channel alignment.
Degradation	Check dams		Control of channel grade
	Channel lining	Concrete or bituminous concrete pavement	Control of channel degradation
	Bridge modification	Increase of bridge opening width	
Aggradation	Channel improvement	Dredging, clearing of channel Formation of a cut-off	Increased sediment transport to reduce sediment deposition at bridge crossing
	Controlled mining		Reducing in sediment input at bridge site
	Debris basin		Reduction in sediment input at bridge site
Local sour	Armoring devices	Riprap, gabions, cable-tied blocks, etc.	Reduced local scour
	Flow altering devices	Sacrificial piles, deflector vanes, collars	Reduced local scour at piers
	Underpinning of bridge piers		Reduced local scour at piers
	Guide banks		Improved flow alignment at bridge crossing; reduction in local scour at abutments

1.11 SELECTION OF COUNTERMEASURES ALTERNATIVE TO RIPRAP

When addressing specific channel conditions, various countermeasures can be grouped together to provide an overall framework of countermeasure applications. The framework would address two nominal categories of channel flow and bed conditions. One category pertains to “moderate flows” when the channel bed is stable and not subject to the passage of large bed forms. The second category, pertains to “severe flows” when bed-sediment transport conditions likely would be disruptive of countermeasures used in the milder flow and bed conditions.

Riprap, though a widely used scour countermeasure, has limited application in conditions marked by very high flow velocities and high intensities of bed-sediment movement. Additionally, in situations where a pier is in close proximity to an abutment, placement of a scour countermeasure may need to take the pier proximity into account. In other situations, suitably large riprap may not be readily available.

Although riprap has been widely used, the following countermeasures may be used as alternatives to riprap.

1. Armoring countermeasures
    a. Gabions and Reno mattresses
    b. Grout filled bags and mats
    c. Cable tied blocks
    d. Tetrapods, dolos and related units
    e. High density riprap
    f. Grade control structures
    g. Grouted concrete, pavement and flexible bed armor
2. Flow-altering countermeasures (with relatively few field applications)
    a. Sacrificial piles
    b. Upstream sheet piles
    c. Collars and horizontal plates
    d. Flow-deflecting vanes or plates
    e. Modified pier shape or texture

f. Slots in piers

g. Suction applied to bridge pier.

Note: Flow altering countermeasures should only be used in combination with primary countermeasures, such as riprap, to improve their effectiveness.

1.12 RISK REDUCTION AT EXISTING BRIDGES

Risk reduction measures serve as less costly alternates compared to regular repairs. Section 12.6 of HEC-18 recommends simplified countermeasures for certain types of existing bridges. These countermeasures will reduce the risk of scour from flood events.

1.13 Design Exception

1. A design exception may be obtained if it can be established that providing a countermeasure is not cost effective.

2. A minimum countermeasure or certain modifications to the recommended guidelines for full-scale countermeasures should be proposed. The modifications may take the form of deviations or a waiver from the guidelines. While meeting the safety demands of the bridge, the rationale for such modifications would include any unusual constraints, low traffic, low functional classification, low risk bridge and economic considerations. The following criteria or bridge type identification may be used to justify a Design Exception approval.

    a. Traffic volume

Importance Type	AADT
Type 1	< 4,000
Type 2	> 4,000 < 25,000
Type 3	> 25, 000

    b. Functional Classification

Importance Type	Feature Carried
Type 1	Local
Type 2	Collector
Type 3	Arterial
Type 4	Interstate

    c. Low Risk Bridge

        1.) Which has only a few years of service life, before it is scheduled for replacement.

2.) Which can be closed due to available alternate routes or available detours.

3.) With limited vertical clearance, where it is difficult to install formal countermeasures effectively.

d. Low Scour Bridge

1.) Provided with overflow relief

2.) Not subjected to backwater

3.) Located away from confluence

e. Type of Channel Configuration (in the increasing order of importance)

1.) Straight

2.) Braided or multi-channel

3.) Meandering

f. Type of Channel Bottom (in the increasing order of importance)

1.) Stable

2.) Aggrading

3.) Degrading

3. The detailed design of countermeasures is a site-specific problem. In proposing countermeasures, the following issues should be addressed.

- Possibility of widening or replacement of the bridge in the near future
- Bridge not listed as scour critical
- Low ADT
- Overtopping floods
- Small bridge span (less than 20 feet) for which hydraulic and scour analyses are not available
- Small culvert size (small diameter pipe culverts)
- Bridge skew exceeding 45 degrees

The resolution of the above practical difficulties should be based on the following factors:

- Environmental constraints
- Constructability
- Geotechnical issues
- Relocation of utilities
- Purchase of right of way
- Construction easement
- Detour and traffic staging.
- Excessive costs of cofferdams
- Temporary underpinning of substructure.

Based on the guidelines provided above, a concurrence to a Design Exception against provision of a countermeasure may be pursued.

CHAPTER 2

GENERAL REQUIREMENTS

2.1 PREREQUISITES TO SELECTION OF COUNTERMEASURES

Any selection and design of a countermeasure is based on a hydraulic analysis, a scour analysis, geology of the area and site-specific situation such as, the importance and the remaining life of the bridge. A designer must apply engineering judgment in examining the results obtained from scour, and hydrologic and hydraulic data. Hydrologic and hydraulic data should include:

1. Performance of the structure during past floods

2. Effects of regulation and control of flood discharges

3. Hydrologic characteristics and flood history of the stream and similar streams

4. Whether the bridge is structurally continuous

Factors which will affect the detailed design of a countermeasure are

1. Natural issues; such as, soil geology

2. Physical factors, such as width of bridge opening and traffic volume on bridge

3. Economic considerations such as existing condition of the bridge and the life of the proposed countermeasure compared to the remaining life of bridge

4. Available resources for monitoring frequency and underwater inspection

5. Priority of funding for repairs from flood damage and providing adequate countermeasures

6. Constructability

Three studies are generally required before the design can commence:

1. Hydrologic Analysis – Refer to Section 46 of the NJDOT Bridge and Structures Design Manual

2. Hydraulic Analysis - Refer to Section 46 of the NJDOT Bridge and Structures Design Manual

3. Scour Analysis - The general design procedure for a scour analysis outlined that is in the following steps is recommended for determining bridge type, size, and location (TS&L) of substructure units. The scour analysis should classify the types of scour into the following categories:

- Long term
- Short term
  - Contraction
  - Local
- Abutment Scour
- Pier Scour
- Wing wall Scour

Scour analysis for a bridge site should be based on procedure outlined in HEC-18 and AASHTO LRFD Bridge Design Specifications. Scour analysis for cohesive soils should be based on NCHRP 24-15.

2.2 DIFFERENCES IN SCOUR AND APPLICABLE COUNTERMEASURES AT ABUTMENT AND PIER

At a given bridge site, the hydraulic conditions and rate of erosion are vastly different at abutments and piers. Theoretical equations used for scour analysis differ for each case.

Abutments may be placed away from the river banks in which case erosion is small. Piers are located in the middle of peak flood zone, where flood velocity is the highest. The direction of flow is at right angles to the pier, which acts basically as an obstruction, with the water flowing on both of its sides. At the abutment, it is on one side and parallel to the length. Countermeasures are required all around the pier while for the abutment they are on one side only.

From a review of forty-five scour critical bridges in New Jersey, it is seen that the total scour depth is higher at pier locations, compared to scour depths at abutments or at wing walls. Based on addressing the need for a countermeasure installation, it can be assumed that a more significant countermeasure will required at a pier than at an abutment.

For reasons of economics and ease of construction, the types of countermeasures used may be the same at the abutment and pier. However, it may not always be possible to apply uniform countermeasures at a given bridge site. For example, structural countermeasures such as sheet piles may be used at abutments while armoring such as riprap may be used at the pier. Hence countermeasures may be different both qualitatively and quantitatively at each location.

Although a vast variety of countermeasures are applicable to pier conditions, there is a relatively smaller number of countermeasures that are recommended for abutments. It appears that much of the research in the past was focused on erosion at piers rather than at abutments.

1. Scour identification

    a. **Degradation and Aggradation**: Long-term effects of general scour such as degradation or aggradation may be assumed to be the same at piers and abutments.

    b. **Scour at bridges located on bends and a confluence of rivers**: Flow depths based on Maynord's or Thorne's equations (given in Appendix 1-References) will be equally applicable at abutments and piers.

    c. **Scour at bridges located on a confluence of a river**: Flow depths based on Ashmore and Parker's [Ashmore and Parker (1983)] or Klaassen and Vermeer's [Klaassen and Vermeer (1988)] equations will be equally applicable at abutment and pier.

    d. **Scour due to thalweg effects and migration of bed forms**: Since their magnitudes are small, they will be neglected.

    e. **Contraction scour**: It is computed from Laursen's equation (HEC-18), for channel contraction within the total bridge opening. In terms of magnitude, it is equal at the piers and abutments. Live bed scour depth increases with the increase in the size of bed material D_{50} in river bed, while clear water scour decreases as mean bed material size Dm increases. If the abutments are located outside the width of channel, no contraction takes place and there will be no contraction scour.

    f. **Local scour**: At the piers it is based on CSU equation (HEC-18). It is dependent upon many factors including length of pier, width of pier and the angle of attack. Abutment scour is computed from Froehlich's and Hire's equations (HEC-18). It is dependent upon many factors including length of embankment.

The flow of water is on both sides of pier, generating vortices and eddy currents, while for abutments the flow is on one side only, resulting in higher scour depth at piers than local scour at abutments.

2. Physical considerations

    a. When the direction of flow changes or for skew bridges the angle of attack would increase the pier obstruction. Guide banks may be needed to divert the flow mainly to minimize the pier scour.

    b. Either guide banks or vanes may be attached to the pier. Alternatively, sacrificial piles may be driven at the upstream of piers only.

3. Installation requirements

Designer’s responsibilities are not over after the countermeasure design has been completed. The following important issues need to be resolved for the successful installation of countermeasures.

a. Permitting

b. Right of Way

c. Relocation of Utilities

d. Construction Coordination

2.3 NOMINAL COUNTERMEASURES REQUIREMENT

If the computed projected scour is small or negligible, theoretically a design of a formal countermeasure will not be required. Such cases are:

1. When a spread footing is located or placed below the total scour depth.

2. When a spread footing is located or placed on hard rock

3. When an additional pile length equal to the projected scour depth is provided.

4. When a pile stiffness exceeds the minimum required and the exposed length of pile due to erosion can safely act as a long column.

Although a minor surface erosion of soil occurrence will not cause a danger to footings, a soil cover or protection to the concrete footing or piles is still required. The following issues should be addressed in maintaining adequate soil cover:

1. Frost resistance (minimum frost depth requirement)

2. As-built cosmetic appearance and

3. Unforeseen error in the scour analysis data or computations

4. 3 feet depth of riprap or an alternative countermeasure provision

2.4 GEOTECHNICAL CONSIDERATIONS

1. Geology of the river bed and its banks are one of the most important considerations in selecting the type of countermeasure. Different materials have different rates of erosion. Loose granular soils are rapidly eroded under water action, while cohesive or cemented soils are more scour-resistant. Sandy soils will be eroded earlier than bedrock.

2. Geologic maps, circa 1912 have been published for New Jersey. The Bedrock Geologic Map of Northern New Jersey, U.S. Geological Survey Map I-2540-A, The Bedrock Geologic Map of Central and Southern New Jersey,

U.S. Geological Survey Map I-2540-B are available from the NJ Dept. of Environmental Protection Maps and Publications Sales Office.

3. Geotechnical Conditions: Piers require deeper foundations due to the drop in profile of the river close to the thalweg. Hence, an abutment footing is generally located at a higher elevation than the pier footing. Notable examples are footing elevations for stub and integral abutment type bridges.

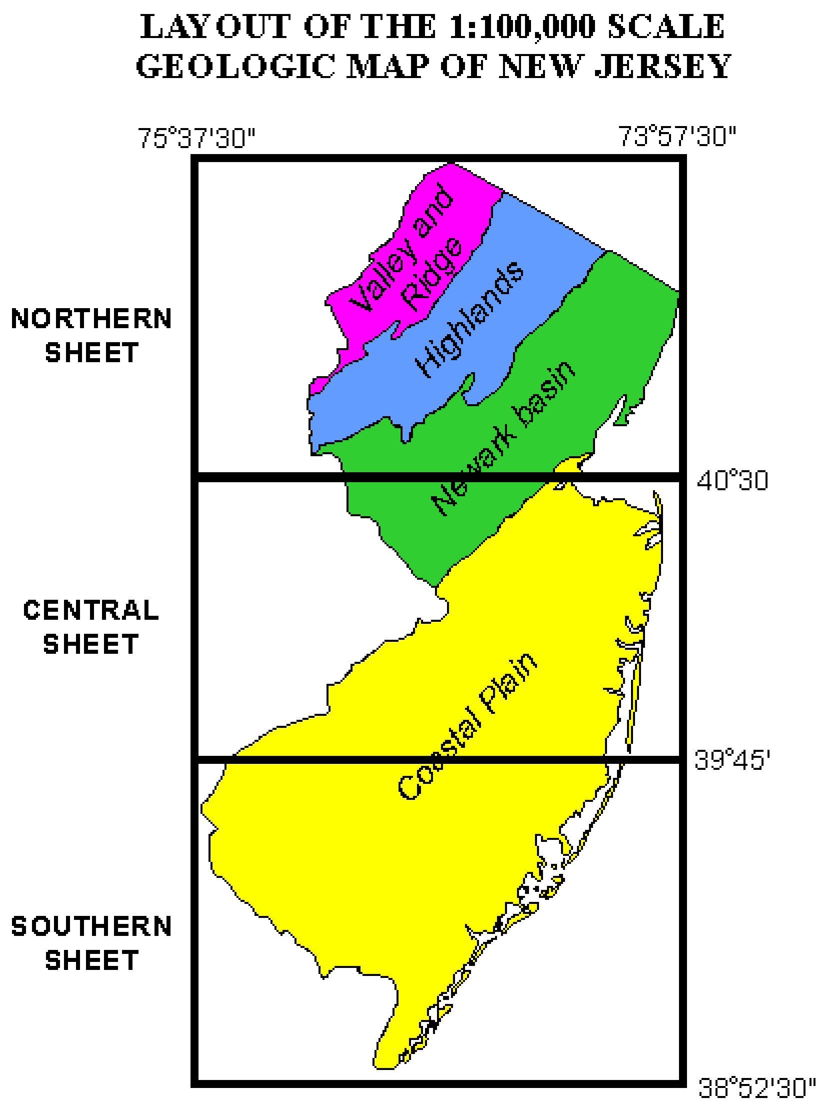

Figure 2.1. Geological Map of New Jersey Showing Latitudes

4. **Soil Types**: Soil types may be broadly classified as:

   a. Non-cohesive materials:

      o Gravels

- Sands
- Silts

Non-cohesive sediments have a granular structure, with individual particles being susceptible to erosion when the applied fluid forces (drag and lift) are greater than the stabilizing forces due to gravity and cohesion with adjacent bed particles. The threshold of movement depends on

- Particle size
- Density
- Shape
- Packing
- Orientation of bed material.

b. Cohesive materials:

Most fine-grained sediments possess some cohesion, the clay content being of great importance. Cohesive sediments typically require relatively large forces to detach the particles and initiate movement, but relatively small forces to transport the particles away. Cohesive materials are:

- Silts
- Clays

c. Sediment strata and layering:

The bed material is comprised of sediments (alluvial deposits) or other erodible material. If bed materials are stratified, a conservative approach needs to be adopted regarding the risks of the scour breaking through the more resistant layer into the less resistant layer.

The soil profile for a particular bridge site should be based on boring logs. To document sample soil profiles at different bridge sites, a detailed review of Phase II NJDOT scour inspection reports was carried out. Table 2.4 shows soil profiles and rock depths for 38 bridges reviewed.

5. **Foundation Types with Scour Problems:** As per NCHRP 24-7, the percentage of foundation types with scour problems are listed in Table 2.1. It is seen that sand foundations have 48% of scour problems while silt foundations don't have any scour problem.

Table 2.1. Foundation types with scour problems (NCHRP 24-7).

Sediment Type	Percent
Sand	48
Cohesive	19
Mixed	13
Gravel	10
Bedrock	5
Uncertain	5
Silt	0
Total	100

6. The intensity and duration of the floods will affect the rate of scour in the soil. NCHRP 24-7 has reported the following important phenomenon of the duration of scour, on different soil conditions.

Table 2.2. Duration for maximum scour depth in different soils

Sand and gravel bed materials	in hours
Cohesive bed materials	in days
Glacial tills, poorly cemented sandstones and shales	in months
Hard, dense, well cemented sandstones and shales	in years
Granites	in centuries

7. Although rocky soil is one of the most scour resistant materials, weathering rock is more erodible than granite. The table below requires laboratory tests to determine rock quality.

Table 2.3. Guidelines for Assessing the Erodibility of Bedrock

Rock property	**Scour criteria**
Rock quality designation (RQD) (ASTM D6032)	An RQD less than 50 indicates a rock that should be considered as a soil in terms of its scour potential
Unconfined compressive strength (ASTM D2938)	Samples with unconfined strengths below 1724 kPa (250 psi) are not considered to behave as rock
Slake durability index (SDI) (International Society of Rock Dynamics)	The SDI test is used on metamorphic and sedimentary rocks such as slate and shale: an SDI value of less than 90 indicates poor rock quality
Soundness (AASHTO T104)	Threshold loss rates of 12 per cent (sodium) and 18 per cent (magnesium) can be used as an indication of scour potential
Abrasion (AASHTO T96)	Rock with losses of greater than 40 per cent should be considered as erodible

8. **Soil Profiles for New Jersey Scour Critical Bridges:** A critical and comprehensive review of NJDOT Phase II inspection reports has been conducted to document soil types across different counties in New Jersey. Table 2.4 shows the soil types and available rock depth in New Jersey. This data is for informational purposes only and should not be used for the design of a scour countermeasure. Detailed geotechnical testing should be carried out to obtained site-specific information.

Table 2.4. Soil Profiles at Selected Bridge Sites in New Jersey

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Bridge No.	Bridge Name	County	Soil type	Rock depth
0119151	US 322 OVER HOSPITALITY BROOK	ATLANTIC	Brown gravel little F/M sand trace silt (GP)	>57’
0119156	US 322 OVER BIG DITCH	ATLANTIC	Gray F/M sand few silt (SM)	N/A
0206166	NJ 4 / HACKENSACK RIVER & ACCESS RD	BERGEN	Silt and muck (fine sand/silt)	N/A
0223151	ROUTE 63 OVER ROUTE 5 & WOLF CRK	BERGEN	N/A	N/A
0310154	NJ ROUTE 70 OVER FRIENDSHIP CREEK	BURLINGTON	Gray fine sand trace silt (SP)	N/A
0324162	US206 OVER ASSISCUNK CREEK	BURLINGTON	Brown F/M sand trace silt (SP)	>35’
0416152	RT 73 OVER PENNSAUKEN CREEK	CAMDEN	Gray silt little clay few fine sand (ML)	>22’
0509150	RT49 OVER MILL CREEK	CAPE MAY	Brown F/M/C sand and gravel trace silt (SP)	>24.5'
0700059	BERKELEY AVE OVER SECOND RIVER	ESSEX	Gray coarse/fine gravel with some coarse/fine sand (GP)	N/A
0700083	TULIP SPRINGS BR/W BR RAHWAY RIV	ESSEX	Brown coarse/fine gravel with some coarse/fine sand (GW)	N/A
0820155	I 295 NB OVER RACCOON CREEK	GLOUCESTER	Brown F/M sand few silt trace gravel (SP-SM)	63'
1006151	ROUTE 29 OVER SWAN CREEK	HUNTERDON	Medium to fine Gravel and coarse to fin Sand, trace Silt	N/A
1019150	RT165 OVER SWAN CREEK	HUNTERDON	Gravel and cobbles (SW)	>8"
1103151	ROUTE US 1 OVER SHIPETAUKIN CREEK	MERCER	Coarse to fine sand with varying proportions of silt and gravel (SM)	>35’
1110158	NJ 29 OVER MOORES CREEK	MERCER	Silt and clay, little fine sand	N/A
1123152	US ROUTE 130 OVER ROCKY BROOK	MERCER	Brown F/M sand trace silt (SP)	N/A
1206151	US9 OVER DEEP RUN BROOK	MIDDLESEX	Brown F/M sand trace silt trace gravel (SP)	N/A

Bridge No.	Bridge Name	County	Soil type	Rock depth
1218158	NJ RT 27 OVER S BRANCH RAHWAY RIVER	MIDDLESEX	Medium to fine gravel, some coarse to medium sand.	N/A
1303155	US RT 9 OVER MILFORD BROOK	MONMOUTH	Brown F/M sand trace silt (SP)	N/A
1304156	ROUTE 33 OVER MANALAPAN BROOK	MONMOUTH	Brown F/M/C sand some gravel little silt (SM)	>18'
1402150	NJ ROUTE 10 OVER MALAPARDIS BROOK	MORRIS	Gray-brown c-f Gravel, little (+) c-f sand, trace (-) Silt	N/A
1404158	NJ ROUTE 15 SB / ROCKAWAY RIVER	MORRIS	Coarse to fine sand and coarse to fine gravel	N/A
1417158	U S 206/S.BR. OF RARITAN RIVER	MORRIS	Boulders	3'
1419151	I-287 RAMP 'NE'OVER MALAPARDIS BK	MORRIS	Brown silt some clay little F/M sand (ML)	N/A
1502157	US 9 OVER CEDAR CREEK	OCEAN	Brown gravel and F/M/C sand trace silt (GP)	N/A
1516152	RT NJ 166 OVER NO. CHANNEL OF TOMS R.	OCEAN	Brown F/M sand trace silt (SP)	>18'
1600129	SICOMAC RD OVER MOLLY ANN'S BRK	PASSAIC	Gray coarse/medium sand with little gravel (SP)	N/A
1600460	WEST BROOK ROAD OVER WEST BROOK	PASSAIC	Gray coarse/fine sand with some gravel (SW)	N/A
1605175	RT 23 NB OVER PEQUANNOCK RIVER	PASSAIC	Gray-brown c-f SAND, some c-f Gravel, trace (-) Silt	N/A
1605175	RT 23 NB OVER PEQUANNOCK RIVER	PASSAIC	Gray-brown gravel, little sand and trace silt	N/A
1712165	I295 SB OVER OLDMANS CREEK	SALEM	Dark grey hard gravely fine to medium sand (GP)	>30.5'
1810155	RT US 206 OVER CRUSERS BROOK	SOMERSET	Medium to fine gravel and coarse to fine sand, trace silt	N/A
18H0809	CHIMNEY ROCK RD (RT525) OV MIDDLE BRK	SOMERSET	88% fine to coarse gravel, 11% fine to coarse sand and 1% silt or clay	N/A
1903153	RT 23 OVER BRANCH OF FRANKLIN LAKE	SUSSEX	Poorly-graded coarse to fine sand and gravel, trace silt	N/A
1922151	NJ.RTE.15 OVER PAULINS KILL CREEK	SUSSEX	Poorly-graded coarse to fine sand with some gravel, trace silt	N/A
2103152	RT 173 OVER POHATCONG CREEK	WARREN	Brown sandy silt	N/A
2105164	RT57 OVER POHATCONG CREEK	WARREN	Brown silty sand with gravel	N/A
2108162	RTE US 46 OVER MUSCONETCONG RIVER	WARREN	Coarse to fine sand and coarse to fine gravel trace silt	N/A

9. **Countermeasures on Soil and Rock Footings:** Figures 2.2 and 2.3 show Illustrations of Countermeasures on bridge footings located on rock and on soil (CIRIA Manual on Scour of Bridges).

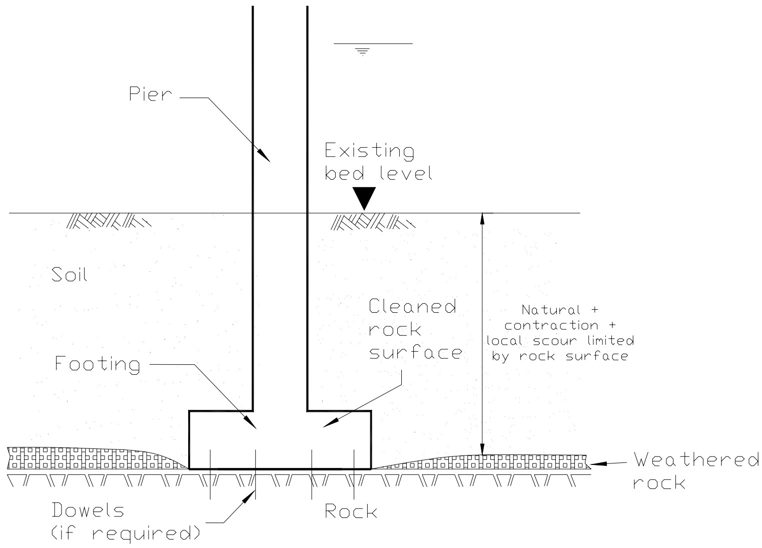


Figure 2.2. Spread footing on sound rock

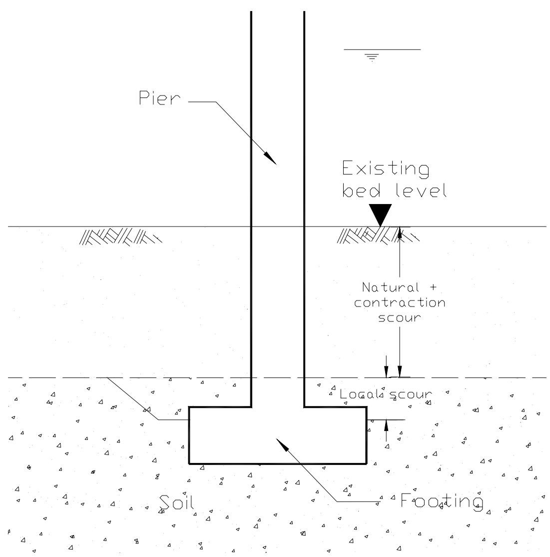


Figure 2.3. Spread footing on soil

2.5 UNKNOWN FOUNDATIONS

Unknown foundations may be defined as those existing buried foundations, which do not have as-built drawings. For scour protection the physical size and the type of foundation needs to be investigated. Table 2.5 shows different nondestructive methods to determine unknown bridge foundations.

Table 2.5. FHWA Procedures for Evaluating Unknown Foundations
(Refer Earth Engineering & Sciences, Inc, Strategies for managing unknown bridge foundations, 1994)

Method	Principle	Application	Limitations	Approximate Cost
Direct with probes, augers, core drills, etc.	Uses exploratory drilling or probes for location of subsurface elements. Cores footings to establish footing thickness	Spread foundations on land	Accessibility of foundations elements	$1500 per one span structure
Direct with adjacent test pits	Exposes subsurface elements for visual inspection and direct measurements	Spread or piled foundations on land	Accessibility. Does not determine length of piles	$1000 per one span structure
Deductive with SPT sampling	Infers foundation elevations of subsurface units from soil density correlated to blow counts	Spread or piled foundations	Accessibility. Provides approximate elevation of sub-surface units. Requires analysis of developed field data	$1500 per one span structure
Deductive with seismic refraction surveys	Infers foundation elevations of subsurface units from developed subsurface stratigraphy based on seismic velocities of soils	Spread or piled foundations. Multi span bridges	Not practical in deep water. Provides approximate elevation of bearing strata. Requires geotechnical analysis	$2000 per structure
Deductive with resistivity surveys	Infers foundation elevations of subsurface units from stratigraphy based on resistivity of soils	Spread or piled foundations on land	Not practical in saturated soils. Provides approximate elevation of bearing strata	$500 per one span structure

Method	Principle	Application	Limitations	Approximate Cost
NDT using Pulse echo	Measures propogation time of longitudinal waves in concrete	Spread or piled foundations. Particularly adapted for multi span exposed pile bent structures	Unsuitable for drilled shafts with high length diameter ratios. Requires access to foundation units and preparation of surface to introduce signal. Signal return for wood or steel piles ma be uncertain	$2000 per day plus mobilization
NDT using Transient dynamic response	Measures dynamic response in the frequency domain	Spread or piled foundations. Particularly adapted for multiple span pile bent structures or piers on piles	Same as 6.	Same as 6.
NDT using Parallel seismic or Pulse echo in borehole	Measures acoustic wave propagation from a probe placed in a borehole adjacent to the foundation unit or with impactor and receiver in borehole	Spread or piled foundations	Borehole with acoustic probe must be drilled within a few feet horizontally of foundation. Best in cohesionless soils	$2000 - $3000 per one span structure plus mobilization
NDT using Impulse radar	Measures electromagnetic pulses radiated in soil and recaptured to graphically indicate obstructions	Spread or piled foundations	Not effective in salt water or saturated soils in delineating stratigraphy. May identify location of foundation elements when angled	$2000 per one span structure plus mobilization

2.6 MODIFIED COUNTERMEASURES MATRIX FOR NEW JERSEY

HEC-23 matrix has generalized applications. Table 2.8 provides a modified version of the HEC-23 matrix that is applicable to hydraulic and scours conditions for rivers in New Jersey. The matrix is organized to highlight the various groups of countermeasures and to identify their individual characteristics. For example, transverse structures have worst effects on the river environment. Monitoring based on flood watch and periodic inspection is most effective in terms of costs. As compared to the HEC-23 matrix, the impact of each countermeasure on the river environment is taken into account in Table 2.8. Selection is also based on the investment into countermeasures vs. the remaining useful life of a bridge that is determined from its structural evaluation.

The modified matrix in Table 2.8 can be used to prioritize or carry out preliminary screening of countermeasures based on general site conditions. Once several candidate countermeasures have been selected, a more detailed design can be done based on guidelines that are presented in this handbook.

TABLE 2.6 Modified bridge scour & stream instability countermeasures matrix for New Jersey

Countermeasures Group	Countermeasures Characteristics – Functional Applications – Local Scour[1] – Abutment	Local Scour[1] – Pier	Contraction Scour – Flood-plain & Channel	Suitable River Environment – Stream Instability – Vertical / Aggradation Degradation	Stream Instability – Lateral Erosion / Meander	River Type – Braided **B** Meandering **M** Straight **S** Suitable for All Cases **B M S**	River Size – Wide **W** Moderate **M** Small **S** Suitable for All Cases **W M S**	Velocity – Moderate **M** Slow **S**	Bed Material – Coarse Sand Fine Suitable for All Cases **C S F**	Maintenance – Estimated Allocation of Resources – High Moderate Low	Environmental Considerations – Does the Countermeasure Meet Current NJDEP Requirements? (Permit Required) – Stream Encroachment	Adverse Effect On Marine Life/ Habitat	Adverse Effect On Vegetation	Bridge Status – Evaluate Structural Health of Bridge: Bridge to be Repaired or Replaced? – Increase in Foundation, Substructure Performance	Increase in Remaining Life with Increased Maintenance, Repairs and by Installing the Countermeasure
GROUP 1. HYDRAULIC COUNTERMEASURES											H High L Long L Low N No S Short	MAJ Major Difference x Not Applicable MIN Minor Difference	o Unsuitable **O** Well Suited D Secondary Use		
	GROUP 1A. RIVER TRAINING STRUCTURES														
TRANSVERSE STRUCTURES															
Impermeable Spurs (Groins, Wing Dams)	D	D	O	o	**O**	B M	W M	M S	C S F	M L	H	Y	Y	MAJ	S
Drop Structures (Check Dams, Grade Control)	D	D	D	**O**	o	B M S	W M S	M S	C S F	M	H	Y	Y	MAJ	S
LONGITUDINAL STRUCTURES															
Retards	D	o	o	o	**O**	B M S	W M S	M S	S F	H M	L	N	N	MIN	S
Plantation	o	o	o	**O**	**O**	M S	W M S	M S	S F	L	N	N	N	MIN	S
Bulkheads	**O**	o	o	o	**O**	B M S	W M S	M S	C S F	M	H	Y	Y	MAJ	S
Guide Banks	**O**	D	D	o	D	B M S	W M	M S	C S F	M L	L	N	N	MAJ	S
	GROUP 1.B ARMORING COUNTERMEASURES														
	REVETMENTS & BED ARMOR														
Rigid															
Concrete Pavement	**O**	D	**O**	D	**O**	B M S	W M S	M S	C S F	M	H	Y	Y	MIN	S
Rigid Grout Filled Mattress / Concrete Fabric Mat	**O**	D	D	D	**O**	B M S	W M S	M S	C S F	M	H	Y	Y	MIN	S
Flexible / articulating															
Riprap on Textile	**O**	D	D	D	**O**	B M S	W M S	M S	C S F	M	N	N	N	MAJ	L
Riprap Fill Trench	D	o	o	o	**O**	B M S	W M S	M S	C S F	M	N	N	N	MAJ	L
Gabion/Gabion Mattress	**O**	D	D	D	**O**	B M S	W M S	M S	S F	M	N	N	N	MAJ	L
Articulated Concrete Blocks (Interlocking / Cable Tied)	**O**	D	D	D	**O**	B M S	W M S	M S	C S F	M L	H	Y	Y	MAJ	L
	LOCAL SCOUR ARMORING														
Riprap (Fill/Apron)	**O**	D	x	x	x	B M S	W M S	M S	C S F	H M	N	N	N	MAJ	L
Concrete Armor Units (Toskanes, Tetrapods)	D	D	x	x	x	B M S	W M S	M S	C S F	M L	L	Y	Y	MIN	S
Grout Filled Bags / Sand Cement Bags	**O**	D	x	x	x	B M S	W M S	M S	C S F	H M	L	Y	Y	MIN	S
Gabions	**O**	D	x	x	x	B M S	W M S	M S	C S F	M	N	N	N	MAJ	S
Articulated Concrete Blocks	**O**	D	x	x	x	B M S	W M S	M S	S F	M L	H	Y	Y	MAJ	L
Sheet Pile	D	D	x	x	x	B M S	W M S	M S	C S F	M L	L	N	N	MAJ	L
GROUP 2 STRUCTURAL COUNTERMEASURES															
	FOUNDATION STRENGTHENING														
Crutch Bents / Underpinning	o	**O**	**O**	**O**	D	B M S	W M S	M S	C S F	L	L	N	N	MAJ	L

Countermeasures Group	Countermeasures Characteristics										Environmental Considerations			Bridge Status	
	Functional Applications			**Suitable River Environment**						**Maintenance**	**Environmental Considerations**			**Bridge Status**	
	Local Scour[1]		Contraction Scour	Stream Instability		River Type	River Size	Velocity	Bed Material	Estimated Allocation of Resources	**Does the Countermeasure Meet Current NJDEP Requirements?** (Permit Required)			**Evaluate Structural Health of Bridge: Bridge to be Repaired or Replaced?**	
Countermeasures **Group**	Abutment	Pier	Flood-plain & Channel	Vertical / Aggradation Degradation	Lateral Erosion / Meander	Braided **B** Meandering **M** Straight **S** Suitable for All Cases **B M S**	Wide **W** Moderate **M** Small **S** Suitable for All Cases **W M S**	Moderate **M** Slow **S**	Coarse Sand Fine Suitable for All Cases **C S F**	High Moderate Low	Stream Encroachment	Adverse Effect On Marine Life/ Habitat	Adverse Effect On Vegetation	Increase in Foundation, Substructure Performance	Increase in Remaining Life with Increased Maintenance, Repairs and by Installing the Countermeasure
Pumped Concrete/ Grout Under Footing	**O**	**O**	D	D	D	B M S	W M S	M S	C S F	M	L	N	N	MAJ	L
Lower Foundation/ Curtain Wall	**O**	**O**	**O**	**O**	**O**	B M S	W M S	M S	C S F	L	L	N	N	MAJ	L
PIER GEOMETRY MODIFICATION															
Extended Footings	x	**O**	x	x	x	B M S	W M S	M S	C S F	L	H	Y	Y	MIN	S
Sacrificial Piles	x	**O**	x	x	x	B M S	W M S	M S	C S F	H M	N	N	N	MAJ	L
GROUP 3 MONITORING															
	FIXED INSTRU-MENTATION														
Sonar Scour Monitor	D	**O**	**O**	**O**	D	B M S	W M S	M S	C S F	M	N	N	N	MIN	S
Magnetic Sliding Collar	**O**	**O**	**O**	**O**	D	B M S	W M S	M S	C S F	M	N	N	N	MIN	S
	PORTABLE INSTRUMENTATION														
Physical Probes	O	**O**	**O**	**O**	**O**	B M S	W M S	M S	C S F	L	N	N	N	MIN	S
Sonar Probes	**O**	**O**	**O**	**O**	**O**	B M S	W M S	M S	C S F	L	N	N	N	MIN	S
	VISUAL MONITORING														
Periodic Inspection	**O**	**O**	O	**O**	**O**	B M S	W M S	M S	C S F	H	N	N	N	MAJ	L
Flood Watch	**O**	**O**	**O**	**O**	**O**	B M S	W M S	M S	C S F	H	N	N	N	MAJ	L

SECTION 2 - SELECTION AND DESIGN OF ARMORING COUNTERMEASURES

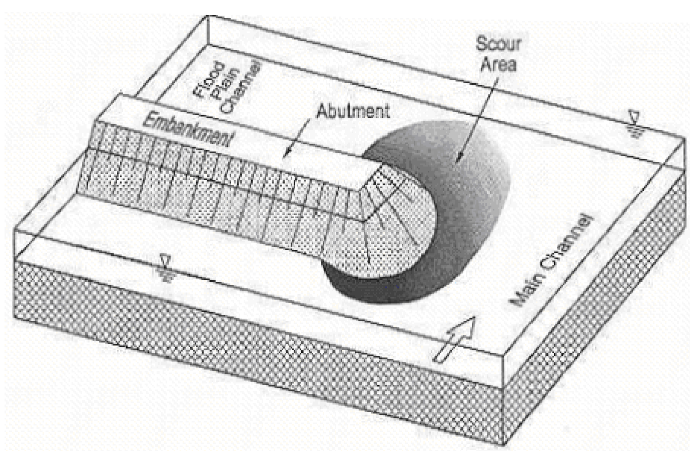
CHAPTER 3

COUNTERMEASURES APPLICABLE TO ABUTMENTS

3.1 SCOUR AT ABUTMENTS

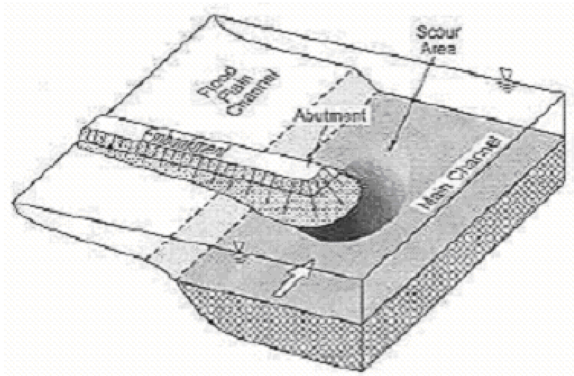
Bridge abutments and their approach embankments are the most commonly damaged bridge components during floods. Figures 3.1 to 3.5 illustrate common causes of scour at bridge abutments. The illustrations are as investigated under laboratory conditions by Ettema.

Illustrated are several scour-related processes that may occur at spill-through abutments. Each scour type and the cause of scour is shown in the figures. Figure 3.1 shows that scour may be attributed to the combined effects of local and constriction scour processes. It also applies to scour at a rectangular abutment or wing wall abutment located in a uniformly deep alluvial channel. Abutment failure occurs when scour reduces support for abutment foundations. Figure 3.2 illustrates essentially the same scour process as in Figure 3.1 except that the presence of a flood plain may alter the flow field at the abutment. In Figure 3.3, the abutment is threatened by a geotechnical failure of the main channel bank. The scour may be caused by a combination of local as well as the constriction scour process and by main channel shifting. Figure 3.4 shows scour of a flood plain at the abutment. Figure 3.5 shows the erosion of embankment approach to the abutment.



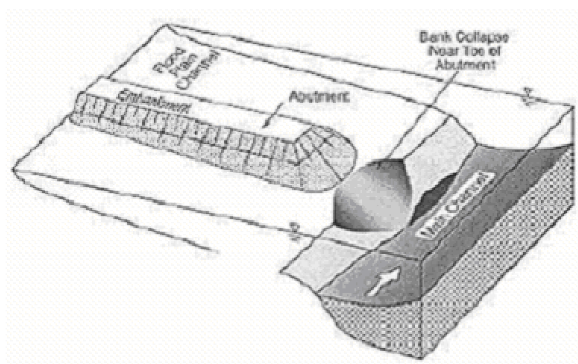
Abutment threatened by scour of main-channel bed.

Figure 3.1. Damage to bridge abutments because of scour of flood main channel under abutments when both embankment and abutment are in the main channel.



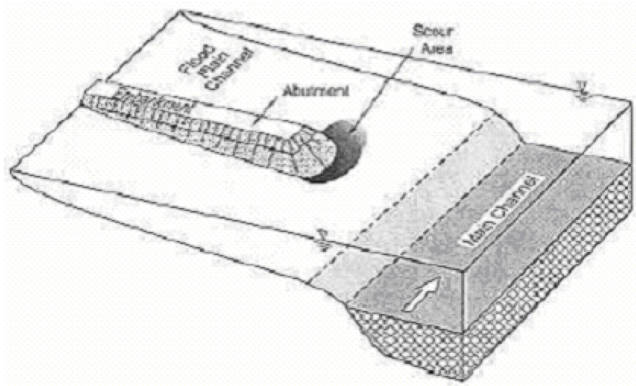
Abutment threatened by scour of main-channel bed. Includes additional effect of constriction scour

Figure 3.2. Damage to bridge abutments because of scour of flood main channel under abutments when only the abutment is in the main channel.



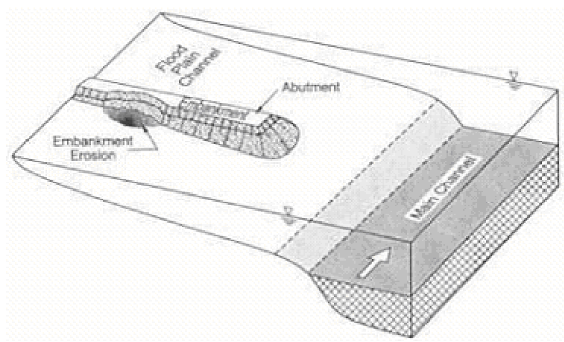
Abutment threatened by collapse of main-channel bank consequent to scour (local and constriction) of main-channel bed.

Figure 3.3. Bridge abutments threatened because of collapse near toe of abutment channel.



Abutment threatened by scour of floodplain.

Figure 3.4. Abutment threatened because of scour of floodplain.



Abutment threatened by scour of embankment.

Figure 3.5. Abutment threatened because of scour of embankment.

There are several countermeasures used for protection of bridge abutments by different state DOTs and federal agencies. Two typical approaches for protecting bridge abutments from scour are

- Mechanically stabilizing the abutment slopes with riprap, gabions, cable-tied blocks, or grout filled bags or
- Aligning the upstream flow by using guide banks, dikes or spurs, or in-channel devices such as vanes and bend way weirs.

Table 3.1 presents advantages and disadvantages of these countermeasures.

Table 3.1. Advantages and disadvantages of various countermeasures

Countermeasure	Advantages	Disadvantages
Local Scour at Abutment		
Peak Flood Closure	Low initial cost	Limits access, constant monitoring
Monitoring	Low initial cost	Does not prevent scour
Riprap	Familiarity, relatively low cost and maintenance, easy to construct, ability to adjust to minor scour	Can wash out, disturbs channel ecosystem until vegetation reestablished
Gabions	Relatively low cost, ability to adjust to minor scour	Can be undermined, stones can wash out of wire mesh, disturbs channel ecosystem
Cable-tied Blocks	Will not wash out as easily	More difficult to construct, higher maintenance
Tile Mats	Will not wash out as easily	More difficult to construct, higher maintenance, easier for water to lift
Alarm Systems	Low initial cost	Provides no scour protection, must be checked periodically
Articulated Mattress	Coherent structure, individual block will not wash out	More difficult to construct, easier for water to lift
Concrete-filled Mattress	Rocks will not wash out, relative ease of construction	Can be undermined, easy for water to lift
Locking Blocks	Coherent structure, individual block will not wash out	More difficult to construct, easier for water to lift
Pavement	Conceptually appealing	High cost and maintenance, can be undermined, easy for water to lift
Rock Bolting	Strong, low maintenance	Costly, only for abutments on bedrock
Grouted Riprap	Rocks will not wash out, relative ease of construction	Can be undermined, easy for water to lift
Sacrificial Piles	Conceptually appealing	Not effective, high cost
Grout Bags	Ease of construction, low cost	Bags can wash out
Sheet Piling	Stops flow, helpful in dewatering	Scour can occur near sheet piling, construction difficult, rust
Hinged-Slab /Tethered Block System	Will not erode under extreme velocities	Could be subject to edge undermining
River Control		
Spur dikes / Guide Banks	Proven effective	Can wash out, need to protect guide bank walls, obstructs navigation
Submerged Vanes	Elegant approach, not too expensive, effective	Obstructs navigation, possible debris snags, construction difficult
Collars	Low cost and maintenance, effective	Does not eliminate scour, not much experience
Attached Vanes	Low cost and maintenance, effective	Does not eliminate scour, not much experience

3.2 RECOMMENDED COUNTERMEASURES FOR ABUTMENTS

Based on the survey, the following countermeasures are recommended for application to bridges in New Jersey.

LOCAL SCOUR COUNTERMEASURES

- Riprap
- Gabions
- Grout / cement Bags
- Cable-tied blocks

RIVER CONTROL COUNTERMEASURES

- Guide Banks
- Spur Dikes

CHAPTER 4

COUNTERMEASURES APPLICABLE TO PIERS

4.1 SCOUR AT PIERS

Figures 4.1 to 4.6 detail the causes and locations of scour at pier foundations at typical bridge sites (CIRIA Manual on Scour of Bridges). Figure 4.1 details a situation when change in channel alignment at bends and skewed flow at piers causes scour at outer abutments and piers. Figure 4.2 details scour caused by construction of adjacent bridges. In such situations, modified flow at upstream bridge pier locations may accelerate scour at downstream bridge pier locations. In turn, the scour hole formed at downstream piers may extend to upstream bridge piers.

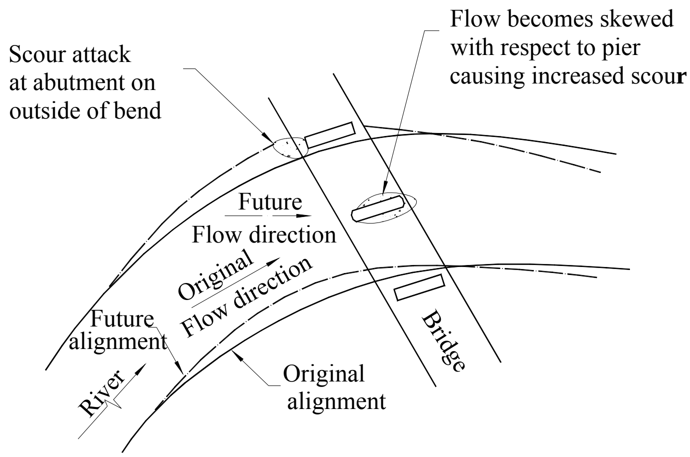


Figure 4.1. Change in Channel Alignment at Bend Causing Scour at Outer Abutment and Skewed Flow at Pier.

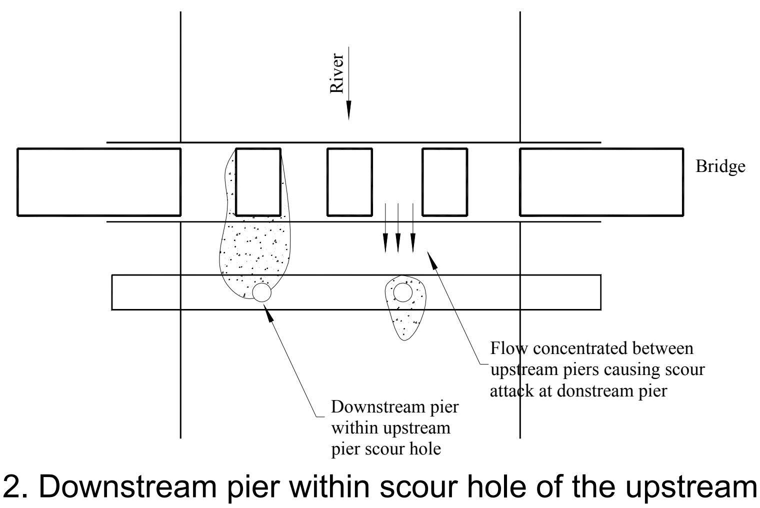


Figure 4.2. Downstream pier within scour hole of the upstream pier and concentration of flow at upstream pier because of Construction of Adjacent Bridges.

Figure 4.3 details a scour hole formed because of lateral movement of a main channel exposing pier foundations. Figure 4.4 details scour occurring because of lack of relief openings. Lack of relief openings may cause scour both at bridge piers and abutments. The change in direction of flow between normal and flood flows may result in skewed flows at bridge piers and abutments. This will result in significant scour activity as shown in Figure 4.5. Removal of downstream flow control devices, such as weirs, may result in gradual degradation of stream bed. This may expose both pier and abutment foundations over a period of time, as shown in Figure 4.6.

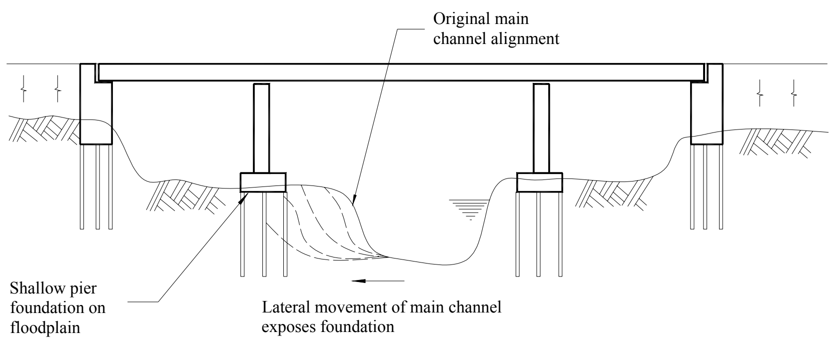


Figure 4.3. Lateral movement of main channel exposing shallow foundations on floodplain.

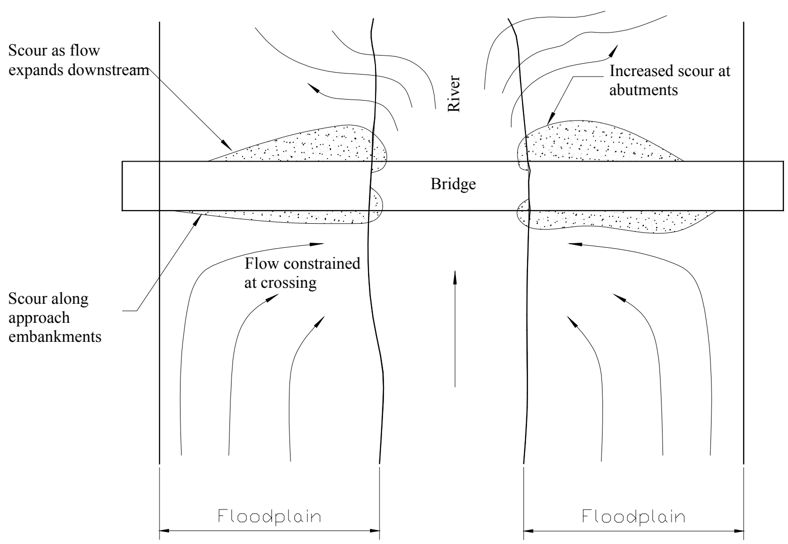


Figure 4.4. Lack of relief openings on floodplain

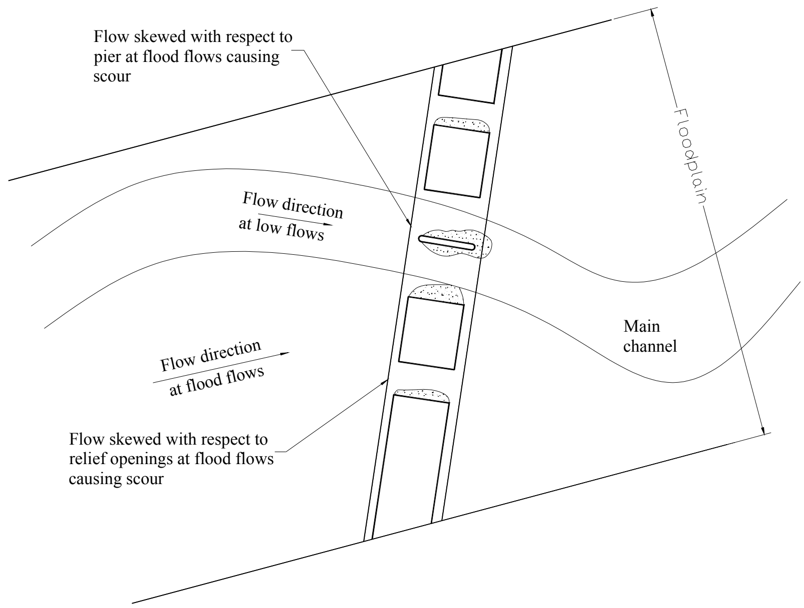


Figure 4.5. Change in flow direction between normal and flood flows resulting in skewed flows at piers, abutments and relief openings

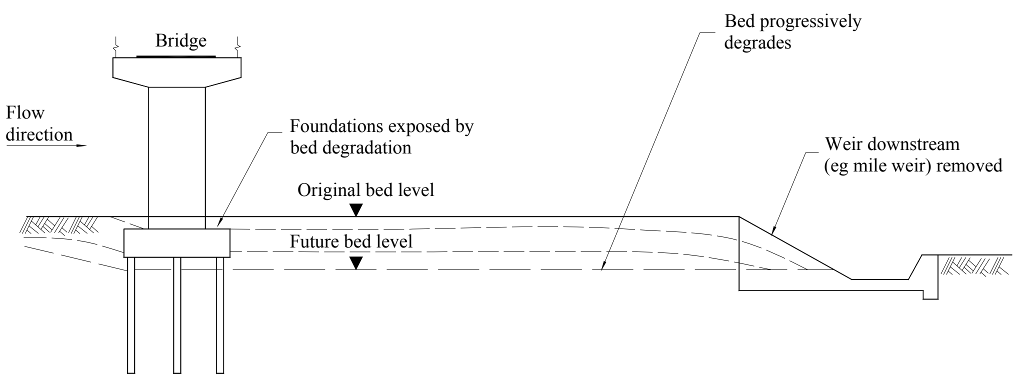


Figure 4.6. Removal of downstream bed control

4.2 EXPERIENCE OF 35 US STATES

Parker et al, 1998 (NCHRP 24-7) conducted an extensive review on scour countermeasures for piers of existing bridges based on literature review. A statistical analysis of the response of thirty five state DOTS provided details about various scour countermeasures in terms of:

1. Feasibility (site characteristics, hydraulics, bridge type, and other factors such as ice susceptibility, debris susceptibility, salt water and regional problems),

2. Technical effectiveness based on hydraulic, geotechnical, geomorphic, hydrologic, climatic, structural, synergistic effects, and inspectibility),

3. Difficulty in construction,

4. Durability,

5. Difficulty with maintenance

6. Cost and

7. Special factors such as social acceptability, aesthetics, and environmental acceptability.

Figure 4.7 shows the average of rankings for 33 countermeasures based on the factors described above. The ranking of the various countermeasures range from riprap, scoring 62% of the total points possible, to tile mats scoring 32%. While high rating of riprap is primarily because of its historic familiarity and wide spread use, the survey does highlight the use of countermeasures that are alternatives to riprap, such as

1. Gabions and Reno mattresses (50%),

2. Cable-tied blocks (44%),

3. Artificial riprap (55%),

4. High density riprap (54%)

Figure 4.8 shows the survey results on difficulty of maintenance of the different scour countermeasures. A higher rating implies a lesser level of difficulty. It is noted that among common countermeasures, riprap, artificial riprap, high density particles, sacked concrete and rock-filled gabions received ratings close to 50% or higher.

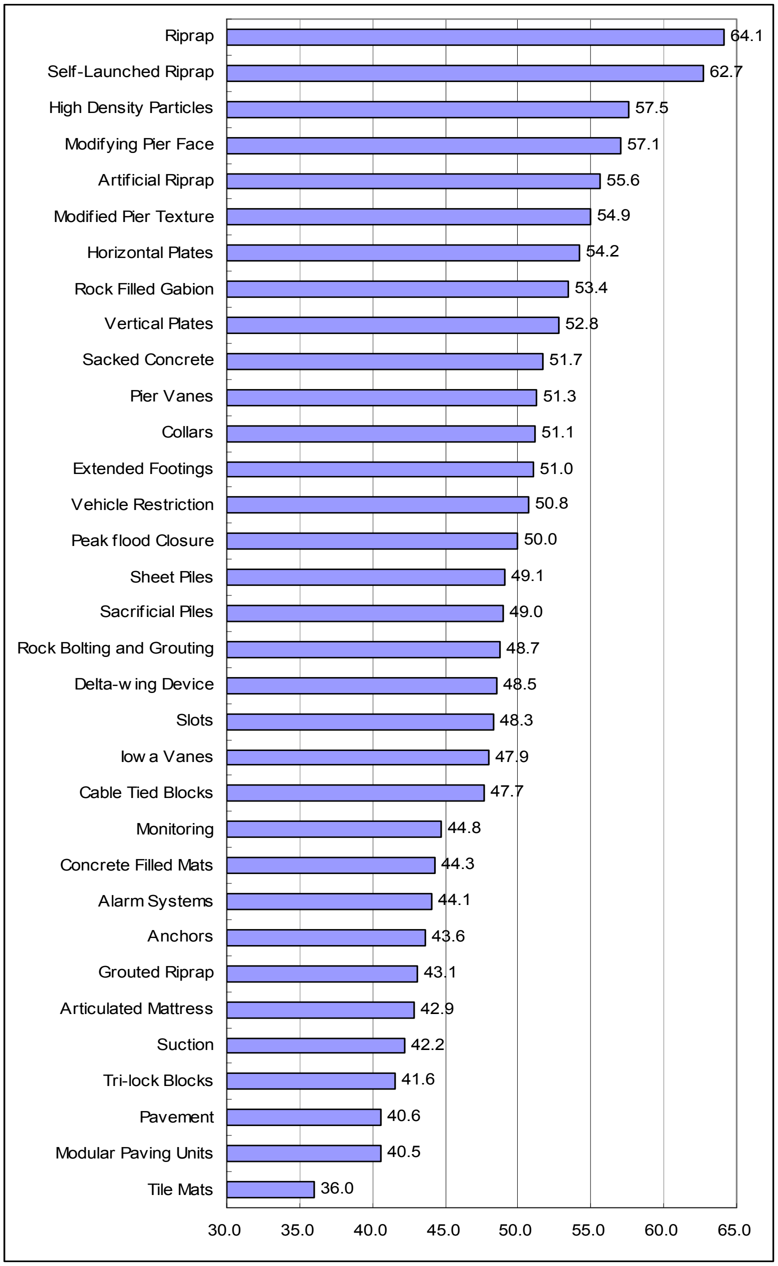


Figure 4.7. Average ranking of different scour countermeasures by 35 State DOTs (Based on data from Parker et al,1998; NCHRP 24-7)

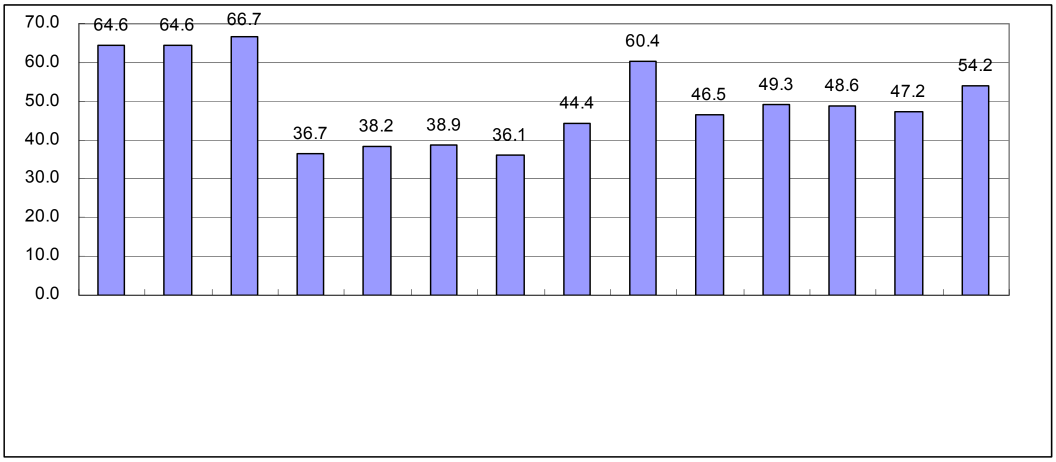


Figure 4.8. Difficulty with maintenance rating of different scour countermeasures by 35 State DOTs

4.3 RECOMMENDED PIER COUNTERMEASURES

Based on the described occurrences in Section 4.1, different factors will affect a screening and selection of countermeasures for a particular bridge site. The following pier countermeasures are recommended for applications to New Jersey bridges:

1. Armoring Countermeasures
    - Riprap
    - Gabions and Reno Mattresses
    - Articulated Concrete Blocks/Cable Tied Blocks
    - Concrete Armor Units

2. Flow Altering Countermeasures
    - Upstream Sheet piles
    - Flow Deflecting Vanes or Plates

Note: Flow altering countermeasures alone are not considered effective countermeasures for local scour at bridge piers. They must be used in combination with armoring countermeasures, to increase their effectiveness during high flow conditions.

3. Structural Countermeasures

   - Structural Repairs using Tremie concrete
   - Grout bags
   - Casting concrete aprons
   - Shielding by sacrificial piles
   - Sheet piles

Guidelines on applications of each of these countermeasures are detailed in individual chapters of the handbook.

CHAPTER 5

INTRODUCTION TO ARMORING COUNTERMEASURES AND REVETMENT

5.1 GENERAL

Armoring Countermeasures fall under HEC-23 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.

From a review of over fifty NJDOT Phase 2 In-depth review reports for scour critical bridges in New Jersey, it is seen that a majority of bridges located on rivers will be subjected to 100 year flood velocity (that is less than 10 ft/sec). If a flood velocity is less than 10 ft/sec, armoring countermeasures should be sufficient.

Armoring Countermeasures [HEC-23, Group 1.B]

According to HEC-23, 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.

1. During a flood, erosion may take place more commonly in the vicinity of the river constrictions, such as at bridges and culverts. For protection against scour at bridge and culvert sites, the commonly used armoring countermeasure is bed armoring. It is applicable at the following locations:

    a. Abutment (full height, stub, integral and spill through) with spread footings, pile caps, piles and drilled piers

    b. Piers with spread footings, piles caps, piles and drilled piers

        1.) Wall pier

        2.) Column bents

        3.) Pile bents

    c. Wing walls for abutments with spread footings, pile caps and piles

    d. Headwalls of culverts

    e. Wing walls for culverts with spread footings

2. The primary function of armoring countermeasures is reduction in velocity and energy dissipation, both upstream and downstream of the bridge. The failure of a riverbank or an embankment, upstream or downstream of bridge, may lead to

    a. A change in the direction of flow of river

    b. Increased scour at the bridge due to change in hydraulic conditions

    c. Closure of embankment and roadway

3. The following off bridge and off culvert locations are likely to be eroded after a major flood and where revetment is required:

    a. Banks of rivers, streams and channels

    b. Embankments at the intersection of bridge and roadway

    c. Stream bed adjacent to pier and abutments, where scour holes are formed.

    d. Upstream and downstream of culverts/toe of aprons

4. Commonly used armoring at culvert headwalls

    a. Rock riprap

    b. Concrete pavement or concrete apron

    c. Concrete armor units

5. Commonly used wing wall armoring: The height of wing wall decreases at locations away from the bridge, where scour will be minimum. Wing wall scour is maximum at the interface with an abutment, where its magnitude is the same as that of the abutment. The thickness of armoring may be reduced linearly if the change in wall height is 2:1. Due to ease of construction, use the same type of armoring as for abutments

Table 5.1 provides descriptions, advantages, disadvantages and recommendations for different armoring countermeasures in New Jersey.

Table 5.1. Applications of armoring for stream beds and banks

Countermeasure	Scour Type	Description	Advantages	Disadvantages	Remarks
Placed Rock riprap on geotextile or filter fabric layer	Local scour Degradation Lateral erosion	Graded broken rock placed below river bed in position by machine on geotextile layers and overlaid with soil	Geotextile acts as filter preventing bed erosion to a high degree. Familiarity. Relatively low cost/maintenance. Easy to construct, ability to adjust to minor scour	Labor intensive. Low cost of riprap is offset by placement cost Can wash out, disturbs channel ecosystem until vegetation reestablished	Recommended for low scour and flood velocities
Artificial Riprap	Local scour Degradation Lateral erosion	Alternatives to riprap such as tetrapods / toskanes	Useful if rock riprap is not available locally from quarry	Concrete is likely to prevent vegetation growth, unless buried below river bed	Recommended subject to meeting environmental requirements in NJ
Gabions / Reno mattress on geotextile layer	Local scour Degradation Lateral erosion	Galvanized woven or welded wire mesh baskets, mattress filled with loose stones. Wire should be coated with PVC	Rocks inside baskets do not move. Permits vegetation growth. Useful where large size rock is not available. Relatively low cost, ability to adjust to minor scour	Wire may break due to corrosion or cut by vandals. Tree branches /debris may trap in wires. Regular maintenance required. Scour holes need to be filled prior to excavation. Disturbs channel ecosystem	Cost effective. Recommended
Precast concrete interlocking blocks	Local scour Degradation Lateral erosion	Concrete blocks of a cellular shape placed as revetment	Useful if large size rock riprap is not available locally from quarry	Likely to affect vegetation growth or prevent marine life. Weep holes for relief of hydrostatic pressure required.	Recommended subject to meeting environmental requirements in NJ
Cable-tied blocks	Local scour Degradation Lateral erosion	Concrete blocks / slabs interconnected with steel cables	Minimum maintenance. Conforms to river shape better than concrete blocks. Suitable for piers. Will not wash out as easily	Not suitable for pile bents. Steel cables likely to corrode and affect water quality. Edges of revetment need to be anchored into underlying material. Expensive since divers required to tie blocks. More difficult to construct, higher maintenance	Recommended subject to meeting environmental requirements in NJ

Countermeasure	Scour Type	Description	Advantages	Disadvantages	Remarks
Sacked concrete / grout filled bags	Local scour Degradation Lateral erosion	Fabric bags filled with concrete and stacked to produce a protective layer. Sand filled bags preferable.	Suitable for sandy soils only and for filling scour holes under footings	Undermining of toe may result. Likely to prevent vegetation growth or marine life. Risk of pollution from cement wash out. Catastrophic failure potential due to poor interlocking.	Recommended for filling scour holes under footings
Vegetable planting Grasses, trees and shrubs	Degradation Lateral erosion	Trees planted to prevent bank erosion	Low cost. Suitable for natural appearance and varied habitat	Not applicable for steep banks or for soil with large size stones.	Recommended. Meets environmental requirements

5.2 COUNTERMEASURES COMBINED WITH RIVER TRAINING

Experience has shown that providing armoring countermeasures alone may not be adequate and a combination of river training measures and armoring is necessary for high velocity rivers. By providing river training measures, less pressure will be put on the armoring mechanism. Accordingly, the effectiveness of the system will be increased. However, since large investments would be involved, economic considerations become important. Hence, cost reductions should be adopted in the design detailing by optimizing the depth and width of armoring mechanism that are provided as revetment. Using scaling factors in the next section, riprap or gabion blankets may be used.

It is normal practice to protect 100 to 300 feet of riverbanks by revetment at upstream and downstream of bridges and culverts. They differ from bed armoring in that they have a smaller thickness and are longer.

Their sizing takes into account correction factors for stability, gravity and angle of repose of riprap. In addition to mattresses, continuous framework of articulated concrete blocks and grout bags have been used for revetment. Filter should be used when utilizing concrete blocks and grout bags. HEC-23 Design Guideline 12 provides examples of revetment designs.

The common types of revetment in use and listed in HEC-23 are:

1. Dumped riprap
2. Wire enclosed riprap mattress
3. Articulated concrete block system
4. Grout filled mattresses
5. Concrete pavement

It is recommended that some type of river training measure be provided, in addition to armoring countermeasures when:

1. The flood velocity exceeds 10 ft/sec
2. The bridge carrying traffic volume exceeds an ADT of 500.
3. Revetment

5.3 SCALING DOWN SIZES OF ARMORING

With the view of obtaining an economical solution, certain scaling factors are proposed for use in optimizing an armoring. These factors are approximate but are based on engineering judgment and on observations of past performance of the type of countermeasure. Use of scaling factors is to provide the following advantages:

1. Reducing the volume of required armoring and associated labor charges.

2. Minimizing adverse impacts of armoring on the river's natural environment.

In the following situations, the size of armoring countermeasures may be scaled down by presenting the method presented herein.

1. When the scour depth is determined by use of a 500-year storm, the determined value could be interpreted to be a conservative value. As discussed below, the determined scour value may be divided by using a maximum factor of safety of 2 or by a scale factor with a maximum value of 2.

2. HEC-18 procedures for scour analysis are based on extreme hydraulic conditions, such as designing for 100 years floods and checking for 500 years. This approach may be satisfactory for planning of new bridges for a useful life of 100 years. But when the bridge is earmarked for replacement in the near future, this becomes over conservative. Scaling factors in this section may be used in such situations to reduce the fiscal impact.

3. Deep excavation that is required for armoring anchoring (equal to the computed scour depth) next to bridge footings may cause settlement and may be expensive due to the high cost of steel or timber cofferdams. It may also disturb the fauna and flora in the river environment. For this occurrence, the required armoring may be scaled down.

The influence of scaling factors may be linear or non-linear. The following scaling factors are proposed for modification of computed armoring sizes:

1. Scaling Factor ζ for application to the geology of soil

Weak soil	0.90
Sandy	0.85
Weathering rock	0.80

2. Scaling Factor η for application to width of the bridge opening,

Small width < 30 feet	0.90
Medium width > 30 but ≤ 50	0.85
Large width > 50 but ≤ 70	0.80

3. Scaling Factor ξ for application to river training measures,

With effective river training measures	0.80
For medium river training measures	0.85
For no river training measures	0.90

4. Scaling Factor χ, for application to the remaining bridge life assessed,

For bridge replacement < 10 years	0.80
>10 but ≤ 15 years	0.85
> 15 years	0.90

5. Scaling Factor ψ for application to underwater inspection,

For regular inspections including post floods,	0.80
For routine inspections but not post floods	0.85
For limited underwater inspection	0.90

Scaling Factor (SF) = $(\zeta)\, l \times (\eta)\, m \times (\xi)\, n \times \chi \times (\psi)$ (5-1)

Where l, m and n < 1. where l, m, n are factors which may be assumed as unity.

For New Jersey bridges, they are obtained from calibration of scour critical bridges in the Phase II Study.

Maximum SF = 1.0 i.e. provide armoring thickness = depth of local scour.

Minimum SF = 0.5 for weathering rock. For good quality rock no armoring is required due to minimal erosion.

The depth of riprap acts like a protective shield at the vertical face of spread footing.

Example of Scaling

Given

Footing depth = 3’,

Frost depth = 2’

Determined Depth of Riprap, Y = 5’

Determined Width of Riprap, WR = 14.5’ for 2:1 soil slope

= 9.5’ for 1:1 soil slope

Scale Factor (SF) determined using Equation (5-1) = 0.5

Width WR’= 0.5 x 14.5’ = 7.25’ for soil with 2:1 slope. Use 8’

WR’ = 0.5 x 9.5’ = 4.75’ for 1:1 ground slope. Use 4’ minimum from the face of footing

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CHAPTER VI

ROCK RIPRAP AS BED ARMORING AND REVETMENT

6.1 DESCRIPTION

Riprap is defined as a layer of natural rock. Because of its sheer dead weight, it acts like a shield and protects the soil underneath. It prevents direct contact of soil with the erosive forces that are generated at high flood velocities. Riprap prevents the strong vortex motion at the front of a pier from entraining bed sediment and forming a scour hole. Riprap is flexible and it adapts itself to the new profile after the settlement of soil. The ability of a riprap layer to settle into a developing scour hole and armor the base of scour hole is an important factor in the protection offered by riprap. Riprap blends well with the river environment compared to other countermeasures and provides good habitat for invertebrates. Its availability, economy, ease of installation, and flexibility are considered highly desirable characteristics.

The performance of riprap has been verified for different conditions over a long period of time. It is both cost effective and environmentally acceptable. Properly sized riprap in relation to a peak flood velocity placed over geotextile and constructed according to standard detailing procedures has shown satisfactory results. **However, riprap installation for existing bridges is regarded as a temporary remedy and not as a permanent countermeasure.**

With adequate monitoring, bridge foundations with riprap installations may be maintained indefinitely. Eventually, bridges with such foundations may be replaced.

Riprap may be used alone or in combination with guide banks, sacrificial piles or structural countermeasures. For example, the direction of flow may no longer be parallel to abutments and piers due to river meandering over a long period; thereby flood velocity and local scour are increased. In such cases, riprap alone will not be effective and guide banks may also be required.

6.2 MODES OF FAILURE

Riprap failure mechanisms are affected by riprap size. From extensive studies under peak flood conditions, the failure mechanisms have been identified as follows:

1. Riprap Blanket

    a. Hydraulic erosion failure: Causes include inadequate stone size, steep side slopes, inadequate gradation and removal of stones by impact. Hydrodynamic forces of flowing water may dislodge individual stones.

b. Winnowing failure: Underlying finer material is removed through the voids of the riprap. This type of failure can be avoided by increasing the number of layers

c. Edge failure: Due to channel scour, the toe of riprap blanket is undermined and scour holes are formed. Making the layer of riprap sufficiently thick can prevent edge failure.

2. Sloping Riprap

   a. Slump failure: The causes include steep side slopes, presence of impermeable material that acts as fault plane when subject to excess pore pressure. Due to shear failure of underlying material, a rotational-gravitational movement of material may occur along a rupture surface with a concave upward curve shape. Riprap stones cannot withstand the horseshoe vortex associated with scour mechanism. This type of failure can be avoided by increasing the median size of stone.

   b. Modified slump failure: The causes include steep side slopes and disturbance of material in the lower riprap layers. Mass movement of material may occur along an internal slip surface within the riprap layer.

   c. Transnational slide failure: The causes include steep side slopes, excess pore pressure, and toe failure due to undermining. A down-slope movement of a mass of riprap occurs, with the fault line on a horizontal plane.

   d. For a spill-through abutment, the initial failure zone begins at the armored floodplain downstream of the contraction near the toe. For a vertical-wall abutment, the initial failure zone occurs at the upstream corner of the abutment [Pagan-Ortiz (1991)].

6.3 GENERAL DESIGN GUIDELINES

The following general design guidelines apply to both piers and abutments for the design and installation of riprap.

1. Existing Foundations: Records of construction or drawings showing footing sizes and elevations of existing bridges may not be available to plan the extent of riprap and excavation. Probes or boreholes may be drilled adjacent to abutment and pier walls to locate the footings.

2. Quality of Stones: Only natural field stone or quarry stone shall be used for riprap installations. The stone shall be hard, angular or of approximately rectangular shape. The quality shall be such that it will not disintegrate on exposure to water or weathering.

3. Excavations: The top layer of riprap shall be at least 6 inches below a riverbed to avoid encroachment of the river or dislodging of stones by floating debris, ice or currents. Prior to placing the riprap, excavation of a ditch of specified dimensions shall be required and shall be carried out in layers. Abutments and piers need to be supported during excavation. Underpinning of footings may be required. Footing widths or raking piles may inhibit excavation. Adequate precautions should be taken to avoid damage to existing foundations. For safety of construction personnel, the slope of excavation should be properly planned. Depending on the angle of repose of the soil, slopes shall conform to OSHA standards.

4. Geotextile Layer / Filter: A geotextile or granular filter layer shall be placed under the riprap layer. It is not necessary to apply geotextile or granular filter on gravel streams. Specialized construction techniques are required for geotextile placement. A 3-inch thick gravel cushion placed on geotextile is recommended to avoid rupturing. More details on geotextile layer / filter are provided in Chapter 11 of this handbook.

5. Equipment: Excavation machinery, drilling equipment and small cranes for lifting heavy stones may be required.

6. Maintenance: Since riprap is buried, no maintenance is required. However, if riprap gets exposed or scattered on riverbed except after heavy floods, remedial work should be planned.

7. Riprap for Pile Caps and Exposed Piles: Since pile clusters are generally closely spaced, it may not be feasible to place riprap around them. Instead Tremie concrete may be used under a pile cap. Riprap may then be placed around the pile cap.

8. Design Parameters: Specific design procedures for piers and abutments are based on size of riprap, gradation, layer thickness, horizontal extent, placement techniques, filters and type of equipment. A set of scour calculations illustrated in Table 6.1 that is based on HEC-18 procedures, shall be prepared. Scour depth increases with the increase in the Froude Number and it is directly proportional to velocity. The depth of riprap will increase proportionally to both approach velocity and the Froude Number.

9. Scour Depth: Scour Depth is higher for finer bed material and decreases as bed material becomes coarse. The depth of riprap will decrease proportionally as size of bed material increases.

Table 6.1. Scour data at abutment / pier

Discharge Frequency	Computed Scour Depths					Flood Data	
	Long term Scour	Short term Scour	Contraction Scour	Local Scour	Total Scour	Max. Elev.	Max. Flood Vel.
50 Year							
100 Year							
500 Year *							

If 500 year Discharge is not available, use Q500 = 1.2 x Q100

10. Flow Parameters: The following flow parameters required to calculate the size of rock are

    a. Flow velocity, V

    b. Velocity magnification factor, K

    c. Density of riprap

    d. Water depth, d

11. Common Causes of Failure: In order to prevent the failure of riprap during floods, the following common causes of failure need to be considered and controlled in riprap design.

    a. Inadequate rock size

    b. Inadequate gradation

    c. Absence of filter layer

    d. Internal slope failure

    e. Riprap being placed at steep slopes

    f. Poor toe design

    g. Degradation of channel

12. Layout: Three or more layers of riprap will provide a continuous blanket similar to a concrete pavement. Guidelines for the layout of riprap for piers and abutments are detailed in Sections 6.4 and 6.5, respectively.

13. Size: The median size of riprap D_{50} shall be determined using guidelines in Section 6.4 for piers and Section 6.5 for abutments. Minimum D_{50} size of riprap shall be R-6 and maximum D_{50} size shall be R-8 as per NCSA rock size and gradation in Table 6.2.

Table 6.2. NCSA Rock Size and Gradation

Percent Pass (Square Openings)						
Class, Size No. (NCSA)	**R-8****	**R-7****	**R-6**	**R-5**	**R-4**	**R-3**
Rock Size (Inches)						
48	100*					
30		100*				
24	15-50		100*			
18		15-50		100*		
15	0-15					
12			15-50		100*	
9		0-15		15-50		
6			0-15		15-50	100*
4				0-15		
3					0-15	15-50
2						0-15
Nominal Placement Thickness (inches)	48	36	30	24	18	12

*Maximum Allowable Rock Size.
**Use Class 2, Type A Geotextile

14. Gradation: The resistance of riprap to erosion depends on gradation of stones. For riprap gradation, the diameter of largest stone size shall be 1.5 times D_{50} size. The following size distribution will be used unless otherwise recommended.

Table 6.3. Rock riprap gradation [Brown and Clyde (1989)]

Stone Size Range	% of Gradation < Than
1.5 D_{50} to 1.7 D_{50}	100
1.2 D_{50} to 1.4 D_{50}	85
1.0 D_{50} to 1.15 D_{50}	50
0.4 D_{50} to 0.6 D_{50}	15

15. Exceptions to depth of riprap: The following circumstances may preclude the use of ripraps.

    a. If computed scour depth is high, it may not be feasible to provide riprap depth equal to full scour depth. Deep excavations adjacent to spread footings are not permissible. In such situations, the thickness of riprap may be reduced to d=0.5y, where y = maximum scour depth, provided ‘d’ is not less than 3 feet, a filter layer is used and, if effective, monitoring is done.

b. If the excavation depth is not excessive and does not cause instability of footing, a higher value of ‘d’ between 0.5y and y may be used. The size of stone, however, will not be reduced and will be based on calculated velocity. If the design depth ‘d’ is greater than the available depth between riverbed elevation and bottom of footing or bedrock is not available within the depth ‘d’, an alternate countermeasure should be considered.

6.4 DESIGN OF RIPRAP FOR BRIDGE PIERS

The use of riprap blankets is recommended only for existing bridges due for replacement in a few years. The use of riprap should be restricted to low and medium flood velocities not exceeding 10 feet per second.

1. **Sizing**: The HEC-18 equation [Richardson and Davis (1995)] is recommended for sizing riprap at pier footings. However, the maximum rock size should not exceed $2D_{50}$.

$$D_{50} = \frac{0.692(KV)^2}{(S_s - 1)2g} \qquad (6\text{-}1)$$

Where:

D_{50} = Median stone diameter, (ft)

K = Coefficient for pier shape

V = Velocity on pier, (ft/s)

S_s = Specific gravity of riprap (normally 2.65)

g = 32.2 ft/s^2

K = 1.5 for round-nose pier and 1.7 for rectangular pier

2. **Thickness of Mat:** The recommended thickness of riprap mat is $2D_{50}$. The top of riprap should be flush with bed at low flow. The minimum thickness should not be less than 12 inches.

3. **Layout of Riprap Mat**: Figures 6.1 and 6.2 show the layout of riprap for bridge pier footings. The width of a riprap layer adjacent to a footing (WR), around the pier should be the greater of the following:

   a. Width of scour hole

   b. 1.5W (W=width of pier at base) or 1.5W/cos(β) when β > 15º in Figure 6.2.

   c. (1 foot minimum + d cot φ) where d is the design scour depth at any abutment or pier and φ is the angle of natural repose for the soil, as obtained from geotechnical report.

Place riprap around footings with the slope starting at a distance that is a minimum of 1 foot from the vertical face of the footing.

4. **Filter Requirement:**

   The use of a filter, or alternatively a geotextile (filter cloth), is of particular importance in ensuring that finer material does not leach through or winnow around the riprap. No filter required for gravel beds. For sand beds, use a geotextile filter cover equal to the width of a pier (W) from the face of the pier in each direction, as shown in Figure 6.2.

5. **Gradation:** The following distribution of riprap sizes should be used.

   100% finer than $1.5D_{50}$
   80% Finer than $1.25D_{50}$
   50% Finer than $1.0D_{50}$
   20% Finer than $0.6D_{50}$

6. **Effectiveness Issues**

   a. Effectiveness depends on the seal around piers

   b. Reduced tendency of rock dispersal

   c. 50% less volume of rocks

   d. Granular filters subject to degradation

   e. More effective if tied into abutment countermeasure when pier is located within 3 pier diameters of abutment footings.

7. **Constructability Issues**

   a. Excavation Required

   b. Sealing geotextile to pile bents is difficult

   c. Limited ability to pre-excavate due to pier footing and/or pile geometry

   d. Specialized construction techniques for geotextile placement

   e. Gravel cushion on geotextile to avoid rupturing

   f. Performance dependent on construction sequence

8. **Placement of Riprap:** Riprap around bridge piers can either be installed from bridge deck or from the bed / banks of a stream.

a. **Installation from Bridge Deck**: Riprap is installed from a bridge deck by dumping from trucks and spreading by loader. Since stones are placed irregularly, they are unstable. Also, there are voids between the stones, through which fine particles of soil travel under water pressure and cause erosion. The success rate with dumped riprap at many existing sites may be misleading since the bridge may not have been subjected to peak design floods of 100 years. For large bridges, installation from bridge deck can be done by machine placing from a dragline or from buckets.

b. **Installation from Bed / Banks of Stream**: In this method, by hand placing and packing, a compact, mortarless masonry type construction can be achieved. Stones pack into a close interlocking layer will minimize the size of voids. This method of installing riprap results in a stable configuration and in uniformity of stone size distribution. The quality of construction is better than the dumping method. Also, a fish channel made within the top stone layers can be maintained. Although this method is more expensive than other methods, its use is recommended.

9. **Durability Issues**

   a. Broad band of failure threshold potential

   b. Catastrophic failure if riprap is exposed

   c. Geotextile fails abruptly

10. **Maintainability Issues**

   a. Under-bed installation increases durability.

   b. Less maintenance than dumped riprap

   c. Difficult to repair ripped geotextile or locate damage to riprap

   d. Clean up difficult after failure

   e. Gravel filters easier to maintain than geotextile

11. **Cost Issues (may vary according to local conditions)**

   a. Geotextile more expensive than granular filter

   b. Pre-excavation costs \$25/yd^3

   c. Disposal costs

   d. Less stone costs

   e. Traffic disruptions

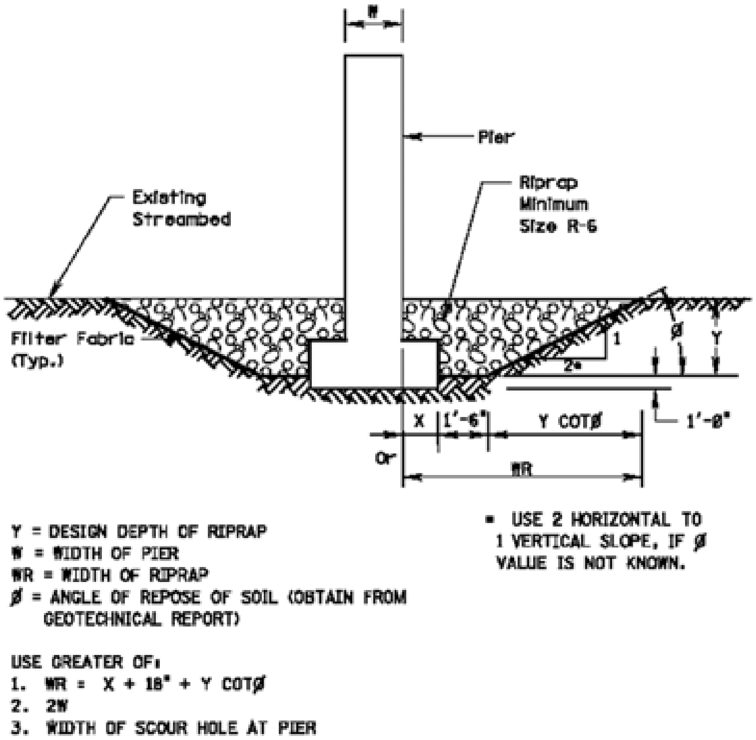


Figure 6.1. Pier Cross-Section Showing Riprap Details at Piers

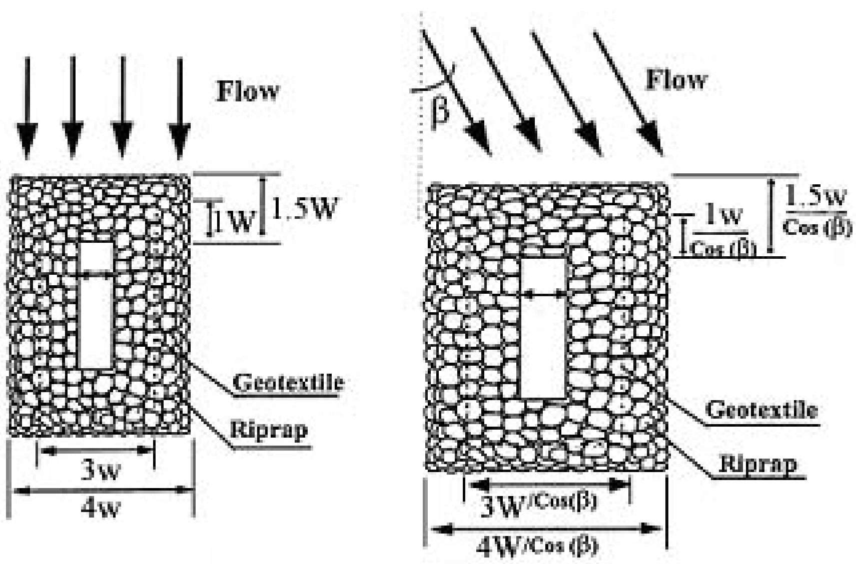
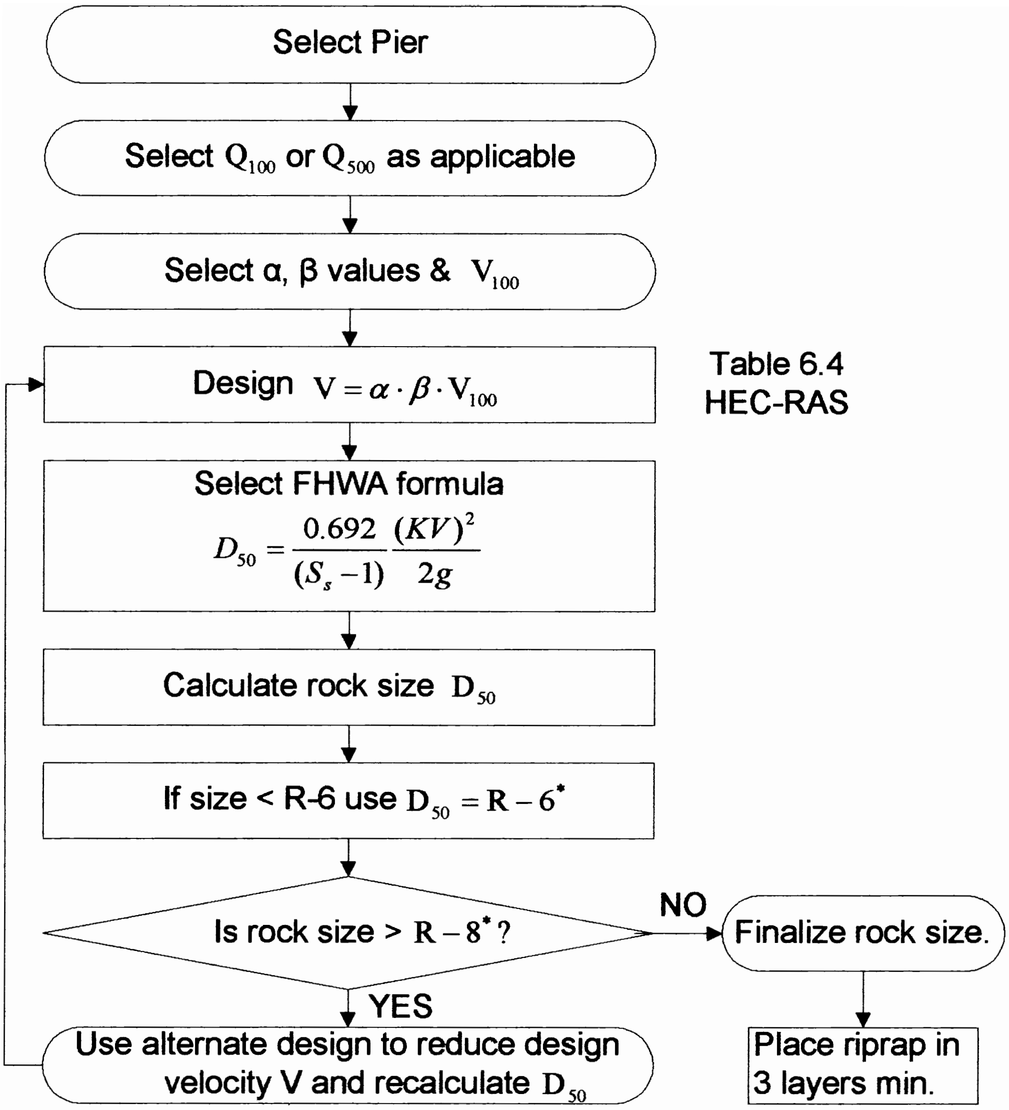


Figure 6.2. Extent of Riprap and Geotextile Filter at Piers

Figure 6.3 presents a flowchart for the systematic design of riprap sizing based on the Equation 6-1 described above. Table 6.4 presents flow parameters required in flowchart in Figure 6.3 for the calculation of riprap size.



* NCSA Classification in Table 6.2

<u>Riprap at Pier on Spread Footing (on soil).</u>

NOTES:

1. If rock exists at a depth lower than the design depth, place bottom of footing at 150mm below rock surface.
2. Set design depth =½ scour depth if only riprap countermeasure is used.

Figure 6.3. Flowchart for the design of riprap at bridge piers.

Table 6.4. Coefficients for evaluating riprap sizes at existing piers.

FLOOD EVENT	Velocity Multipliers				Shape Factor K	Shape Factor K
	Q100		Q500			
COEFFICIENTS	α Location factor	β 100 year flood factor	α Pier location factor	β 500 year flood factor	Round Nose	Rectangular
PIER	1.2	1.0	1.2	1.1	1.5	1.7

V = velocity , S_s = Specific gravity of stone (assume 2.24)
D_{50} = Median riprap diameter

6.5 DESIGN OF RIPRAP FOR BRIDGE ABUTMENTS

Design guidelines for riprap at bridge abutments are based on,

1. "Stability of Rock Riprap for protection at the toe of abutments located at the floodplain" published in 1991
2. "Design of Riprap Revetment", Hydraulic Engineering Circular -11.
3. Design Guidelines 8 and 12 of HEC-23.
   a. **Side Slopes**: For preventing slump failure*,* the side slope is a significant factor in the stability of riprap. It is desirable to decrease the steepness of abutments; thus increasing the stability of the riprap on the slopes. Recommended minimum value for side slopes varies from 1:2 to 1:1.5, (H:V).

   b. **Riprap sizing for abutments**: For New Jersey conditions, HEC-18 equation is recommended until better equation based on hydraulic tests is available,

$$\frac{D_{50}}{y_2} = \begin{cases} \dfrac{K_s}{(S_r - 1)} Fr_2^{\,2} & Fr_2 \le 0.8 \\ \dfrac{K_s}{(S_r - 1)} Fr_2^{\,0.14} & Fr_2 > 0.8 \end{cases} \tag{6-2}$$

Where:

Fr_2 = Froude number in the contracted section = $\dfrac{V}{(gy)^{0.5}}$

K_s = shape factor

= 0.89 for $Fr_2 \le 0.8$, 0.61 for $Fr_2 > 0.8$ for spill-through abutments

= 1.02 for $Fr_2 \leq 0.8$, 0.69 for $Fr_2 > 0.8$ vertical-wall abutments

c. **Extent of Riprap Protection**:

Vertical-Wall Abutments: For vertical wall abutments without wingwalls, wingwalls at 90° or splayed wingwalls, the width of a riprap layer (WR) adjacent to a footing at the river side of an abutment should be the greater of the following:

1.) Width of scour hole

2.) 2W (W=width of abutment at the base) or 2W/cos(β) when β > 15º (β is the same as defined in Figure 6.2).

3.) X+18"+y cot φ where X is the width of abutment footing, y is design scour depth at abutment and φ is the angle of natural repose for the soil, as obtained from geotechnical report.

Place riprap around the footings with the slope starting at a distance of a minimum of 1 foot from the vertical face of the footing. Figures 6.4 to 6.6 show a plan view of riprap layout for vertical wall abutments without and with wingwalls.

Spill-Through Abutments: For spill-through abutments, extend the riprap around the abutment and down to the expected scour depth. The launching 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. 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, not exceeding 24.6 feet (7.5 m). Figure 6.7 shows the plan view of riprap apron for spill-through abutment.

Figures 6.8 to 6.10 show typical layouts of abutment riprap for abutments near a channel bank, stub abutment near the top of high channel bank, stub abutment near top of high channel bank and abutment near flood plain used by Maryland HAS. In these figures, minimum riprap blanket thickness, minimum D_{50} size and approximate D_{50} weight for Class 1, 2 and 3 types are shown in Table 6.5.

Figure 6.11 presents a flowchart for the systematic design of riprap sizing based on Equation 6-2 described above. Table 6.6 presents various parameters that are required in the flowchart of Figure 6.11 for riprap sizing.

Table 6.5: Riprap D_{50} Size and Blanket Thickness in Figures 6.8 to 6.10.

Riprap Class	D_{50} Minimum Size (in)	Approximate D_{50} Weight (Pounds)	Minimum Blanket Thickness (In)
1	9.5	40	19
2	16	200	32
3	23	600	46

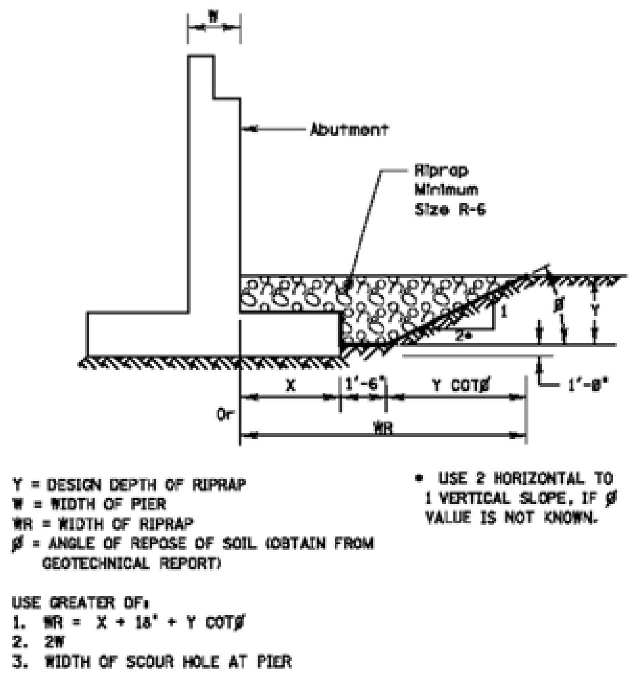


Figure 6.4. Cross Section of Riprap Details At Abutment

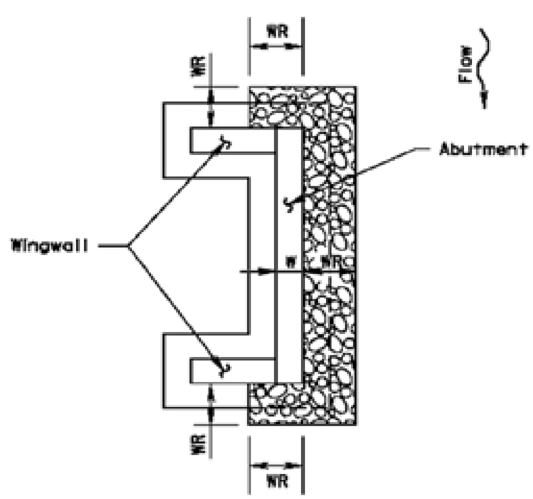


Figure 6.5. Plan of Riprap At Abutment - Wingwalls at 90 degrees.

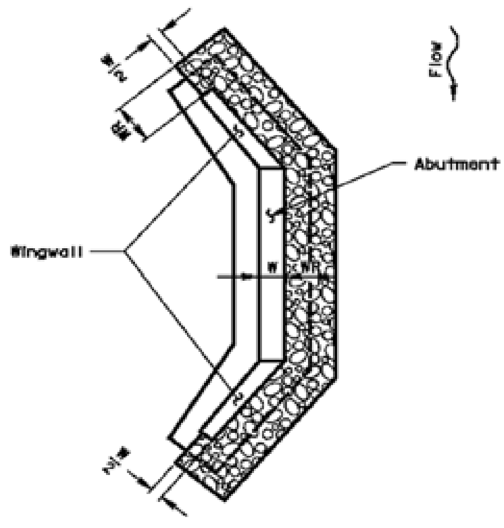


Figure 6.6. Plan of Riprap at Abutment - Wingwalls Splayed.

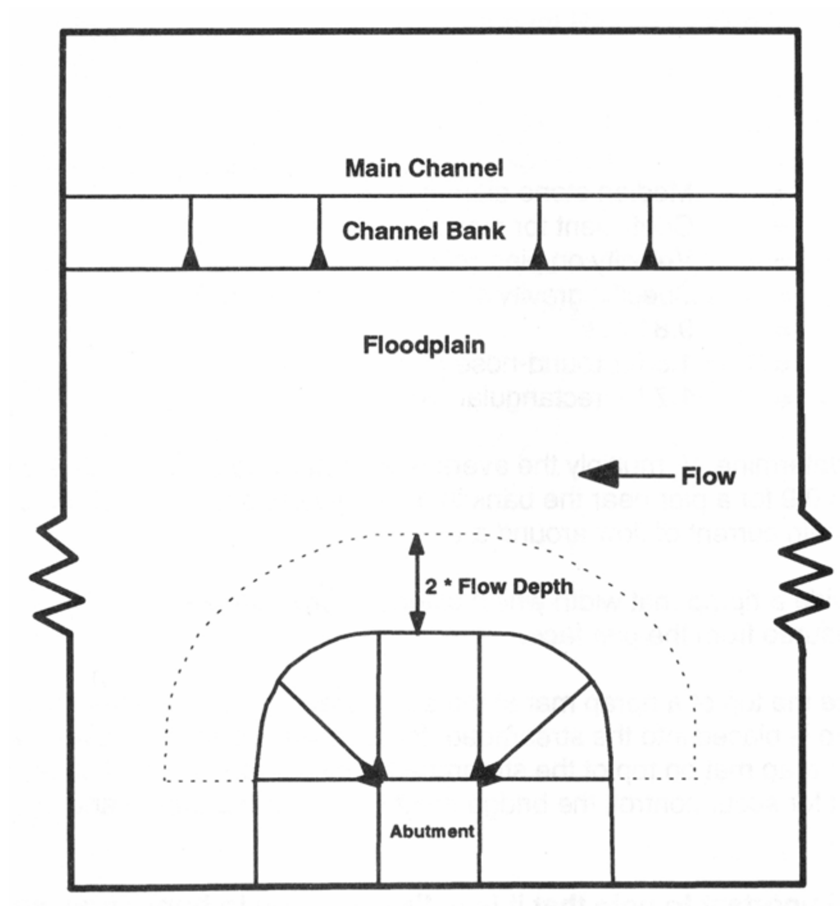


Figure 6.7. Plan view of the extent of rock riprap apron [HEC-23].

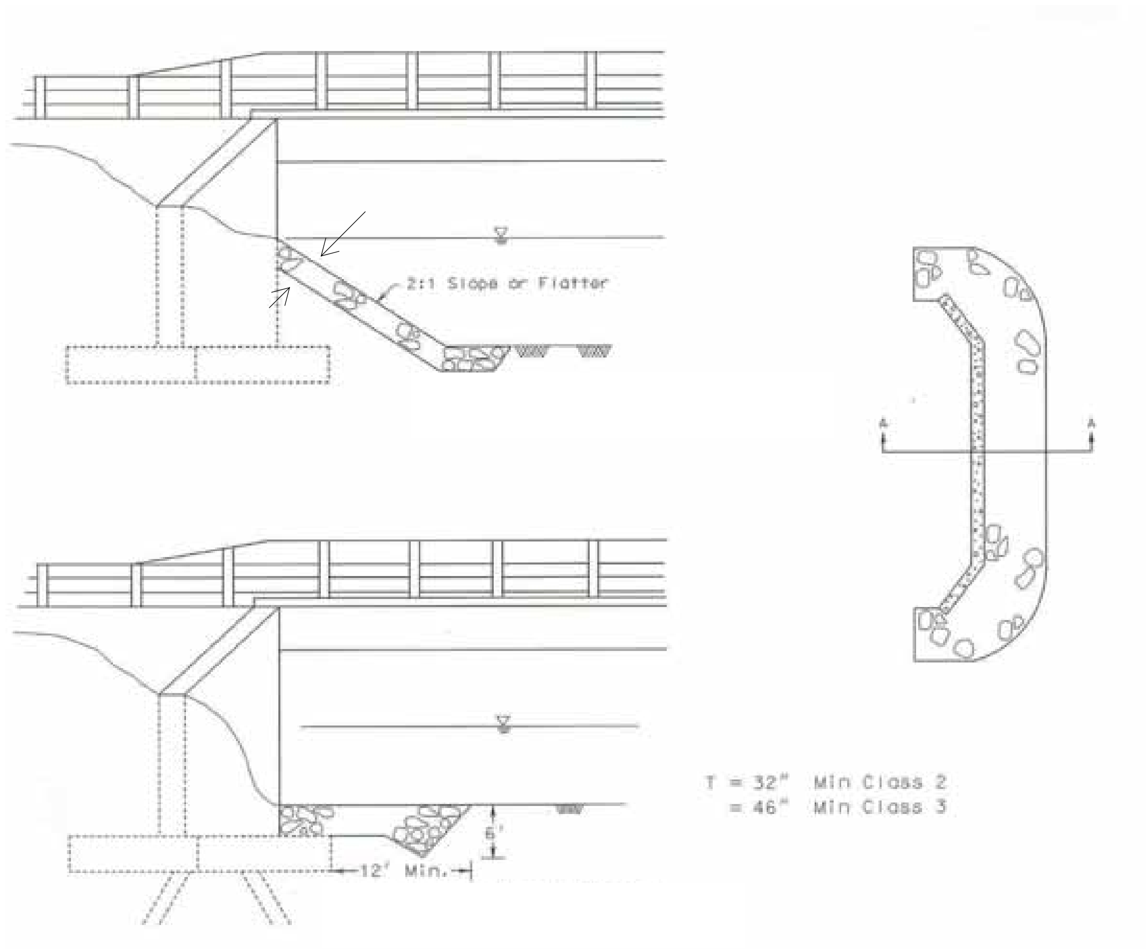


Figure 6.8. Plan & Typical Sections A-A of Abutment Near Channel Bank (Reference MD SHA)

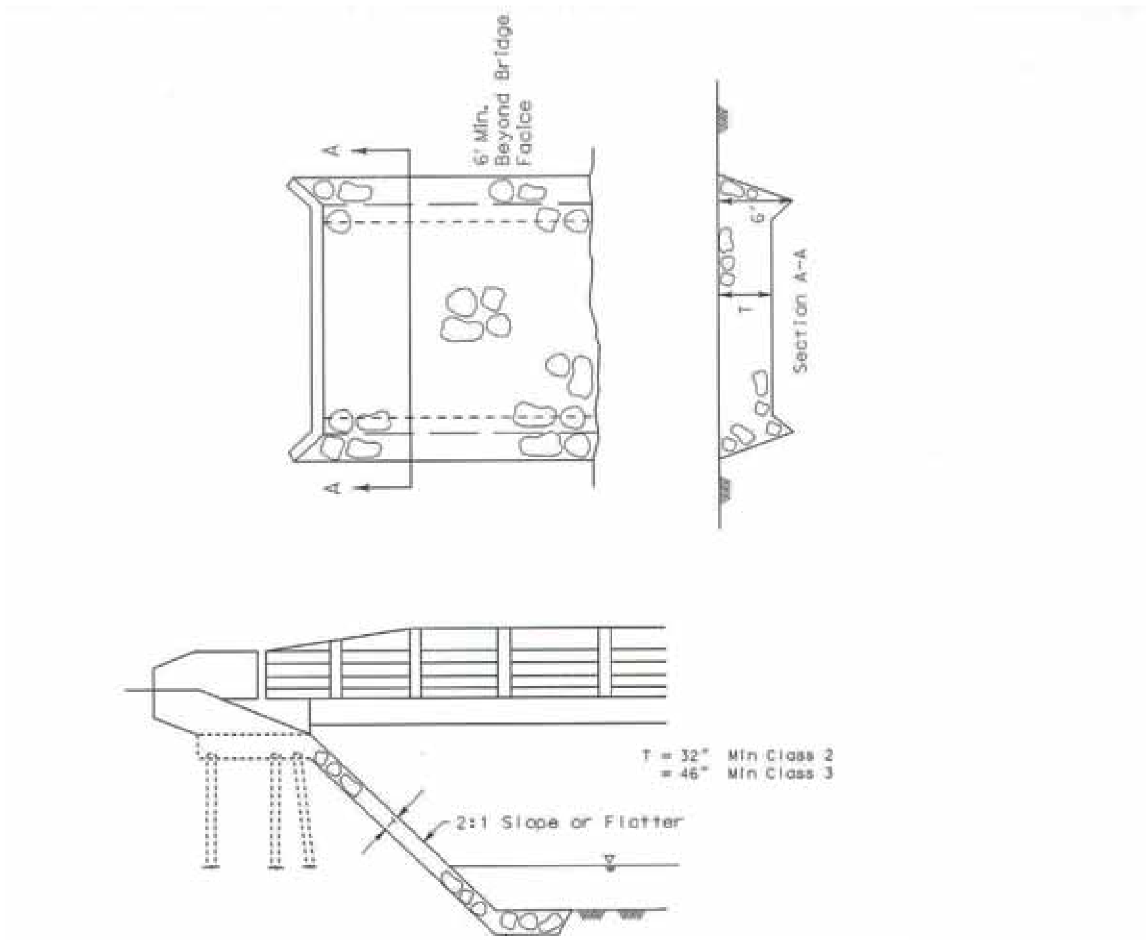
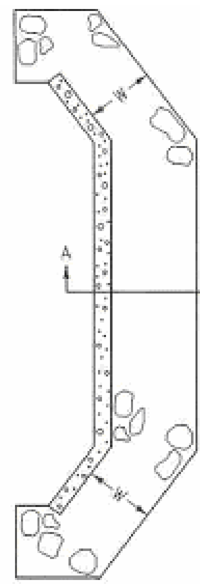


Figure 6.9. Plan & Typical Section of Stub Abutment Near Top of High Channel Bank
(Reference MD SHA)



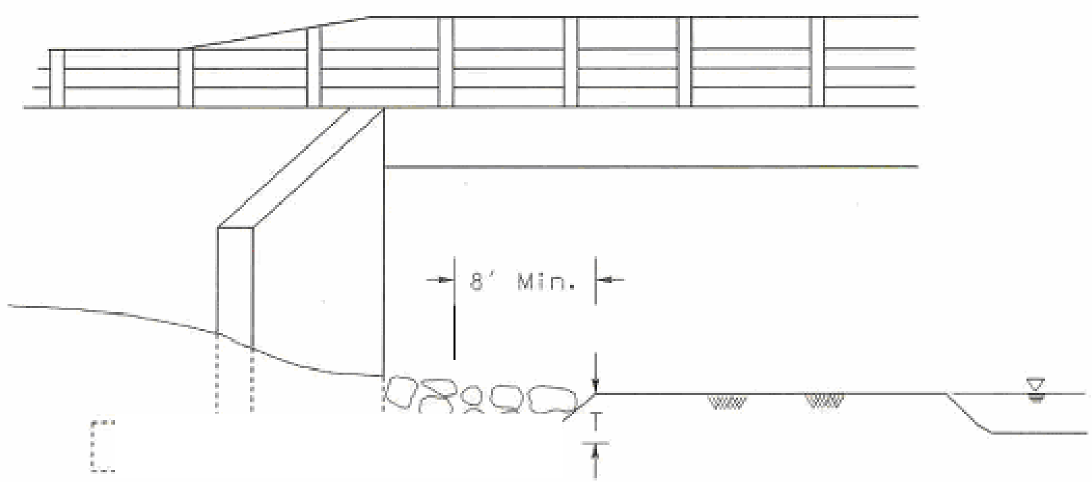
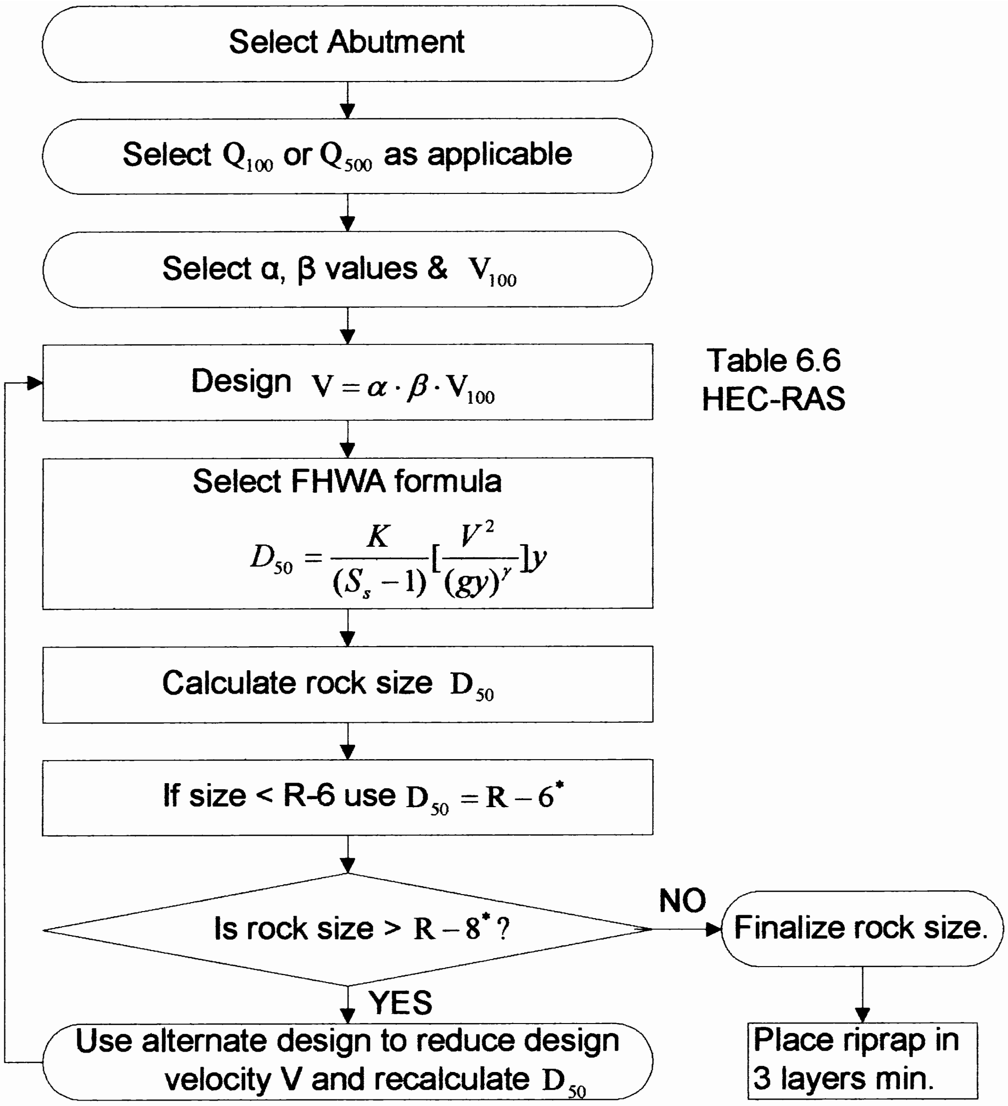


Figure 6.10. Plan & Typical Section A-A of Abutment in Flood Plain (Set Well Back from Channel Bank with Low Flow Depths and Velocities for Worst Case Scour Conditions).



* NCSA Classification in Table 6.2

<u>Riprap at Abutment on Spread Footing (on soil).</u>

NOTES:

1. If rock exists at a depth lower than the design depth, place bottom of footing at 150mm below rock surface.
2. Set design depth =½ scour depth if only riprap countermeasure is used.

Figure 6.11. Flowchart for the design of riprap at bridge abutments.

Table 6.6. Coefficients for evaluating riprap sizes at abutments

Velocity Multipliers				Froude Number $V/(gy)^{0.5} \leq 0.8$ Local Acc. Factor K			Froude Number $V/(gy)^{0.5} > 0.8$ Local Acc. Factor K		
Q100		Q500							
α Location factor	β 100 year flood factor	α Pier location factor	β 500 year flood factor	Spill-Through	Vertical -Wall	γ	Spill-Through	Vertical -Wall	γ
1.0	1.0	1.0	1.2	0.89	1.02	1.0	0.61	0.69	0.14

4. **Riprap Size Distribution:** Riprap size distribution should be the same as that for bridge piers.

5. **Constructability / Placement of Riprap at Bridge Abutments:** Performance of riprap as a scour countermeasure at bridge abutments is dependent on the accuracy of the placement of the riprap at the site. Riprap is often placed inaccurately due to the inherent difficulties of handling the large riprap stones, especially when they are being placed under water. There are two main methods of placement - end-dumping, e.g., Figure 6.12, where the riprap is tipped off the back of a truck, and individual placement by grab, where each riprap stone is positioned individually, e.g, Figure 6.13. Individual placement is more costly, but results in a more effective riprap blanket. **Individual placement is recommended as a standard practice in New Jersey.**

Figure 6.12. End-dumping of riprap (Smart, 1990).

Figure 6.13. Riprap placement by grab (Smart, 1990).

6.6 LIMITATIONS ON THE USE OF RIPRAP

1. **Monitoring**: Riprap shall be used as a countermeasure only if accompanied by field inspection, that occurs immediately after floods and by the use of monitoring equipment during floods.

2. **Critical velocities**: If a 100-year flood velocity exceeds 11 ft/sec., riprap shall not be used.

3. **Scour Depth**: If calculated scour depth is high and excavation to place riprap under the riverbed would endanger the stability of soil adjacent to the footing, riprap shall not be used.

4. **Economic Considerations**: If riprap is not available locally or at a reasonable distance it may not be economically feasible. In such situations, other alternates may be considered. Also, if cost of hand placement of riprap is high, other less expensive countermeasures may be considered.

5. **Dumped Riprap**: Truck dumped riprap can easily get dislodged during floods and get washed away due to high velocities. It is less stable compared to hand placed riprap and its use is therefore not recommended.

6.7 TRAFFIC ISSUES AND UTILITIES

1. Site access: Adequate access to the site shall be provided for trucks to deliver riprap.

2. Right of Way: Construction easement and right of way may be purchased for the duration of construction.

3. Detours: Detour, lane closure or nighttime work may be necessary. Coordination with Traffic Control would be required. Emergency vehicles and school bus services should not be affected by lane closures.

4. Utilities: Relocation of any utilities at the sides of an abutment or a pier may be necessary for the duration of construction. Coordination with utility companies would be required.

6.8 RIPRAP DETAILING

1. Construction drawings have to be prepared. Conceptual sketches for the layout of riprap with details for riprap placement at abutments and piers based on this handbook, should be used.

2. In addition to hydraulic data, construction drawings shall show tables summarizing flood elevations, flood velocities and scour depths.

3. Maximum side slope is 1V:2H although, where excavation is difficult, 1V:1H may be used with fractured rock.

6.9 COST

Estimated Cost is $150 to $200 per SF for planning riprap at a pier area. Long distance freight charges for riprap may increase the unit cost by 10%.

6.10 CONSTRUCTION PERMITS

Stream encroachment and other applicable permits must be accounted for. Refer to Guidelines that are given for permit applications in the NJDEP Stream Encroachment Technical Manual.

6.11 COMPUTER SOFTWARE

The following software may be used

1. Riprap Design System by West Consultants Inc., San Diego

2. US Army Corps. Of Engineers Riprap Design Software.

6.12 DESIGN EXAMPLES

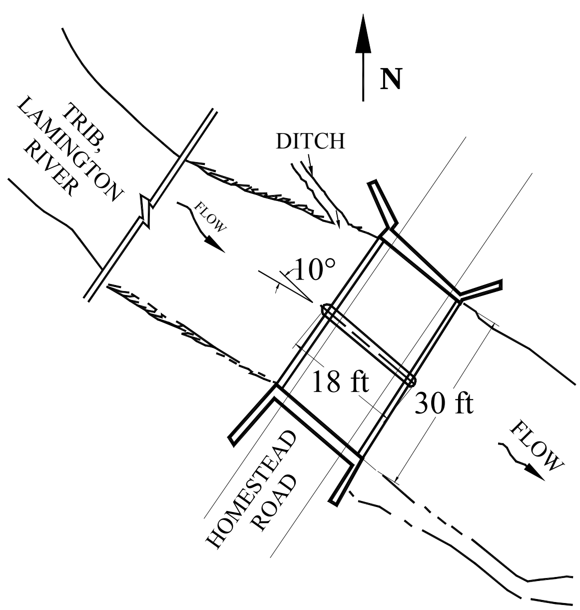
New Jersey bridge number 1000-065 over Tributary to Lamington River has a single round nosed pier and vertical wall abutments. Layout of the bridge site is shown in Figure 6.14. Elevations of pier and abutment are shown in Figure 6.15. Hydraulic, foundation and scour data for the bridge are as follows:

Pier

Design scour depth at pier	= 5 ft
Width of pier footing	= 2 ft
Length of Pier (Width of Bridge)	= 18 ft
Width of pier	= 3'-6"
V_{100}	= 3.67 ft/sec
V_{500}	= 5.08 ft/sec
Specific Gravity of stone, S_s	= 2.24 for 500yr

Abutment

Flow Depth, y_0	= 6.12 ft
V_{100} at abutment	= 4.0 ft/sec
V_{500} at abutment	= 5.54 ft/sec
Specific Gravity of Stone, S_s	= 2.24
Width of abutment footing	=1 ft
Width of abutment	= 8 ft
Design scour depth at abutment	= 5'-6"



PLAN (NOT TO SCALE)

Figure 6.14. Plan

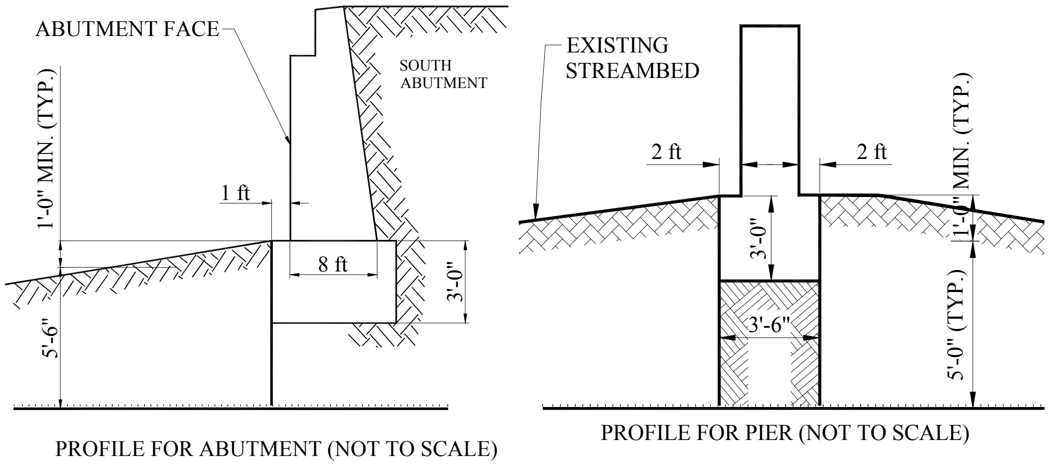


Figure 6.15. Profiles

Riprap Countermeasure Design for Pier

1. Design Velocity

   V = $\alpha \cdot \beta \cdot V_{500} = 1.2 \times 1.0 \times 5.08 = 6.096$ ft/sec
   ($\alpha = 1.2$, $\beta = 1.1$ from Table 6.4)

2. Stone size as per Equation (6-1).

   K=1.5 (For round nosed pier from Table 6.4)
   $$D_{50} = \frac{0.692}{(S_s - 1)} \frac{(KV)^2}{2g} = \frac{0.692}{(2.24-1)}[\frac{(1.5 \times 6.096)^2}{(32.2 \times 2)}] = 0.73' \approx 9''$$
   Minimum D_{50} = 9”
   Place in minimum 3 layers.

   Thickness of riprap mattress = 5 ft
   (Contraction scour depth) > $2D_{50}$ = 18” Hence OKAY
   From NCSA classification in Table 6.2, NCSA stone size = R-5, Use minimum R-6.

3. Extent of Riprap Protection Design = 8’-6”

   Greater of
   As per Figure 6.4, $\Phi = 45^\circ$
   WR = X + 18” + Y COT Φ = 2’ + 18” + 5’ = 8’-6”
   2W = 2(3.5’) = 7’

   Figure 6.16. shows the design of riprap for bridge pier.

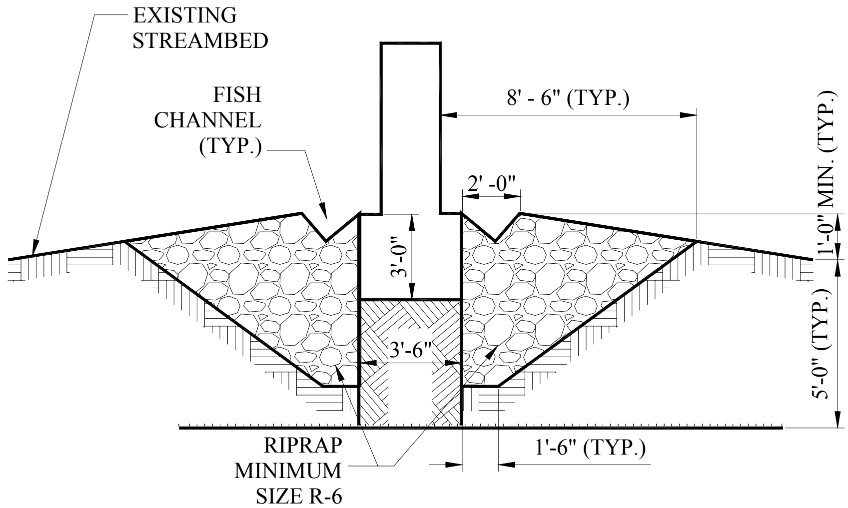


Figure 6.16. Countermeasure details at pier

Riprap Countermeasure Design for Abutment

1. Design Velocity

   V = $\alpha \cdot \beta \cdot V_{500} = 1.0 \times 1.0 \times 5.54 = 5.54$ ft/sec
   ($\alpha = 1.0$, $\beta = 1.0$ from Table 6.6)

2. Stone size as per Equation (6-2).

   FROUDE # = $\dfrac{V}{(gy)^{0.5}} = \dfrac{5.54}{(32.2 \times 6.12)^{0.5}} = 0.39$
   Since 0.39 < 0.8 Select $\gamma = 1.0$ as per Table 6.6
   K=1.2 (For Vertical wall from Table 6.6)
   $$D_{50} = \frac{K}{(S_s - 1)}[\frac{V^2}{gy}]y = \frac{1.02}{(2.24-1)}[\frac{(5.54)^2}{(32.2 \times 6.12)}](6.12) = 0.78' = 9.41''$$
   Minimum D_{50} = 0.78 ft = Approx. 10 inch;

   Thickness of riprap mattress = 5 ft
   (Contraction scour depth) > $2D_{50}$ = 20” Hence OKAY
   From NCSA classification in Table 6.2, NCSA stone size = R-5, Use minimum R-6.

3. Extent of Riprap Protection Design = 8’

   As per Figure 6.4, choose maximum $\Phi = 45^\circ$,
   X + 18” + Y COT Φ = 1’ + 18” + 5’-6” = 8’-0”

Figure 6.17. shows the design of riprap for bridge pier. Place along the face of apron wall at abutments and wingwalls.

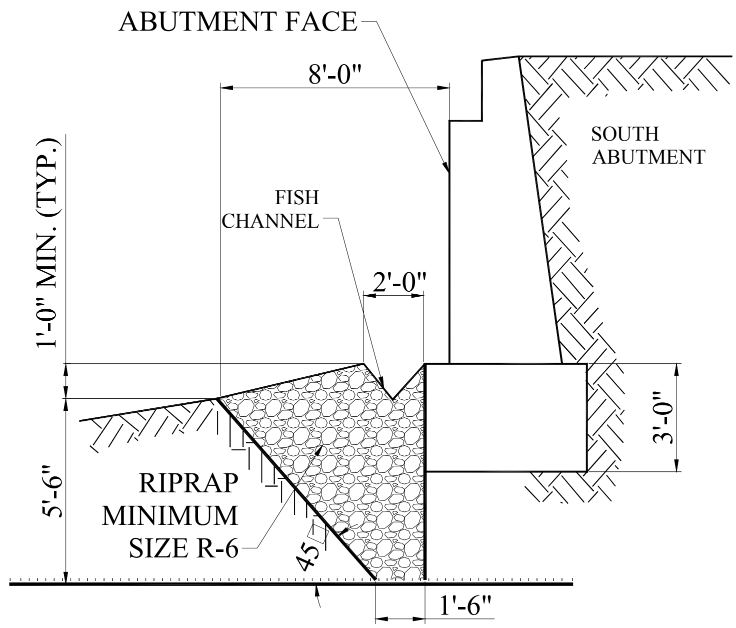


Figure 6.17. Countermeasure details at south abutment

Note: Riprap is not recommended as a permanent countermeasure, but as emergency shielding only for a period of 5 years or longer, after regular evaluation from under water bridge inspection reports.

Riprap can however be used as secondary local armoring, in conjunction with primary structural countermeasures or with river training measures.

CHAPTER 7

GABIONS AS ARMORING COUNTERMEASURE

7.1 DESCRIPTION

Gabions are containers or baskets made up of wire mesh and that are filled with cobbles or coarse gravel. They are filled with stone on site or in a shop.

A thinner version of a gabion is known as a Reno mattress. Whereas gabions typically take the shape of a brick or sausage, Reno mattresses have a short vertical dimension and large lateral extension. The first application of Reno mattresses was to repair a breach of the Reno River in Italy by Officine Maccaferri in 1884.

The wire mesh has flexibility, which allows the containers to deform to the bed profile. Compared to riprap, lesser volumes of excavation of soil, from the riverbed, are required.

Figure 7.1 shows a typical construction scheme and installation of Gabions. Gabions have several advantages when compared with other means of bank protection, such as;

1. Their loose and porous structure reduces their susceptibility to uplift forces,
2. They can be stacked easily in stable configurations,
3. The flexibility of the wire mesh allows gabions to mold themselves so as to restore their stability and provide adaptability to site conditions.
4. Relatively small rock sizes can be used to provide the protection effectiveness of much larger rock units.
5. Due to its smaller thickness, a lesser quantity of stones is required.
6. They have wider applications than riprap; such as:
    a. At abutment and pier footing locations
    b. On steep slopes of river banks, where riprap would be unstable
    c. On upstream or downstream of bridges and culverts, as river training energy dissipaters.

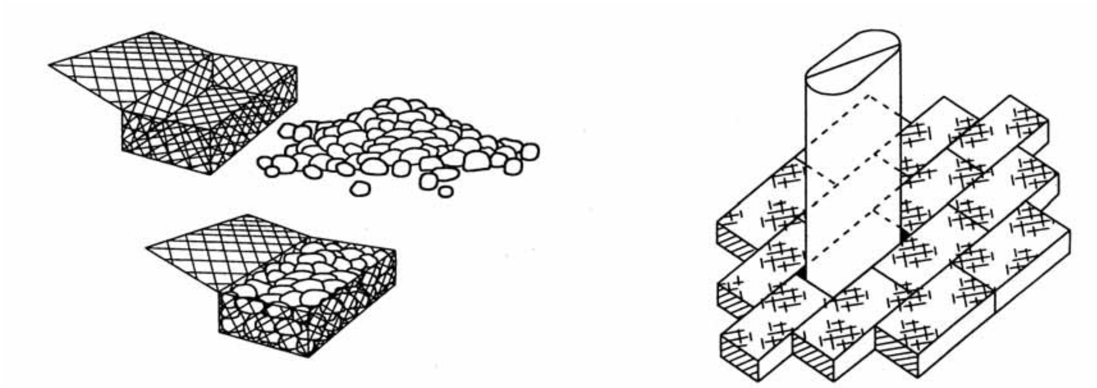


Figure 7.1. Gabions and Reno Mattresses for Bridge Scour Countermeasure.

7.2 TYPES OF GABIONS

The following types of gabions are commonly used as armoring countermeasures:

1. Gabion Sacks

   They are used when construction in "the dry" is not possible. In the absence of cofferdams, gabion sacks are placed directly in water. The minimum dimension of a gabion sack ranges between 18 inches to 3 feet.

2. Gabion Boxes or Baskets

   Gabion boxes are larger in size than the sacks. The minimum dimension of a gabion box ranges between 2 to 4 feet. They are more suitable for higher velocities.

3. Gabion or Reno Mattresses

   They are thinner than sacks or boxes and have less weight per unit area. Minimum thickness varies between 8 inches to 18 inches. The mattress is manufactured in greater lengths than the length of a gabion sack or gabion box. For higher scour depths, two mattresses can be placed on top of each other and tied together. Due to their flexibility, they are more commonly used than gabion sacks or boxes.

4. Wire Enclosed Riprap

   It differs from the Reno Mattress in that it is larger in size and is a continuous framework rather than individual interconnected boxes or baskets. It has been used successfully in New Mexico, Arizona and Colorado. It is used for slope protection at riverbanks and as guide banks. Riprap sizes that are used are less uniform when compared to other three types discussed above. However, wire enclosed riprap may be used in conjunction with gabions placed at the toe of slope. The thickness of wire enclosed riprap varies between 12 to 18 inches. Wire enclosed riprap is generally anchored with steel stakes or spikes to the embankment. Design Guideline 3 of HEC-23 provides detailed guidelines for wire enclosed riprap. Figure 7.2 shows an example of wire-enclosed riprap installed in embankments of the Burlington County Bridge No. 128a near Moorestown.

Figure 7.2. Use of Wire enclosed Riprap at Burlington County Bridge No. 128a.

7.3 FAILURE OF GABIONS

Damage to the wire mesh of gabions, which can result in failure of the entire installation, is a major reliability problem when considering the performance of gabions. The damage to the wire mesh may be because of several processes, such as:

1. Movement of filling rock in highly turbulent flows leading to wire damage, deformation of the basket and ultimately to unsatisfactory performance of the installation, and

2. Long-term corrosion and reduction in strength of the wire mesh leading to installation failure, possibly in combination with the first-mentioned factor.

7.4 DESIGN GUIDELINES

Design guidelines for gabions are based on laboratory experiments conducted by Parker et al (1998) under the NCHRP 24-07 project, CIRIA (2000) and HEC-23. Other recommended sources of information on design of gabions are U.S. Army Corps of Engineers (1991b), Maynord (1995) and Racin (1993).

1. **Flow Parameters**: The design storm shall be the same as that required for riprap. The worst-case conditions in terms of water depth and flow velocity should be established for sizing gabions.

2. The Manning's "n" value used for gabions shall be 0.025.

3. **Sizing of Gabions**: By enclosing the stones within the wire mesh, smaller size stone can be used, as compared to the size of stones used for conventional riprap. Typically, thickness of gabions varies between 1/3rd to 2/3rd of the thickness of riprap.

   Sizing of gabions should be based on technical advice, information and design manuals provided by manufacturers, if available. Following procedures based on CIRIA (2000) and Parker et al (1998) should be used in the absence of any design manual provided by the manufacturer.

   The thickness of gabions should be determined on the basis of critical velocity of flow in Table 7.1 below. Here, "critical" velocity is the velocity at which the mattress reaches the acceptable limit of deformation. The thickness of gabions should be determined from Table 7.1 by considering the design velocity as the critical velocity. This will provide extra factor of safety up to the limiting velocity when gabions fail.

   Table 7.1 provides lower and upper bounds on critical (design) velocity, stone size and limiting velocity for a gabion thickness. Thickness and stone size for gabions should be selected for lower bound on the critical (design) velocity for a conservative design. For example, for a critical velocity of 14 ft/sec, gabion thickness of 1.0 feet and stone size of 3.9 inch should be selected on the basis of 13.8 ft/sec critical velocity in Table 7.1.

Table 7.1: Sizing of Gabions Based on Design Velocity.

Gabion Thickness (ft)	**Stone Size (inch)**	**Critical Velocity (ft/sec)**	**Limiting Velocity (ft/sec)**
0.49-0.56	3.3	≤11.5	13.8
	4.3	13.8	14.8
0.75-0.82	3.3	11.8	18.0
	4.7	14.8	20.0
1.0	3.9	13.8	18.0
	4.9	16.4	21.0
1.64	5.9	19.0	24.9
	7.5	21.0	26.2

Based on Agostoni (1988) and CIRIA (2000)

In sizing gabions using Table 7.1, following additional requirements should be observed:

(Minimum) stone size > 1.25 x Maximum spacing between wires

(Maximum) stone size < 2/3 x Height of basket or box

(Minimum) height of basket > 6 in.

Stone size for gabions can also be calculated by Pilarczyk Equation [CIRIA (2000)]

$$d_{n50} = \frac{\mu}{(1-p)(s-1)} \frac{0.035}{\Psi_{CR}} \frac{K_T K_Y}{K_s} \frac{U^2}{2g} \tag{7-1}$$

where

Ψ_{CR} = Stability Factor = 0.07 for gabions

p = Porosity of stone filling the gabions < 0.4

μ (Stability correction factor)

= 0.75 for continuous protection

= 1.0 to 1.5 at edges and transitions

s = Relative density of stone

K_T (Turbulence Factor)

= 1.0 for normal river turbulence

= 1.5 to 2 for high turbulence at bridge piers

K_Y (Depth Factor) = $\{ dn_{50} / y_0 \}^{0.2}$

y_0 = Local water depth

K_S (Slope Factor) = (k_d k_l)

$$k_d = \cos\varepsilon\sqrt{1-(\tan\varepsilon/\tan\varphi)^2}$$

ε = Angle of bank to the horizontal

φ = Internal angle of friction of the revetment

k_l = sin (φ-χ) / sin φ

χ = Angle of the channel invert to the horizontal

Equation (7.1) needs to be calculated iteratively using the following procedure:

Step 1: Using stone size based on Table 7.1, calculate K_Y

Step 2: Calculate d_{n50} using K_Y in step 1. Go to Step 1 and use calculated value of d_{n50} to repeat the process.

Step 3: Repeat steps 1 and 2 till difference in d_{n50} between two iterations is negligible.

Selected stone size should be the greater of stone sizes obtained from the Table 7.1 and Equation (7-1).

For piers in sand bed streams only (D_{50} < 2 mm): Minimum volume of a gabion basket can be calculated by the following equation recommended by NCHRP 24-7

$$V = \frac{0.069U^6K^6}{\rho_r^3(\rho-1)^3g^3} \tag{7-2}$$

V = Allowable volume of individual unconnected basket in ft^3

U = Approach design flow velocity in ft/sec

K = 1.5 for round nosed piers and 1.7 for square nosed piers

ρ_r = Density of rock in the basket or box

ρ = Density of water

g = 32.2 ft/s^2

Selected volume of gabion basket should be greater than the volume obtained from Equation (7-2).

Special Sizing Notes:

a. When sizing gabions, both lower and upper bounds on stone size should be established using Table 7.1 and Equation (7-1).

b. The smallest stone size to be placed in wire net shall not be less than 3 inches or greater than 12 inches. Actual size may be based on manufacturer's recommendations.

4. **Extent and Layout of Gabions at Bridge Piers:** The recommended extent and layout of gabions shall be 2D from all faces of piers, where D is the width of the pier (see Figure 7.3 and 7.4). The extent of filter shall be smaller than the extent of gabions.

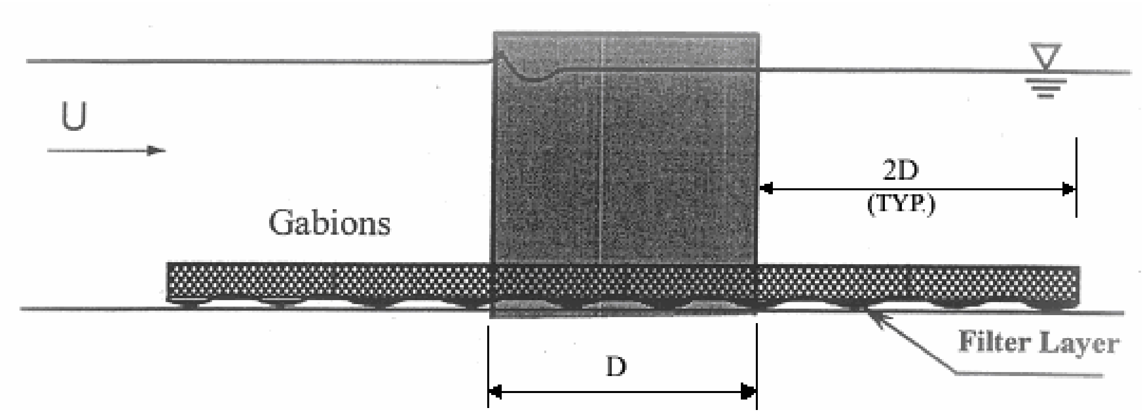


Figure 7.3. Gabion Installation of Bridge Piers.

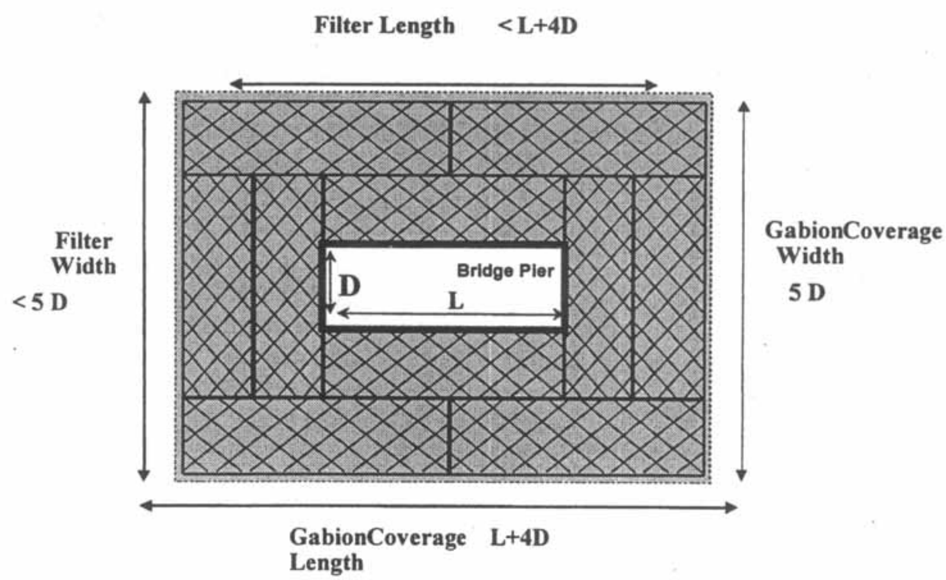


Figure 7.4 Extent of Gabions and Geotextile Filter around Bridge Piers.

5. **Placement of Gabions at Embankments and Abutments:** For vertical wall abutments, the extent of gabions should be 2D from the face of abutment, where D is the width of the abutment. The layout of gabions shall be the same as that for riprap in Figures 6.6 to 6.8. For spill-through abutments, gabions should be installed in the same configuration shown in Figure 6.9 for riprap. The width of riprap from the toe of the abutment should at least be two times the flow depth.

Figures 7.5(a) and 7.5(b) show typical layouts of gabions recommended for embankments and spill-through bridge abutments. For small openings, the gabion nets should extend the full width between the two abutments.

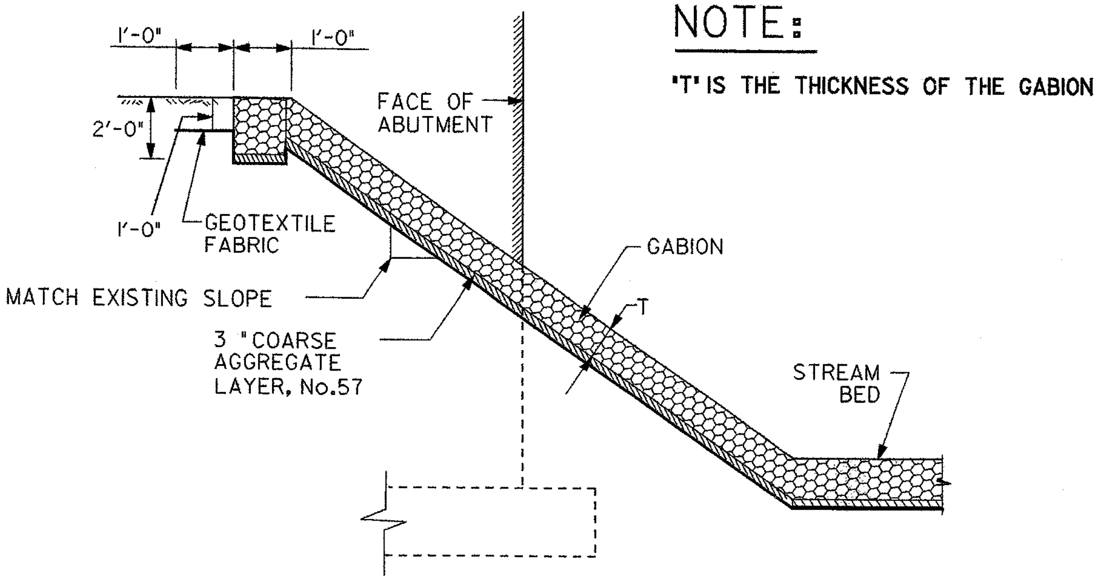


Figure 7.5a. Gabion Detail at Embankment

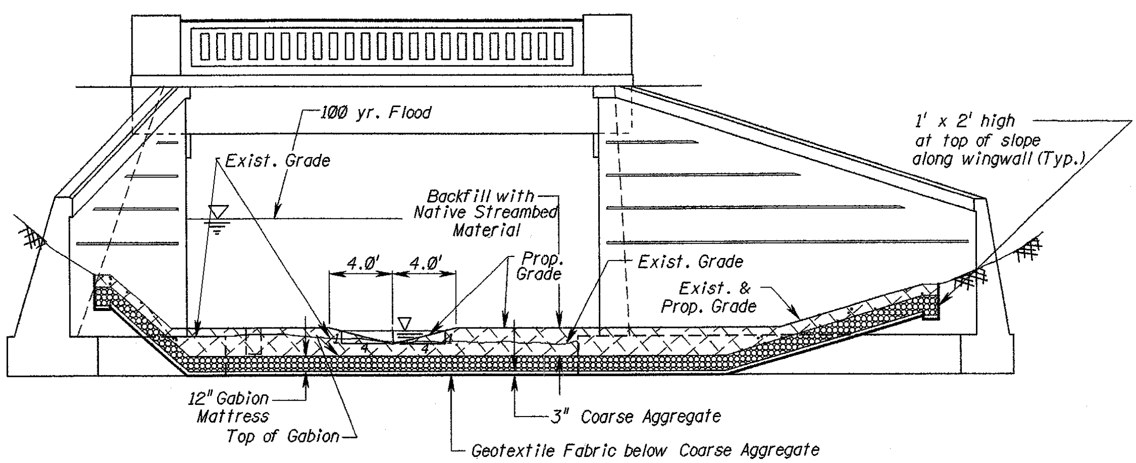


Figure 7.5b. Gabion Detail at Embankment: Upstream Elevation, Looking Downstream.

6. **Scale Factors**: Scale factor for depth in Section 5.3 shall not be applicable for sizing of gabions.

7. **Types of Wire to be Used**

The strength and durability of wire plays an important role in the durability and maintenance of gabions. The basket should be made of single strand galvanized or PVC coated wires. The wire should be formed with a double twist like a fence link, and basket sidewalls should be reinforced with wires of diameter that are larger than the diameter used to make the basket mesh. Specific design details for the wires should be:

Material

- Steel wire 3000 psi ASTM 392 grade - either welded or hand woven. Welded steel wire is rigid but woven wire is flexible and is preferred for slopes.
- Polymer mesh – woven type.

Coating

- Steel wire galvanized with a zinc coating.
- In addition to galvanizing, it should be enclosed in PVC. The PVC coating helps to resist corrosion and increases the life of wire.

Mesh Details

- The Mesh Pattern can be hexagonal, rectangular or V-shaped (See Figure 7.6)
- For rivers with gravel and cobble beds, the abrasion of wires is greater. In such cases, use double layers of mesh or increase the mesh diameter to minimum 3 mm.

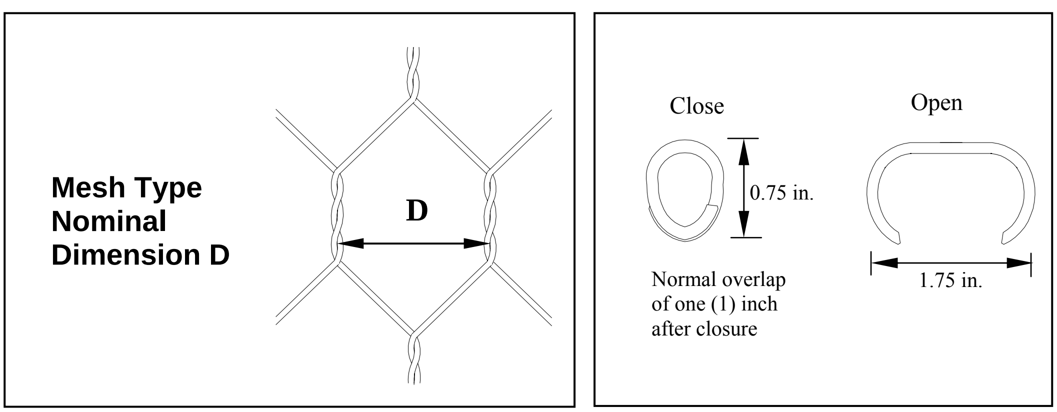


Figure 7.6. Mesh patterns of gabions.

7.5 CONSTRUCTABILITY ISSUES AND CONSTRUCTION EQUIPMENT

Delivery from long distances will be cost prohibitive. The type and size of gabions should be selected from locally available sizes.

In all cases, gabion designs must be based on hydraulic conditions, long-term durability and ease of maintenance.

Excavation machines and small cranes may be used for pre-excavation and for lifting and placing of sacks, boxes and mats in position. The crane can be located on bridge approaches (usually shoulder) or adjacent to riverbed, if access is possible.

7.6 UNDERWATER CONSTRUCTION

Armoring must not reduce the cross sectional area of a channel or protrude above the riverbed to comply with environmental permit requirements.

For rivers with 2 feet depth of water, cofferdams are not required and sand bags may be used.

However, cofferdams would be required for greater depths. Watertight timber or steel sheeting should be driven into riverbed. The excavated soil should be placed on the banks for reuse. After placing the gabions, six inches to one-foot layer of excavated soil should be placed on top and compacted. Temporary sheeting should be withdrawn and any voids filled up.

A layer of grass or thin vegetation may be grown to stabilize the topsoil.

In locations where riverbed has eroded due to recent floods, excavation may not be required and gabions may be deposited directly under water by a barge. This is more economical since cofferdam driving costs are higher than the cost of gabions.

7.7 USE OF FILTER LAYERS

On sand river bed, a geotextile filter should be placed underneath the gabions to prevent sand leaching.

The geotextile filter should be sealed to the pier by a flexible tube containing a cable that can be tightened around the pier.

For more details on filters, please refer to the section on granular filters in Chapter 11.

7.8 DESIGN TOOLS

1. Approved commercial software may be used.

2. Construction drawings must be based on detailed designs.

3. A design spreadsheet may be used if approved by NJDOT.

7.9 COST

An estimated Cost of $ 85 to $ 95 per SF of pier area may be used.

7.10 CONSTRUCTION PERMITS

Stream encroachment and other applicable permits will be required. Refer to Guidelines given for permit application in NJDEP Stream Encroachment Technical Manual.

7.11 DURABILITY AND MAINTENANCE

The following types of failures may occur and may be avoided by good construction practice:

1. Failure of meshes and stones fallout due to corrosion, abrasion and damage during construction.

2. Winnowing failure due to erosion of underlying bed material through the gabions due to failure of filter layers and inadequate gabion thickness during floods.

3. Excessive movement of stone within the baskets may occur at high currents due to poor packing.

7.12 DESIGN EXAMPLE

New Jersey bridge number 1000-065 over Tributary to Lamington River has a single round nosed pier and vertical wall abutments. Layout of the bridge site is shown in Figure 7.7. Elevations of pier and abutment are shown in Figure 7.8. Hydraulic, foundation and scour data for the bridge are as follows:

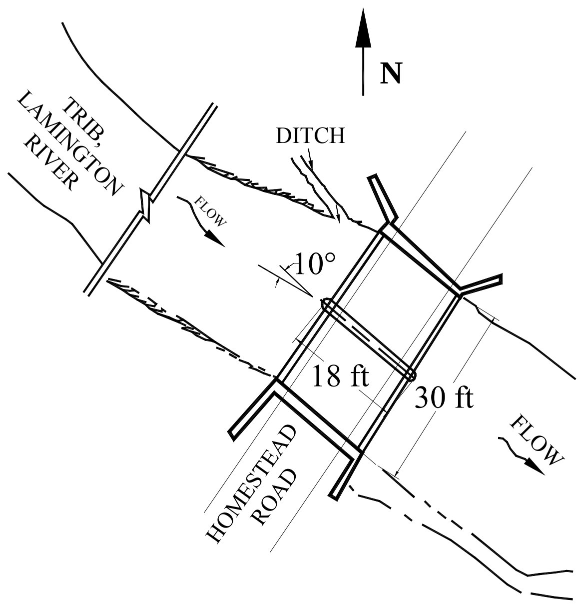
Pier

Design scour depth at pier	= 5 ft
Width of pier footing	= 2 ft
Length of Pier (Width of Bridge)	= 18 ft
Width of pier	= 3'-6"
V_{100}	= 3.67 ft/sec
V_{500}	= 5.08 ft/sec
Specific Gravity of stone, S_s	= 2.24 for 500yr

Abutment

Flow Depth, y_0	= 6.12 ft

V_{100} at abutment = 4.0 ft/sec
V_{500} at abutment = 5.54 ft/sec
Specific Gravity of Stone, S_s = 2.24
Width of abutment footing =1 ft
Width of abutment = 8 ft
Design scour depth at abutment = 5'-6"



PLAN (NOT TO SCALE)

Figure 7.7: Site Layout of the New Jersey Bridge 1000-065

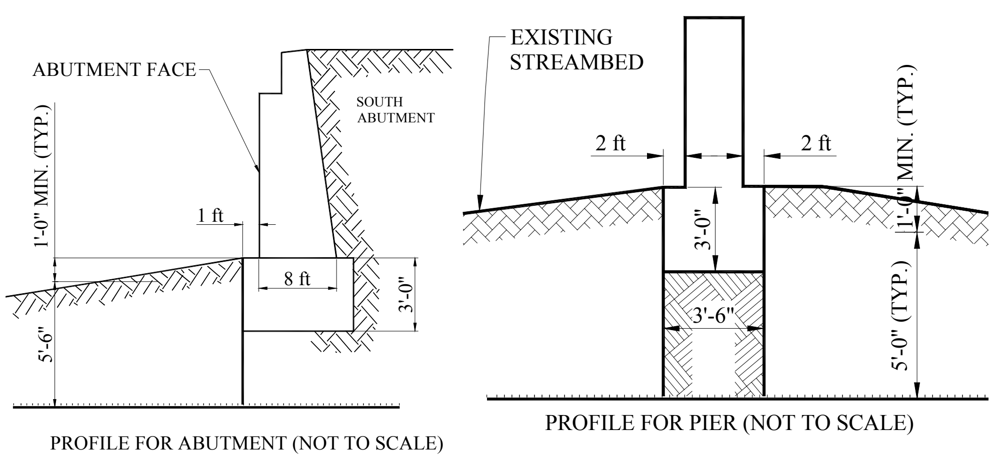


Figure 7.8: Elevations of Pier and Abutments.

Design of Gabions for Piers:

Critical Velocity = Average Velocity for 500 years, V_{500} = 5.08 ft/sec
As per Table 7.1, for critical velocity < 11 ft/sec,

Thickness of Gabion	**= 0.5 ft**
Maximum Stone Size	**= 4.3 inch**
Minimum Stone Size	**= 3.3 inch**

Stone size as per Equation (7-1) = 4.0 inch

Using the following parameters in Equation (7-1)

Ψ_{CR} = 0.07
p = 0.2 < 0.4
μ = 0.75 for continuous protection
s = 2.24
K_T = 1.0 for normal river turbulence
K_Y = $(dn_{50} / y_0)^{0.2}$
y_0 = 6.12 ft
K_S = $(k_d\, k_l)$, $k_d = \cos\varepsilon\sqrt{1-(\tan\varepsilon/\tan\varphi)^2}$

ε = Angle of bank to the horizontal = 1V:2.5H = 21.8°
φ = Internal angle of friction of the revetment = 46.5°
k_l (a longitudinal slope term) = sin (φ-χ) / sin φ
χ = Angle of the channel invert to the horizontal = 2.23°

and assuming an initial value of d_{n50}= 3.3 inch, d_{n50} = 4.04 inch is obtained after several iterations.

Recommended stone sizes = 4.0 to 4.3 inch
Minimum Volume of Gabions for Piers (Equation 7-2) = 0.20 ft^3

Calculated by assuming following values in Equation (7-2):

U = Approach design flow velocity in ft/sec = 5.03 ft/s
K = 1.5 for round nosed piers
ρ_r = Density of rock in the basket or box = 140 lb/ft^3
ρ = Density of water = 62.4 lb/cft
g = 32.2 ft/s^2

Minimum height of basket = 6 inch

Height of basket $> \frac{3}{2}$ **maximum stone size**

> 1.5 x 4.3 inch = 5.7 inch

Spacing between wires $< \frac{\text{Min. Stone Size}}{1.25} = \frac{3.3}{1.25}$ **= 2.64 inch**

Extent of gabions:

Gabion coverage along the direction of flow = L+4D

= 18' + (4)(3'-6") = 32'

Gabion Coverage along the longitudinal direction of the bridge = 5D

= (5)(3'-6") = 17'-6"

Filter requirements:

Use geotextile filter with extent smaller than gabions.
Stone fill filter fabric to be as per manufacturer's requirements.

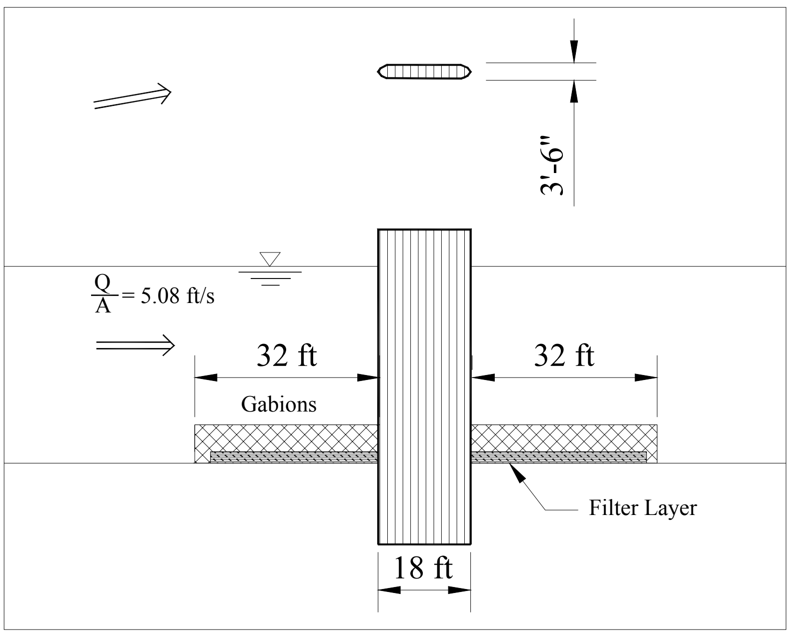


Figure 7.6: Design details of gabions around bridge piers.

Design of Gabions for Abutments:

Critical Velocity = Average Velocity for 500 years, V_{500} = 5.54 ft/sec
As per Table 7.1, for critical velocity < 11 ft/sec,

Thickness of Gabion	**= 0.5 ft**
Maximum Stone Size	**= 4.3 inch**
Minimum Stone Size	**= 3.3 inch**

Stone size as per Equation (7-1) **= 5.0 inch**
(follow the procedure for piers)

Recommended stone sizes **= 4.3 inch to 5.0 inch**

Minimum height of basket **= 6 inch**

Height of basket **> $\frac{3}{2}$ maximum stone size**

> 1.5 x 5.0 inch = 7.5 inch

Spacing between wires **<** $\frac{\text{Min. Stone Size}}{1.25} = \frac{4.3}{1.25}$ **= 3.44 inch**

Extent of gabions: 16 ft from the face of abutments

The extent of gabions for abutments will be the same as that for riprap in Section 6.5. As per section 6.5, extent of gabions for vertical wall abutments should be the greater of

2W = 2 x 8 = 16 ft

X+18"+y cotϕ = 1+1.5+5.5 cot (46.5°) = 7.72 ft

Since gabions extend 32 ft from the face of the pier and 16 ft from the face of the abutment, the full width of the bridge opening should be covered by gabions, since the total length of the bridge is 30 feet.

CHAPTER 8

ARTICULATED CONCRETE BLOCKS

8.1 DESCRIPTION

Articulated concrete blocks (ACBs) consist of preformed units, which either interlock or are held together by steel rods or cables or abut together to form a continuous blanket or mat. ACB's comprise of a single layer of cellular blocks interlocking with adjacent blocks, with about 25% plan area open.

Interlocking provides strength and can be increased by filling with gravel. ACBs are laid on a geotextile or granular filter. They are also known as "artificial riprap" and are used as an alternate for rock riprap, especially when there is a short supply of rock riprap or large rock sizes are required to resist extreme hydraulic forces. These units have greater stability compared to riprap due to their interlocking characteristics. ACBs can be factory manufactured precast concrete units and are commercially produced by proprietary firms. They have been used for erosion control in recent years mainly as revetments but less frequently as countermeasures for bridge substructures. Figure 8.1 shows examples of interlocking and cable-tied block systems.

Cable Tied Blocks or Cable Tied Mattresses are the most popular forms of ACBs. They are concrete blocks tied together by steel cables to form a mat, as shown in Figure 8.2. The cables are either galvanized steel or polyester. The network of manageable blocks tied together has the capability of resisting mobilizing forces of a severe flood to provide protection against erosion.

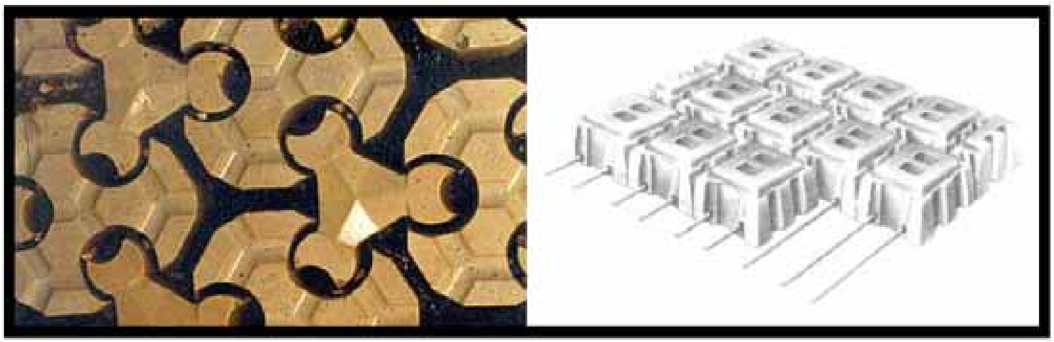
Cable-tied blocks have been used successfully by the US Army Corps of Engineers when the streams are coarsely bedded and capable of moving large stones. Their advantages include ability to withstand strong currents and resistance to ice. The block size is designed based on flood velocity, slope of riverbed and side slopes. Spacing between cable-tied block units should be enough to allow sufficient flexibility to the mattress.

Cable-tied mats are placed by machinery, and can be placed under shallow depths of water. However, they are less adaptable for placing in small or confined areas.

8.2 ADVANTAGES OF ARTICULATED CONCRETE BLOCKS

1. Flexibility,
2. Ability to withstand strong currents,
3. A Pre-attached geotextile,
4. Resistance to ice

Cost-competitiveness. An example of installation of cable-tied blocks can be found in Neill and Morris (1980).



(a) (b)

Figure 8.1. Examples of Interlocking and Cable -Tied Articulated Concrete Blocks (from HEC-23).

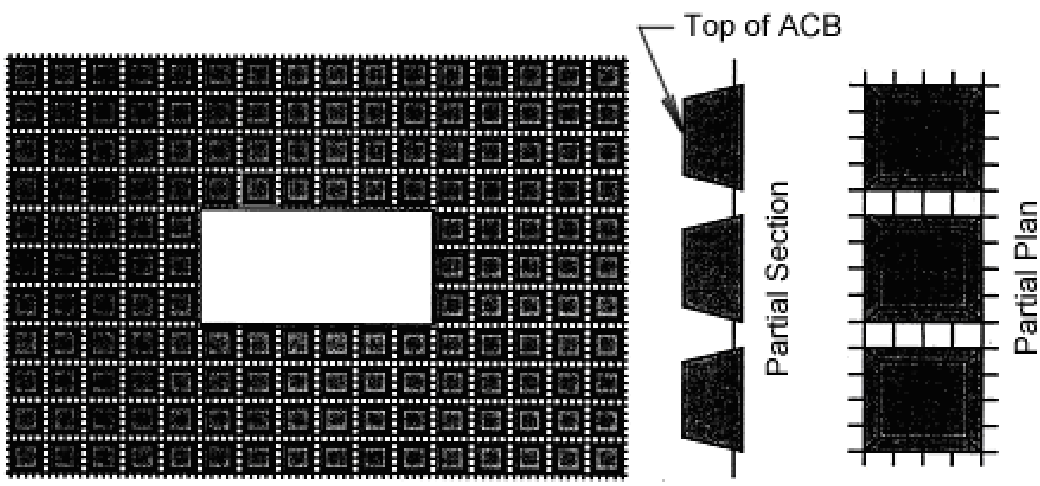


Figure 8.2. Cable-tied blocks for bridge scour countermeasure.

8.3 FAILURE MODES OF CABLE-TIED BLOCKS

Two failure modes of cable-tied blocks that are similar to those for grout filled mats are the following [McCorquodale (1993)],

1. Overturning and rolling-up of the leading edge, which can occur in the absence of sufficient anchoring or toeing in.

2. Uplifting of the inner mat at much higher flow velocities when the leading edge is anchored. The seal between the face of the pier, the countermeasure and geotextile plays a significant role in this mode of failure.

8.4 DESIGN GUIDELINES FOR CABLE-TIED BLOCKS

Design guidelines for Articulated Concrete Blocks (ACBs) for abutments are based on Design Guidelines 4 in HEC-23. Design guidelines for Articulated Concrete Blocks (ACBs) for piers are based on NCHRP 24-07. Other recommended sources of information on design of ACBs are HEC-11 and McCorquodale (1993).

1. **Sizing at Bridge Abutments**

   Design Guideline 4 of HEC-23 presents a factor of safety approach for the design of ACB's for revetment or bed armor. This approach is developed by considering the stability of a single concrete block on a sloping surface and is directly applicable for the design of ACB's for bridge abutments.

   The failure of the countermeasure is defined as finite movement of a block and hence doesn't include the effects of resistive forces due to cables, rods or interlocking for conservatism.

   For sizing of ACBs for bridge abutments, design charts developed by the ACB manufacturer should be used. These charts relate the allowable shear stress or velocity to the channel bed slope for a given factor of safety. Figure 8.3 and 8.4 shows sample design charts in Design Guideline 4 of HEC-23. The design chart in Figure 8.3 represents the stability of ACBs placed flat on the channel bed neglecting the influence of abutment slope. Chart in Figure 8.4 accounts for the effect of abutment slope in the factor of safety. The factor of safety can then be computed by

$$SF = \frac{\tau_a}{\tau_0}(SF_a)K_1 \text{ or } SF = \frac{V_a}{V_0}(SF_a)K_1 \tag{8-1}$$

   where

   τ_a and V_a = Allowable shear stress and velocity for the factor of safety for which the chart was developed.

   τ_0 and V_0 = Design shear stress and velocity

   K1 = side slope correction factor.

   SF_a = Factor of Safety in developing charts.

Notes:

Charts in Figures 8.3 and 8.4 are for illustration purposes only and should not be used for design. Instead, similar charts developed by the ACB manufacturer based on ACB properties and hydraulic tests should be used.

A higher factor of safety (approximately by a multiple of 2) should be used If the effects of projecting blocks is not considered in the development of design charts.

Alternate guideline based on block properties and hydraulic tests provided by the ACB manufacturer may be used. The manufacturer must provide sufficient documentation for such design guidelines.

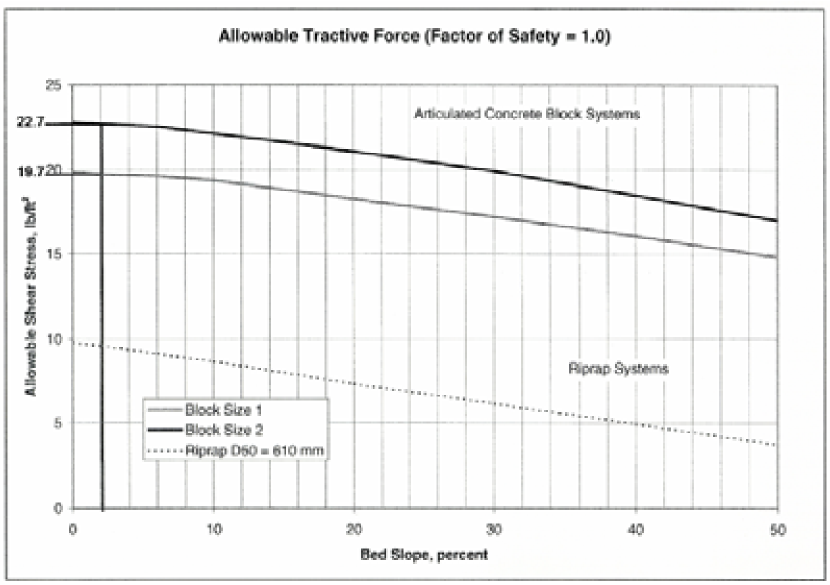


Figure 8.3 Chart for Allowable Shear Stress vs. Bed Slope (From HEC-23, **Chart for Illustration only**, use similar chart provided by the manufacturer for the design).

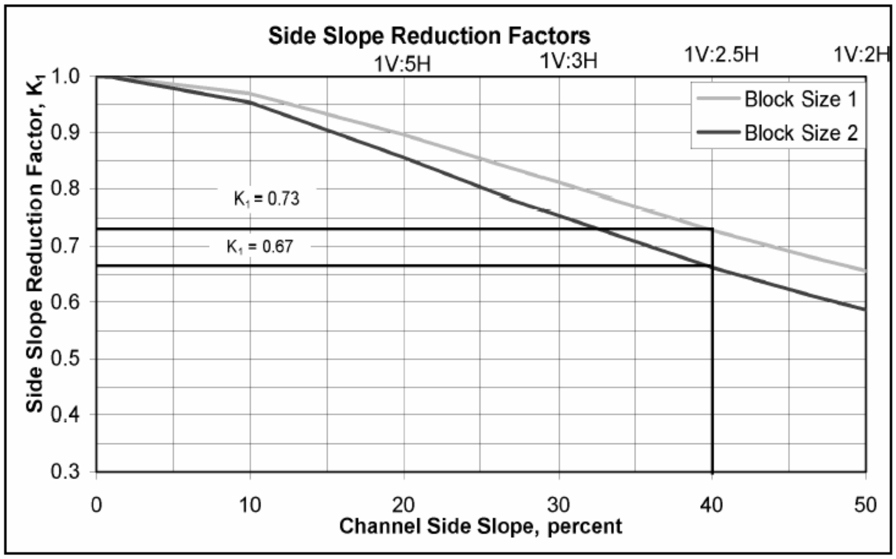


Figure 8.4 Chart for Side Slope Correction Factor (From HEC-23, **chart for illustration only**, use similar chart provided by the manufacturer for the design).

For cases when design charts provided by the manufacturer are not available, Equation (8-2) based on factor of safety approach in HEC-23 can be used to calculate the factor of safety as

$$SF = \frac{\cos\left(\frac{l_2}{l_1}\right)}{\eta'\left(\frac{l_2}{l_1}\right) + \sin\theta\cos\beta + \frac{l_3 F_D' \cos\delta + l_4 F_L'}{l_1 W_A}} \tag{8-2}$$

where

l_1 and l_2 =	Moment arms of the weight of the block for side slope and longitudinal slope as shown in Figure 8.5
l_3 and l_4 =	Moment arms of the lift and drag forces on the block, as shown in Figure 8.5.
W_A =	Submerged weight of the block
F_D' =	Drag Force due to vertical projection of blocks
F_L' =	Lift Force due to vertical projection of blocks
θ =	Side slope angle with respect to the horizontal plane
δ =	90-β–λ
λ =	Angle between horizontal plane and velocity vector = Bed slope when flow id parallel to the bed

$\beta =$ Angle between the block movement direction and vertical plane

$$= \tan^{-1}\left[\frac{\cos\lambda}{\left(\frac{\frac{M}{N}+1}{\eta}\right)\left(\frac{l_1}{l_2}\right)\sin\theta+\sin\lambda}\right]$$

$$\eta' = \left\{\frac{\frac{M}{N}+\sin(\lambda+\beta)}{\frac{M}{N}+1}\right\}\eta; \quad \frac{M}{N}=\frac{l_4F_L}{l_2F_D}$$

$$\eta = \frac{\tau_0}{\tau_c}$$

τ_0 = Shear stress on the channel boundaries

τ_c = Critical shear stress when failure occurs

$F_D =$ Drag Force on the block

$F_L =$ Lift Force on the block

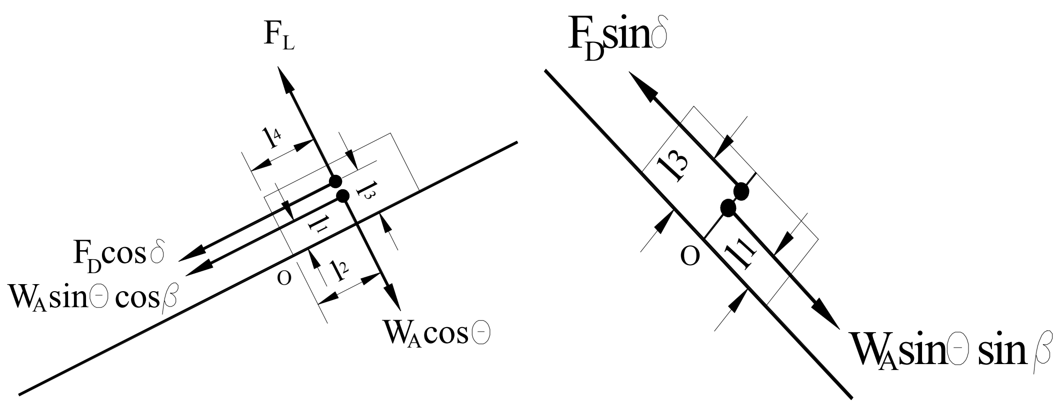


Figure 8.5 Definitions of Moment-Arms in Equation (8-2).

Cable-tied blocks should be designed by Equation (8-2) with much higher factor of safety than that using the Chart in Figures 8.3 and 8.4 to account for uncertainty in hydraulic characteristics of blocks.

2. **Layout at Bridge Abutments**

Layout of ACBs around bridge abutments should be the same as that for riprap in Chapter 6.

3. **Sizing at Bridge Piers**

According to NCHRP 24-7 [Parker et al (1998)], the weight per unit area of mattress, ζ, can be obtained as

$$\zeta = a_{cb} \frac{\rho_{cb}}{\rho_{cb} - \rho} \rho V^2 \tag{8-3}$$

where:

a_{cb} = 0.20
ρ = density of water = 62.166 lb/ft^3
ρ_{cb} = density of block material, and
V = flow velocity.

The height of the blocks H_{cb} and the volume fraction pore space p (typically 25% to 35%) in the mattress are related to ζ by the relation

$$\zeta = \rho_{cb} g H_{cb} (1 - p) \tag{8-4}$$

It is recommended that the spacing between cable-tied block units be enough to allow the mattress a sufficient degree of flexibility.

4. **Extent and Layout at Piers**

Prior excavation is not needed unless the block height H_{cb} exceeds 0.25 y_0, where y_0 denotes the flow depth under design conditions.

The mattress cover should be 4D, where D is the pier width. Hence, the mattress should extend outward at least a distance 1.5D from every face of the pier [See Figure 8.6].

In the event that the angle of attack β exceeds 15°, the cover is taken at least 4D/cos(β) [See Figure 8.6]

The mattress and filter should be fastened and sealed to the pier as per requirements in Chapter 11.

5. **Cable Material at Piers and Abutments**

The cable connecting the blocks should be sufficiently flexible so as to allow the mattress to deform, but sufficiently durable to last at least 20-years in fast-water river environment. Stainless Steel cables are recommended for harsh environment.

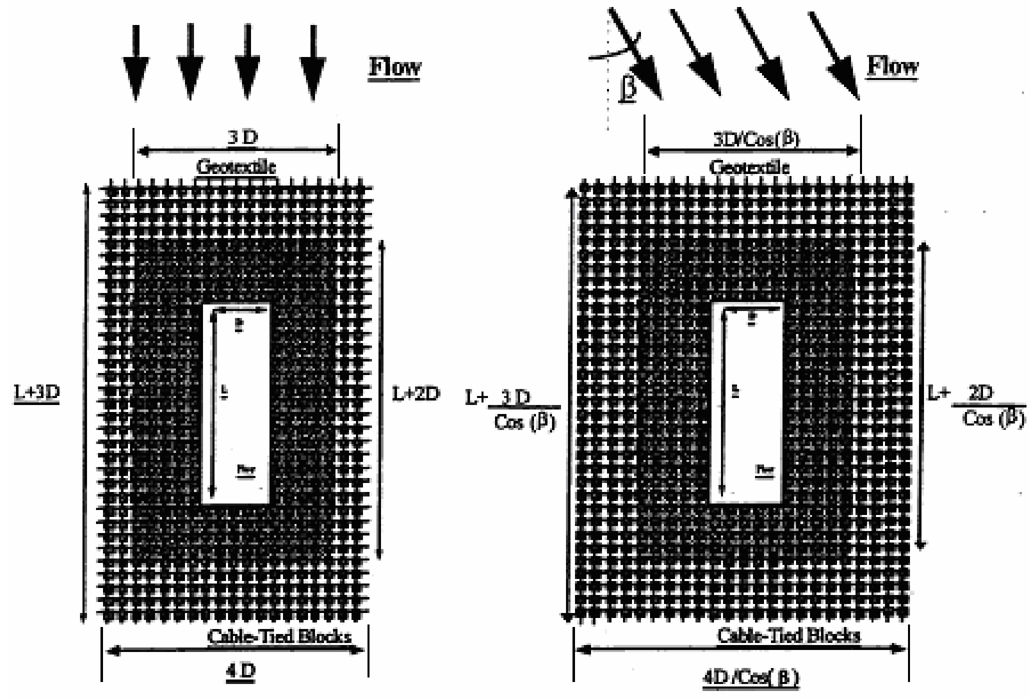


Figure 8.6 Installation and Layout of Cable-Tied Blocks Around Bridge Pier.

6. **Geotextile Filter at Piers and Abutments**

Geotextile filter should not be used for gravel bed rivers.

Fasten the geotextile filter firmly under the base of block mattresses in case of sand bed rivers.

Geotextile should extend outwards 1D from every face of piers and should never extend as far out as the mattress.

7. **Pier Sealing Requirement**

Performance of cable-tied blocks depends on sealing between pier and the block layer. Structural stability of piers should be investigated when attaching the mat to the piers. Cable-tied blocks should be sealed to the pier as per one of the approaches below, depending on site conditions [Design Guideline 4 in HEC-23]:

a. Anchors: Use of tension anchors in addition to grout around the pier seal can provide additional support for the mat. Grout at the pier seal will reduce scouring underneath the mat. Following types of anchors, based on field applications by MnDOT for a pier at TH 32 over Clearwater River at Red Lake Falls, are recommended:

   Duckbill anchors, 0.9 - 1.2 m (3 - 4 ft) deep. Use Duckbill anchors at corners and about every 2.4 m (8 ft) around pier footings.

b. 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 of, or both, to provide a seal between mat and pier [Based on MnDOT Field Application].

c. Pier seal can be provided by placing grout bags on top of the mat at the pier location to provide the necessary seal, as shown in Figure 8.7 [Based on MDOT field application].

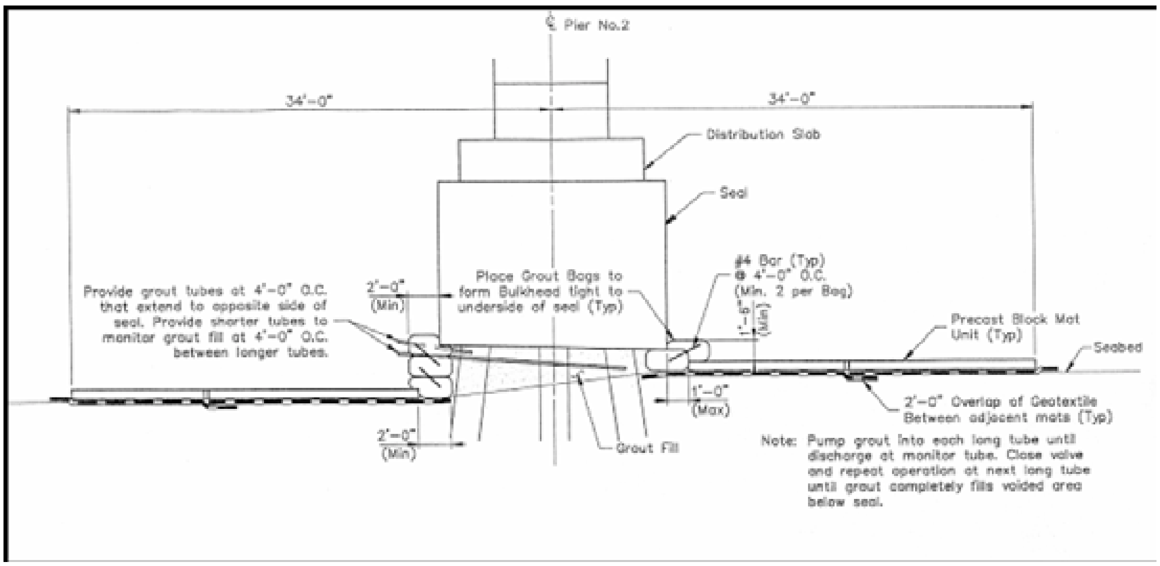


Figure 8.7 Design Plans and Pier Seal Used by MDOT.

d. Pier seal can be provided by placing granular filter material to a depth of about 3 ft below the streambed for about 16 ft around the pier. The geotextile filter and block mat placed on the streambed should overlap this granular filter layer and the remaining gap between the mat and the pier should be filled with riprap, as shown in Figure 8.8 [Based on field applications in Netherlands].

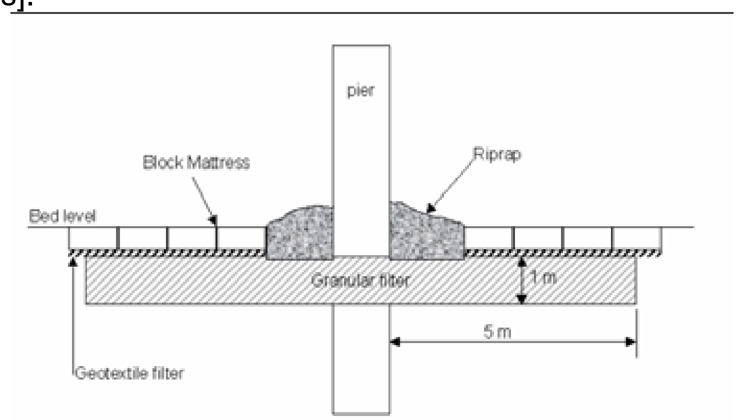


Figure 8.8 Use of granular filter and riprap to seal the joint between a bridge pier and ACB Mat.

8. **Maintenance and Durability**

Cable durability is critical. Corrosive activity (salinity and/or acidity) of water is an important factor in the durability. Concrete durability should be considered, although it is less critical than cable durability.

9. **Limitations of ACBs**

Steel cables get corroded.

The blocks need to be anchored to the substructure, which may transfer tensile stresses in foundations.

The salinity in water is likely to corrode steel cables and contaminate the water. Cable tied blocks are therefore not recommended for rivers located close to the New Jersey coast lines.

They are not suitable for pile bents or complex pile shapes.

They are not suitable for rivers with large cobbles or rocks.

8.5 DETAILED DESIGN

Approved commercial software may be used.

Construction drawings must be based on detailed designs.

A spreadsheet may be developed and used in lieu of software.

8.6 COSTS

Typically laid-in-place wet placement cost is approximately $\$15$-$16/ft^2$.

The cost of seal construction is approximately $2000 for a typical pier.

Cost to place cable-tied blocks around a 4 ft x 20 ft rectangular pier is approximately $9,000.

8.7 CONSTRUCTABILITY ISSUES

Construction observation/inspection to ensure that blocks are installed within the design tolerance is essential to the successful performance of ACBs.

Vertical projection of blocks has detrimental effect on the performance. ACBs must be installed with design tolerance provided by the ACB manufacturer.

Pre-excavation of upstream edge of the mattress is required.

On gravel streams, edges must be anchored (pre-excavation).

Divers may be required to tie the mattress together.

Site access for construction, cranes and equipment needed.

To allow for mattress to settle properly, fabric must be cut away from blocks along the outer edge of the mat.

A granular filter around the pier should be used to provide a seal at the pier.

No vertical discontinuity is allowed at junctions.

8.8 DESIGN EXAMPLE

The New Jersey bridge number 1000-065 has a bed slope of 0.039 ft/ft and a side slope of 1V: 2.5H. Two types of cable-tied blocks are considered for scour protection of the bridge. Hydraulic data for the two types of blocks are given below:

Block Size 1	Block Size 2
n = 0.032	n = 0.026
Maximum Depth =2.02 ft	Maximum Depth = 1.80 ft
Average Velocity = 5.10 ft/s	Average Velocity = 5.57 ft/s
Bed Shear, $\tau_0 = 4.9 \quad lb/ft^2$	Bed Shear, $\tau_0 = 4.4 \quad lb/ft^2$

Charts in Figures 8.3 and 8.4 are considered to be supplied by the manufacturers for the design of blocks.

Design of Cable-Tied Blocks for Abutments using Charts:

From Figure 8.3, allowable shear stress for cable-tied blocks on a bed slope of 3.9% with a factor of safety of one (i.e., SF_a = 1) is:

τ_a = 19.7 lb/ft^2 (allowable shear stress for Block Size 1)

τ_a = 22.7 lb/ft^2 (allowable shear stress for Block Size 2)

From Figure 8.4, reduction factor for a 1V: 2.5H (40%) side slope of the abutment is:

K_1 = 0.73 (for Block Size 1)

K_1 = 0.67 (for Block Size 2)

The factor of safety for the two blocks can be calculated from Equation (8-1) as:

$$SF = \frac{\tau_a}{\tau_o}(SF_a)K_1 = \frac{19.7}{4.9}(1)0.73 = 2.9 \qquad \text{(for Block Size 1)}$$

$$SF = \frac{\tau_a}{\tau_o}(SF_a)K_1 = \frac{22.7}{4.4}(1)0.67 = 3.5 \qquad \text{(for Block Size 2)}$$

Since the factor of safety for Block Size 2 is greater than that for Block Size 1, Block Size 2 is recommended for scour protection of bridge abutments.

Design of Cable-Tied Blocks for Abutments using Equation (8-2):

In addition to the hydraulic data given above, following block characteristics are provided:

Block Size	Submerged Weight (lb)	l_1 (in.)	l_2 (in.)	l_3 (in.)	l_4 (in.)	ΔZ (in.)	ω (in.)	τ_c^* (lb/ft^2)
1	28.6	3	8.8	4.8	8.8	0.5	13	20.0
2	33.3	3	8.8	4.8	8.8	0.5	13	23.0

τ_c^* has been determined from testing.

Factor of safety parameters:

Side slope angle: $\theta = \tan^{-1}(\frac{V}{H}) = \tan^{-1}(\frac{1}{2.5}) = 21.8^\circ$

Bed Slope angle: $\lambda = \tan^{-1}(\frac{S}{l}) = \tan^{-1}(\frac{0.039}{1}) = 2.23^\circ$

Stability Number for Block Size 1: $\eta = \frac{\tau_0}{\tau_C} = \frac{4.9}{20.0} = 0.245$

Stability Number for Block Size 2: $\eta = \frac{\tau_0}{\tau_C} = \frac{4.4}{23.0} = 0.191$

Conservatively assuming that $F_L = F_D$,

$$\frac{M}{N} = \frac{l_4 F_L}{l_3 F_D} = \frac{8.8}{4.8} = 1.83$$

$$\beta = \tan^{-1}\frac{\cos(2.23)}{(\frac{1.83+1}{\eta})(\frac{3.0}{8.8})\sin(21.8) + \sin(2.23)}$$

For Block Size 1: $\beta = 33.65^\circ$

For Block Size 1: $\beta = 27.56^\circ$

For Block Size 1: $\delta = 54.12^\circ$

For Block Size 1: $\delta = 60.21^\circ$

Stability Number of Side Slope for Block Size 1: $\eta' = 0.209$
Stability Number of Side Slope for Block Size 1: $\eta' = 0.157$

It is assumed that an installation specification tolerance of 0.5 inches in the vertical direction will be maintained (blocks are assumed to protrude 0.5 inches vertically)

Drag force due to vertical Projection:

$$F'_D = 0.5(0.0417(1.083)(1.94)(V)^2) = 0.044V^2$$

Design Velocity for Block Size 1	5.10 ft/s
Design Velocity for Block Size 2	5.57 ft/s

Drag Force, F'_D for Block Size 1	$F'_D = 0.044(5.10)^2 = 1.14$
Drag Force, F'_D for Block Size 2	$F'_D = 0.044(5.57)^2 = 1.37$

Now assuming that the additional lift due to the vertical projection is equal to the additional drag, i.e., $F'_D = F'_L$:

Factor of Safety for Block Size 1:

$$SF = \frac{\cos(21.8^\circ)(\frac{8.8}{3.0})}{0.209(\frac{8.8}{3.0}) + \sin(21.8^\circ)\cos(33.65^\circ) + \frac{4.8(1.14)\cos(54.12^\circ) + 8.8(1.14)}{3.0(28.6)}} = 2.53$$

Factor of Safety for Block Size 2:

$$SF = \frac{\cos(21.8^\circ)(\frac{8.8}{3.0})}{0.157(\frac{8.8}{3.0}) + \sin(21.8^\circ)\cos(27.56^\circ) + \frac{4.8(1.37)\cos(60.21^\circ) + 8.8(1.37)}{3.0(33.3)}} = 2.89$$

Block Size 2 has higher factor of safety and is recommended for abutment scour countermeasure.

Layout and Extent of Cable-Tied Blocks for Abutments: The layout and extent of cable-tied blocks is the same as that for riprap in Section 6.5.

Design of Cable-Tied Blocks for Piers:

Sizing of cable-tied blocks for bridge piers should be based on proprietary information provided by manufacturers. In general, manufacturers also provide design guidelines and software support for the design of cable-tied blocks.

These resources should be used in consultation with manufacturers and NJDOT engineers.

For preliminary sizing of cable-tied blocks for bridge piers, Equations (8-3) and (8-4) can be used. For example, for concrete cable-tied blocks with ρ_{cb} = 115 lb/ft^3, weight per unit area of the mattress for design velocity, U = 5.10 ft/sec can be obtained as:

$$\zeta = 0.20 \frac{115}{115 - 62.166} 62.166\,(5.10)^2 = 703.89\,\text{lb/ft}^2$$

The height of the block, H_{cb}, with pore space p = 0.35 (35% pore space) can be obtained from Equation (8-4) as:

$$H_{cb} = \frac{\zeta}{\rho_{cb} g\,(1 - p)} = \frac{703.89}{115 \times 32.2(1 - 0.35)} = 0.29\ \text{feet}$$

Hence, block size with minimum 3.5 inch height should be used.

Extent of cable-tied blocks around piers should be based on layout in Figure 8.5:

Extent of filter layer perpendicular to the flow:	3D	= 10.5 ft	
Extent of filter layer along the flow:	L+2D	= 25.0 ft	[L=18 ft]
Extent of ACB layer perpendicular to the flow:	4D	= 14.0 ft	
Extent of filter layer along the flow:	L+3D	= 28.5 ft	

The layout and extent of cable-tied block is shown in Figure 8.9 below. Cable-tied block mat should be sealed to piers as per Item 7 of Section 8.4 above.

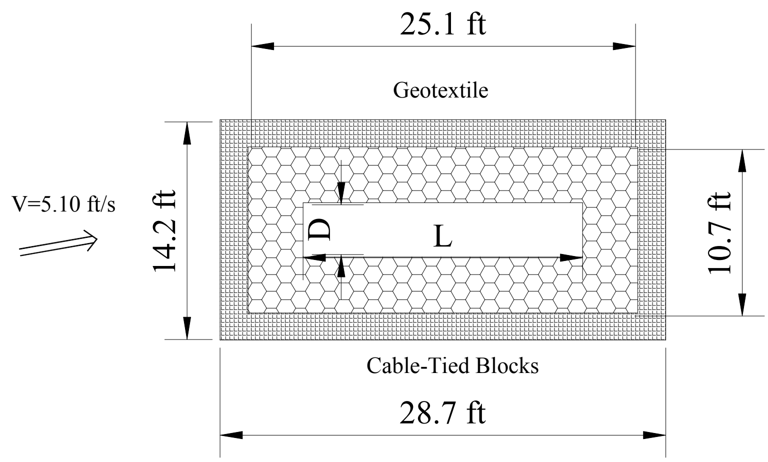


Figure 8.9: Installation and Layout of Cable-Tied Blocks Around Bridge Pier.

CHAPTER 9

CONCRETE ARMOR UNITS

9.1 DESCRIPTION

Concrete Armor Units are also known as “artificial riprap”. They are used as an alternate for rock riprap, especially when there is a short supply of rock riprap. The primary advantage of armor units is their usually greater stability.

The increased stability allows their placement on steeper slopes or the use of lighter weight units for equivalent flow conditions as compared to riprap. This is significant when riprap of a required size is not available. However, unlike natural riprap it is difficult to grow vegetation over concrete blocks.

Concrete armor units are factory manufactured precast units and are commercially produced by proprietary firms. They have been used for erosion control in recent years mainly as revetments but less frequently as countermeasures for bridge substructures. Figure 9.1 shows examples of concrete armor units.

9.2 APPLICATIONS OF CONCRETE ARMOR UNITS

They have been used in environments where riprap availability is limited or large rock sizes are required to resist extreme hydraulic forces. Armortec three-dimensional units have been used for abutments and pier protection and as revetments for channels and stream banks. They are used both for fluvial and coastal conditions to resist wave attack.

The commonly used concrete armor blocks are Toskanes (shown in Figure 9.1) and A-jacks (shown in Figure 9.2) developed by Armortec Company. Examples of pier scour countermeasure are presented in Design Guideline 6 of HEC-23.

Since these blocks are proprietary in nature, standard details of these blocks can be obtained from vendors supplying them. Three prominent companies supplying these blocks in New Jersey are:

1. Contech Construction Products Inc. (contech-cpi.com)
2. Pavestone Company (hydropave.com)
3. Armortec Inc., Bowling Green, KY (Prepared by Ayres Associates)

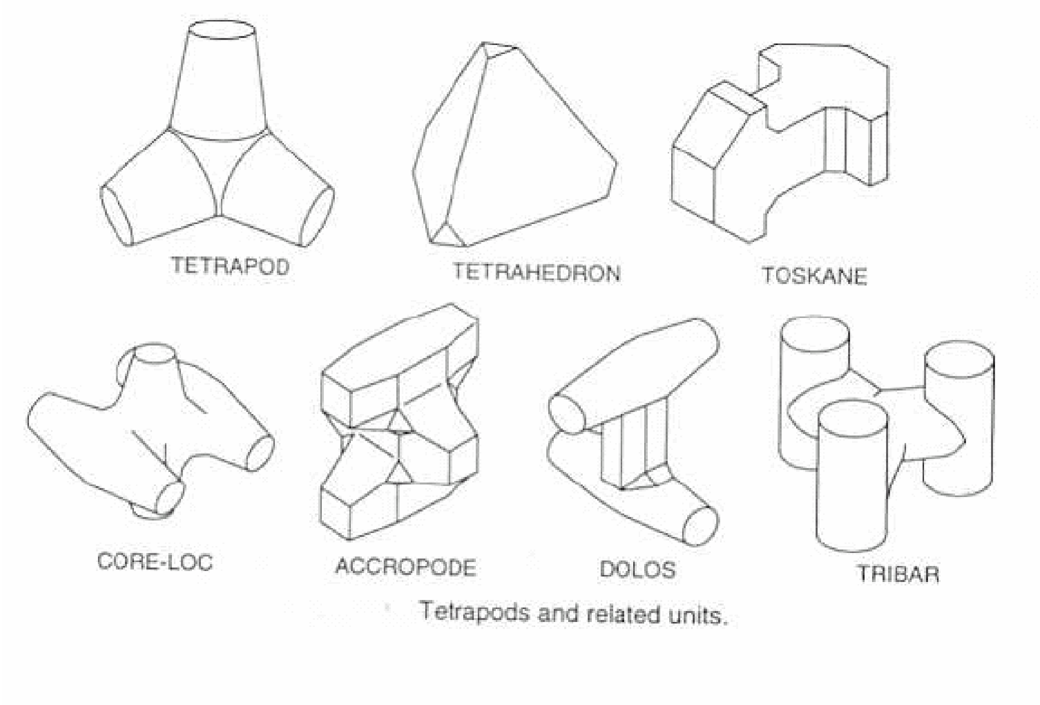


Figure 9.1. Precast concrete armor units

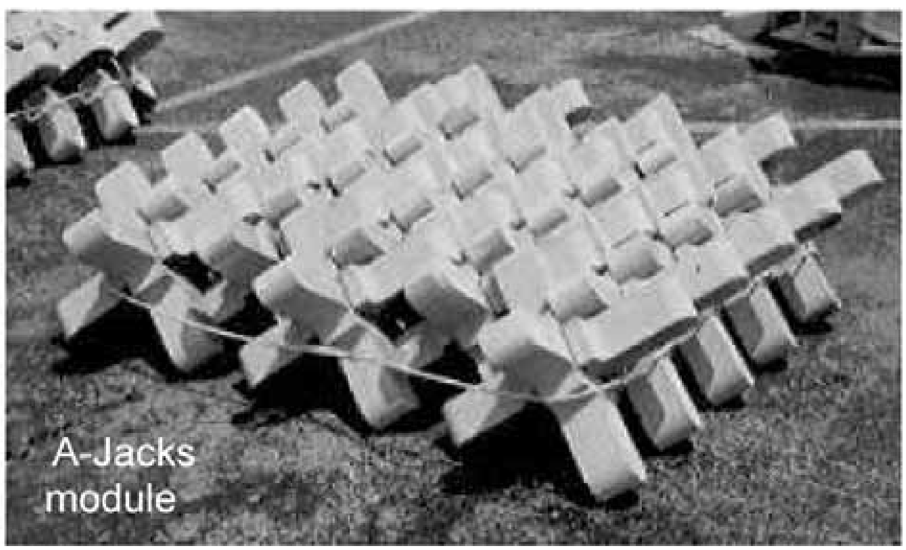


Figure 9.2. A-Jacks Module For Pier Scour Countermeasures.

9.3 DESIGN PROCEDURE FOR TOSKANES

Whenever available, design guidelines provided by manufacturers should be used for specific proprietary systems. Comprehensive design procedure for standard sizes of Toskanes for bridge piers and abutments should be used when manufacturer specific guidelines are not available. The following design procedure is based on Design Guideline 6 of HEC-23.

1. **Design Velocity**: The design velocity should be calculated as:

$$V_v = 1.5 V_0 C_l C_s C_h C_i \qquad (9\text{-}1)$$

where factor "1.5" is the factor of safety. Other parameters in Equation (9-1) are defined as,

V_0 = Average velocity directly (approximately 10 ft) upstream of the bridge by considering the number of substructure elements in the flow at the bridge cross section and effects of construction.

C_l = Location Adjustment Coefficient
= 0.9, for a location near the bank of the river.
= 1.0, for most applications
= 1.1, for a structure in the main current of flow at a sharp bend.
= 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.

C_s = Shape Adjustment Factor. If the angle of attack, α, is greater than 5°, set all shape coefficients to 1.0.

= 1.0, for a circular pier.
= 1.1, for a square nose pier.
= 0.9, for a sharp nose pier streamlined into the approach flow.
= 1.1, for a vertical wall abutment.
= 0.85, for a vertical wall abutment with wingwalls.
= 0.65, for a spill through abutment.

C_h = Top surface alignment factor (if the top surface of the pad is placed level with the channel bed)

= 1.0, Level - Top of pad is flush with the channel bed.
= 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 development of the guideline in HEC-23. Pad heights were kept at 0.2 times the approach flow depth or less.

C_h = Random or pattern installation factor. A random installation refers to the units beings dumped into position. In a pattern installation, every Toskane is uniformly placed to create a geometric pattern around the pier. Pattern installation is recommended in New Jersey.

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

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 a cross section on a relatively straight reach. A 2-dimensional hydraulic model (i.e., FESWMS, RMA-2V) could be used to estimate local velocities in meandering reaches or reaches with complex flow patterns.

2. **Adjusted structure width (b_a)**:

For a pier

b_a = pier width b if the angle of attack (α) for high flow conditions < 5°

$b_a = L\sin\alpha + b\cos\alpha$ if the angle attack (α) for high flow conditions is > 5°, L being length of the pier.

If a footing extends into the flow field a distance greater than 0.1 y_o (flow depth) use footing width instead of pier width for b.

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

If $b \leq$ 5 ft,	b_a = 5 ft
If 5 ft $\leq b \leq$ 20 ft,	$b_a = b$
if $b_a \geq$ 20 ft,	b_a = 20 ft

The distance an abutment projects perpendicular to the flow will be different at river level and at the channel bed because of the slope of river bank. In such situation b_a can be calculated as an average of b at river level and at the channel bed.

3. **Sizing of Toskanes**:

The equivalent spherical diameter is the size of a sphere that would have the same volume of material as the armor unit as determined by:

$$Du = \frac{0.255 V_v \sqrt{\frac{b_a}{g}}}{S_g - 1} \quad (9\text{-}2)$$

where S_g is specific gravity of Toskanes. Figure 9.3 shows Toskanes design parameters and dimensions. Table 9.1 shows Toskanes design dimensions in terms of Toskanes height H. Table 9.2 shows recommended standard sizes of Toskanes. Table 9.3 shows Toskanes design dimensions and parameters.

Sizing of Toskanes can be done through the following steps:

a. Determine Du from Equation (9-2) based on velocity value, V_v, and the adjusted structure width, b_a.

b. Select standard Toskane size from Table 9.2 such that Du of selected Toskane is greater than that calculated from Equation (9-2).

c. Check the b_a / D_u ratio using the diameter, D_u, of a standard Toskane size in Table 9.1. If the $b_a / D_u > 21$, select the next larger size of Toskane. Repeat until $b_a / D_u < 21$.

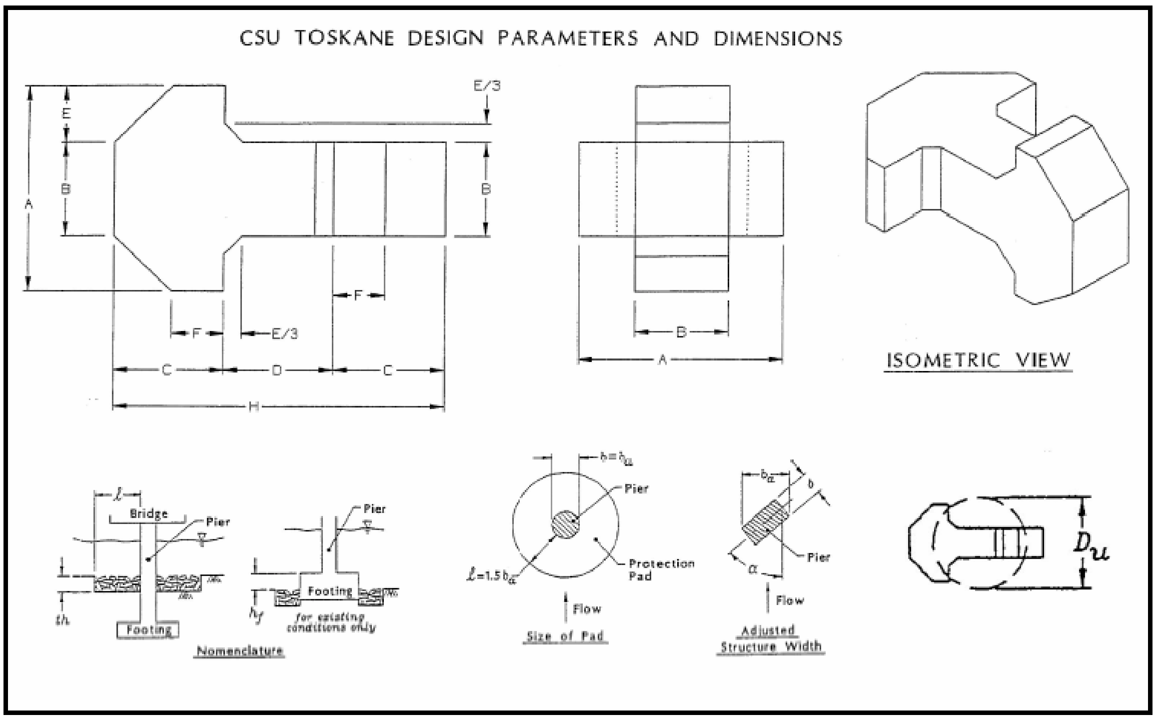


Figure 9.3: Toskanes Design Parameters and Dimensions.

Table 9.1: Toskanes Design Dimensions.

Du	0.622H
A	0.616H
B	0.280H
C	0.335H
D	0.330H
E	0.168H
F	0.156H

Table 9.2: Standard Sizes of Toskanes.

Du (ft)	Weight (lb)
1.47	250
1.85	500
2.12	750
2.33	1,000
2.67	1,500
2.94	2,000

Table 9.3: Toskane Design Parameters and Dimensions

Design Parameter Dimension	
Toskane length (H)	1.608Du
Equivalent spherical diameter (Du)	0.622H
Volume (V)	$0.5236Du^3 = 0.1263H^3$
Specific weight (γ)	23.5 KN/m^3 150 lb/ft^3
Density (ρ)	2400 kg/m^3 4.66 $slug/ft^3$
Number of Toskanes per unit area (N)**	$0.85V^{-2/3} = 1.309Du^{-2}$
2 layer thickness (*th*)	2.0Du = 1.24H
Filter requirements	$D_{85(filter)} = 0.22Du$
Size of Pad (*l*)	$l_{min} = 1.5b_a$ (piers) $l_{min} = 2.0b_a$ (abutments)
**Toskanes per unit area assuming a 2-layer thickness of 2Du.	

4. **Pad radius**: Use pad radius as $1.5b_a$ for piers and $2b_a$ for 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. (e.g. pads around piers and abutments)

If more than one Toskane pad is present in the stream cross section, check the spacing between the pads. If a distance of 5 ft or less exists between pads, extend the width of the pads so that they join.

5. **Number of Toskanes per unit area**: Use Table 9.2 to determine number of Toskanes per unit area and the pad thickness. Pads with randomly placed units have to be a minimum of two layers thick.

6. **Filter Requirement:**

   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, d_{85} of the filter material directly below the Toskane layer can be determined from Table 9.3. Additional layers of filter, that may be needed based on the gradation of the bed material, can be designed according to standard requirements in Chapter 11.

7. **Placement**

   Toskanes can be installed around bridge piers and abutments in the configuration as shown in Figure 9.4. This is based on the example design presented in HEC-23.

   When more than one layer is used, placing the units in their most efficient interlocking position provides greater stability. This is as opposed to dropping them into an arbitrary position, irrelevant to the previous layer. Machinery may be needed for large armor units.

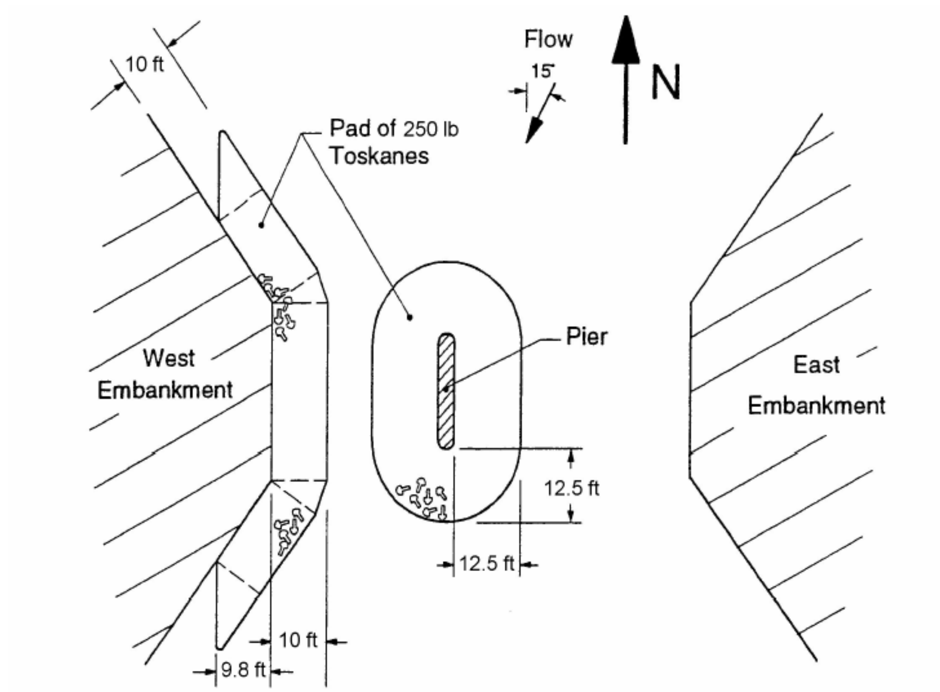


Figure 9.4. Typical Placement of Toskanes around Bridge Piers and Abutments Based on Example Design in HEC-23.

9.4 DESIGN PROCEDURE FOR A-JACKS

The basic construction element of A-jacks for pier scour applications is a "module" comprised of 14 individual A-jacks banded together in a densely-interlocked cluster, described as a 5x4x5 module. The following design procedure for A-Jacks systems for pier-scour protection is based on Design Guidelines 6 of HEC-23.

1. **Hydraulic Stability of A-Jacks**: Hydraulic stability of 5x4x5 A-Jacks module can be estimating by equating the overturning moment due to total drag force to the resisting moment due to submerged weight,

$$F_d H_d = W_s L_w ; F_d = 0.5 C_D \rho A V^2 \tag{9-3}$$

where:

F_d: Drag Force
C_d: Drag Coefficient = 1.05 (a value of 1.2 can be assumed for conservative design)
ρ: density of water
A: Frontal area of A-Jacks module
V: Flow velocity immediately upstream of A-Jacks module
H_d: Moment arm of drag force (at full height of A-Jacks module)
W_s: Submerged weight of A-Jacks module

L_w: Moment arm for submerged weight

Parameters A and L_w in Equation (9-3) are obtained from physical characteristics of A-Jacks. Table 9.4 shows hydraulic characteristics of prototype size 5x4x5 A-Jacks module based on laboratory testing.

Table 9.4 Hydraulic characteristics of 5x4x5 A-Jacks modules

A-Jacks System	Tip-to-Tip Dimension of Armor Unit (in)	Module Dimensions (HxWxL) (in)	Weight (or Mass) in Air, lbs (kg)	Submerged Weight (or Mass, lbs (kg)	Limiting Upstream Velocity, ft/s (m/s)
AJ-24	24	16 x 52 x 40	1,030 (467)	540 (245)	10.7 (3.3)
AJ-48	48	32 x 104 x 80	8,270 (375)	4,300 (1,950)	15.1 (4.6)
AJ-72	72	48 x 156 x 120	27,900 (12,655)	14,500 (6,577)	18.5 (5.6)
AJ-96	96	64 x 208 x 160	66,200 (30,028)	34,400 (15,604)	21.4 (6.5)

Notes:

1. Volume of concrete in ft^3 for a 14-unit module is $14 \times 0.071 \times L^3$ where L is tip-to-tip dimension of armor unit in feet.
2. Values in table assume a unit weight (or mass) of 130 lbs/ft^3 (2,083 kg/m^3) for concrete.

2. Geometry of A-jacks modules:

   Figure 9.5 shows recommended layout of A-Jacks modules around a pier of width “a” and unprotected depth of scour, y_s (as calculated on HEC-18).

   Stability of installation can be improved by placing modules with long axis parallel to the flow.

   Partial burial of modules will improve the stability.

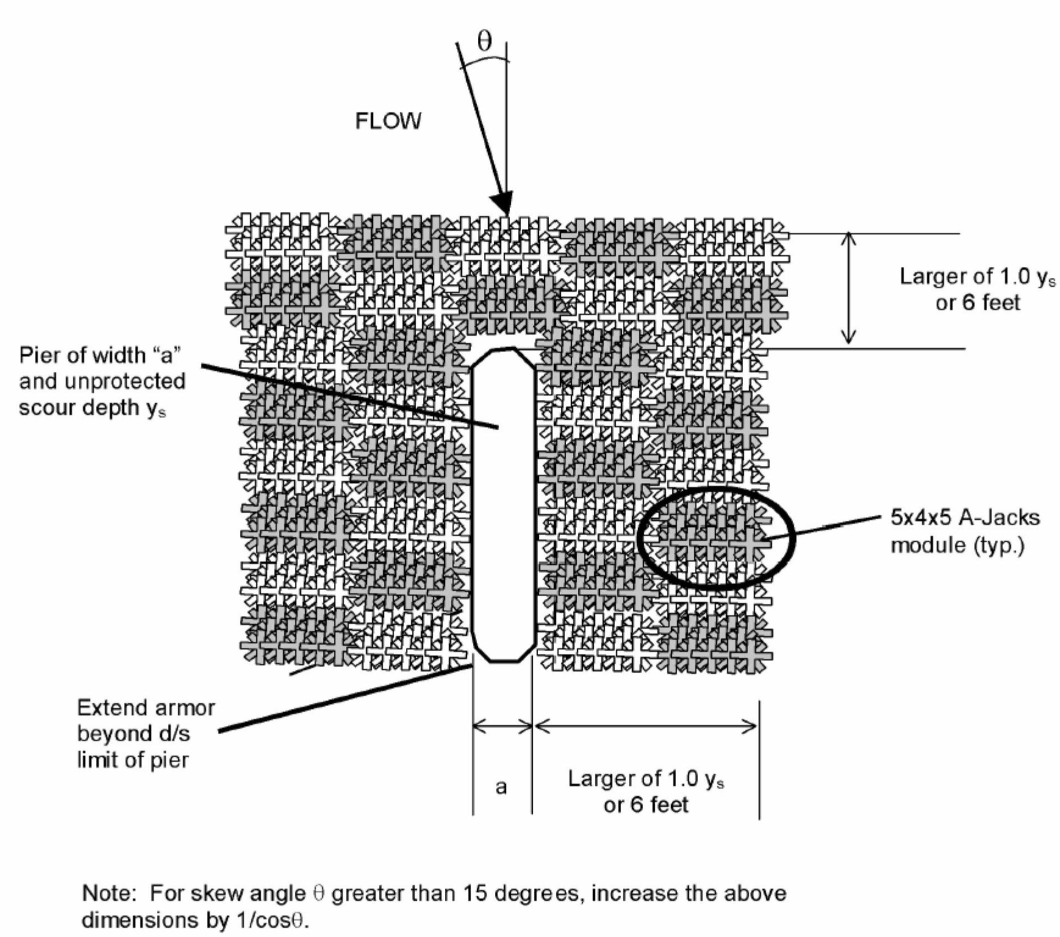


Figure 9.5. Typical Layout of A-Jacks Modules for Bridge Piers.

3. Placement of A-Jacks:

   A-Jacks can be constructed on site in the dry and banded together in 5x4x5 clusters in place around the pier, after the placement of suitable bedding layers. Alternatively, the modules can be pre-assembled and installed with a crane and spreader bar. This arrangement may be more practical for placement in or under water.

   Bands should be comprised of cables made of UV-stabilized polyester, galvanized steel, or stainless steel, as appropriate for the particular application. Crimps and stops should conform to manufacturer's specifications. When lifting the modules with a crane and spreader bar, all components of the banding arrangement should maintain a minimum factor of safety of 5.0 for lifting.

   Where practicable, burial or infilling of the modules to half-height is recommended so that the voids between the legs are filled with appropriate sized stones based on bedding considerations.

4. Bedding Considerations:

   Bedding layer of stone, geotextile fabric, or both, can appreciably enhance the performance of A-JACKS by limiting the depth of scour at the pier nose.

   The size of bedding stone is determined to (i) to retain the finer fraction of native bed material that could otherwise be pumped out between the legs of the A-JACKS armor units, (ii) to relieve potential pore water pressure under the installation, and (iii) to resist being plucked out through the legs of A-Jacks during turbulent flows.

   Recommended sizing criteria for bedding stone is:

   Retention: $D_{85(Lower)} > 0.25D_{15(Upper)}$
   $D_{50(Lower)} > 0.14D_{50(Upper)}$

   Permeability: $D_{15(Lower)} > 0.14D_{15(Upper)}$

   Uniformity: $D_{10(Upper)} > 0.10D_{60(Upper)}$

   where Dx is the particle size for which x percent by weight are finer and designations Upper and Lower refer to respective positions of various granular bedding layers in case of multiple layers. Lowest layer corresponds to native streambed material.

   Each bedding layer should be 6 to 8 inches thick. The thickness of upper layer should be based on Table 9.5.

   When using geotextile, a layer of blast stone must be placed on top before installing A-Jacks modules. Geotextile should have a permeability at least 10 times that of the native streambed material.

   Apparent Opening Size (AOS) of geotextile should retain at least 30 percent, but not more than 70 percent, of the grain sizes present in the bed.

   Geotextile should be strong enough to endure stresses encountered during the placement of A-Jacks modules.

Table 9.5 Recommended Properties of Uppermost Bedding Layer

A-Jacks System	D_{50} Size of Uppermost Layer, in (mm)	Recommended Minimum Thickness of Uppermost Layer, in (mm)
AJ-24	2-3 (50-75)	8 (200)
AJ-48	4-6 (100-150)	12 (300)
AJ-72	6-9 (150-225)	24 (600)
AJ-96	8-12 (200-300)	30 (750)

9.5 ANCHORS

Details of end anchors and anchors to the riverbed should be provided by the block manufacturers.

The edges of the concrete block layer should be adequately tied into the underlying material to prevent edges from being lifted under high turbulence and undermined.

9.6 FILTER LAYERS

On sand river beds, a geotextile filter should be placed underneath the units to prevent sand leaching. The geotextile filter may be sealed to the pier by a flexible tube containing a cable that can be tightened around the pier. For details of filters, see the section on granular filters in Chapter 11.

9.7 CONSTRUCTION PERMITS

Stream encroachment and other applicable permits will be required. Refer to Guidelines given for permit application in NJDEP Stream Encroachment Technical Manual.

9.8 DURABILITY AND MAINTENANCE

The following types of failures may occur and may be avoided by good construction practice:

1. Undermining of blocks may occur because of bed movement.
2. Development of a gap between edge of concrete blocks and the structure, leading to loss of underlying material.
3. Edge failure due to erosion of a scour hole in the natural bed adjacent to ACB protection in which blocks can fall in.

4. Movement and progressive collapse can occur at slopes if edges are not tied in.

9.9 COSTS

Materials and construction costs for both Concrete Armor Units are higher than those for riprap and may depend on vendor supplied specifications.

9.10 DESIGN EXAMPLES

Design of Toskanes for Piers

A bridge over tributary to Lamington River has a single pier located on the outside of a bend. The pier is round nosed and is 3.5 ft wide and 18 ft long. The footing is not exposed and bed material consists of cobbles and gravel. The average velocity directly upstream of the bridge during high flow is 5.08 ft/s for 500 years flood and has an angle of attack of 10°. Design appropriate scour protection measures using Toskanes.

1. Velocity value, V_v (ft/s)

   C_l = 1.1 (The pier is located in the thalweg of the bend)
   C_s = 1.0 (Angle of attack, α = 10° > 5°)
   C_h = 1.0 (Top of the pad is level with the bed)
   C_i = 1.0 (Randomly installed pad of Toskanes)
   $V_0 = \alpha \cdot \beta \cdot V_{500} = 1.2 \times 1.0 \times 5.08 = 6.10$ ft/sec
   $V_v = 1.5\ V_0\ C_l\ C_s\ C_h\ C_i = (1.5)(6.10)(1.1)(1.0)\ (1.0)\ (1.0) = 10.06$ ft/s

2. Adjusted structure width, b_a (ft)

   Angle of attack, α = 10°.
   Length of pier, L = 18 ft.
   Pier width, b = 3.3 ft.
   $b_a = L \sin \alpha + b \cos \alpha = 6.57$ ft

3. Using Equation (9-2), the equivalent spherical diameter, Du, for V_v = 10.06 ft/s, b_a = 6.57 ft and S_g = 2.24 is calculated as,

   $$D_u = \frac{0.255(10.06)\sqrt{\frac{6.57}{32.2}}}{(1.24)} = 0.93 \text{ ft} = 11.2 \text{ in.}$$

   From Table 9.2, a standard sized 250 lb Toskane unit with Du = 1.47 ft is selected.
   The ratio b_a / D_u = 6.57/1.47 = 4.5 < 21. Hence, the selected 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 = 1.5b_a is chosen.

   Pad Radius, $l = 1.5(6.57) = 9.9\text{ft} \approx 10\text{ft}$

   The Toskanes will be installed around the pier, a horizontal distance of 10 ft from the wall of the pier, as shown in Figure 9.6.

5. From Table 9.3, the number of Toskanes per unit area for the 250 lb Toskane size with a pad thickness of 2*Du* is 0.61 Toskanes/ft^2. Hence, the total area of the pad in Figure 9.6 is:

   Area = 2(14.5(10)) + (π(11.75^2 – 1.75^2)) = 1139 ft^2

   No. of Toskanes = 0.61 (1030) = 695 Toskanes

   The pad thickness is 2Du = 3 ft

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

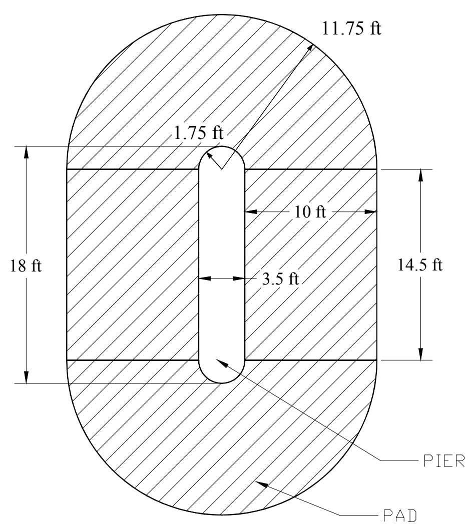


Figure 9.6 Layout of Toskanes around the Bridge Pier.

Design of Toskanes for Abutments

The bridge Tributary to Lamington River in Figure 1.1 has vertical wall abutments with wing walls. During normal flows the west abutment extends 2 ft into the flow, but during high flows it obstructs 7.9 ft 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. V_{500} = 5.54 ft/s.

1. Velocity value, V_v (ft/s).

   C_l = 0.9 (The abutment is located near the bank, outside of the thalweg)
   Cs = 0.85 (the abutment has wing walls)
   C_h = 1.0 (top of the pad is level with the bed)
   C_i = 1.0 (randomly installed pad of Toskanes)
   $V_0 = \alpha \cdot \beta \cdot V_{500} = 1.0 \times 1.0 \times 5.54 = 5.54$ ft/sec
   $V_v = 1.5\, V_0\, C_l\, C_s\, C_h\, C_i$ = 6.36 ft/s

2. Adjusted structure width, b_a (ft)

   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 7.9 ft at the water surface and 0 ft at the channel bed (see Figure 9.3). Therefore an average value of b_a is

   $b_a = \frac{7.9}{2} + 0.0 = 4.0\, ft$ < Minimum value of 5 ft
   Hence, b_a = 5 ft

3. Using Equation (9-2), the equivalent spherical diameter, Du, for V_v = 6.36 ft/s, b_a = 5 ft and S_g = 2.24 is calculated as,

   $$D_u = \frac{0.255(6.36)\sqrt{\frac{5.0}{32.2}}}{(1.24)} = 0.52 \text{ ft} = 6.2 \text{ in}$$

   From Table 9.2, 250 lb Toskane with D_u =1.47 ft is selected. A smaller 125 lb Toskane could have been selected, but this non-standard size may not be economical.

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(5.0) = 10 ft

The Toskanes will be installed along the abutment and wingwalls a horizontal distance of 10 ft from the wall. Figure 9.7 shows the layout of Toskanes along abutments. Other dimensions in Figure 9.7 depend on specific abutment and wingwall dimensions.

5. The pad thickness is 2*Du* which will result in 0.61 Toskanes/ft^2. The total area of the pad in Figure 9.7 is,

   Area = (10)(29.5) + 2(10)(16.4) + 2(0.5)(10)(5.9) + 2(0.5)(10)(13.1) = 813 ft^2

   Number of Toskanes = (813)(0.61) = 496 Toskanes.

6. Granular filter with d_{85} = 95 mm is placed under the pad for the bed material consisting of cobbles and gravel.

   The distance between the pier and the west abutment is not specified in this example. If the spacing between the two protection pads is 5 ft or less, it is recommended that the pads be joined to form a continuous pad between the abutment and the pier.

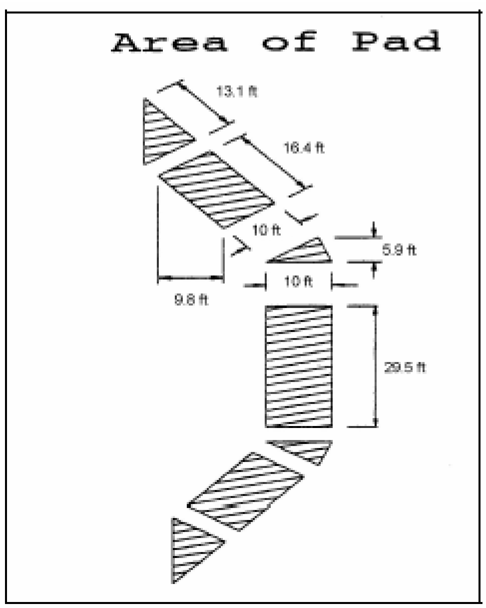


Figure 9.7 Layout of Toskanes for Bridge Abutment.

CHAPTER 10

GROUT FILLED BAGS AND MATS

10.1 GENERAL

1. Grout bags are fabric shells that are filled with concrete. Grout mats are single, continuous layers of fabric with pockets, or cells that are filled with concrete. Grout bags are smaller units that can be stacked in a manner similar to gabions. Hence, grout-filled mats or bags are essentially artificial riprap.

2. It was found that properly-installed grout mats and grout bags reduce scour depth to a degree generally comparable with riprap.

3. In cases of small bridges, bags can be installed where it is difficult to bring in equipment for the placement of riprap.

4. The main body of literature pertaining to grout filled mats and bags is contained in Fotherby (1992, 1993) Bertoldi et al. (1994), and Jones et al. (1995a, b). Bertoldi et al. (1994) report that grout mats have been employed by the U.S. Army Corps of Engineers to prevent bank erosion. They confirmed the need for anchors in the case of grout mats placed on top of a loose, erodible bed. It was again found that placement is extremely important for successful performance.

5. Figure 9.1 shows three configurations for the placement of grout bags. Properly placed grout mats extending 1.5 times a pier width were found to provide significant protection to bridge piers. In the case of grout bags, bags along the side of the pier aligned flush with its front end tended to be prone to failure. A staggered placement (Figures 9.1 (b) and (c)) provided better protection.

10.2 DESCRIPTION AND LIMITATIONS

Grout bags, sacks or mattresses are one of the cheapest and simplest types of armoring against erosion.

Grout bags are more stable than riprap under flood conditions and are useful in the following conditions:

1. In underwater conditions to form a concrete apron, over slopes and banks

2. In filling scour holes

3. For temporary repairs

They are both rigid types and flexible types. They can be used in combination with riprap, which is placed at the toe to prevent undermining.

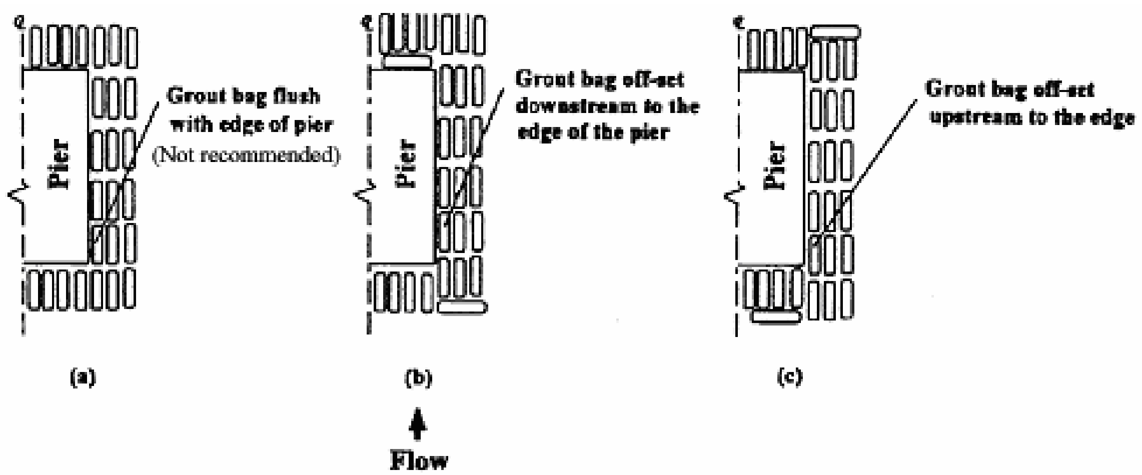


Figure 10.1. Grout Filled Bags and Mattresses for Pier Scour Countermeasures (NCHRP 12-47)

10.3 MATERIALS

The fabric material that is used is Burlap, Jute, Hessian or a synthetic material, such as polyester and polypropylene. The fabric acts as a shutter to retain sand or grout and form the shape of revetment.

If construction is underwater bags are filled up, before they are placed in position.

The bags are filled up, generally on the site with

1. Dry sand
2. Wet sand
3. A dry mixture of cement and sand: The mixture hydrates and hardens on contact with water. A mixture of 15 % cement and 85% dry sand, by weight is suitable for use.
4. Wet grout pumped into bags.

Figures 10.2 to 10.7 show applications using grout bags at abutments and piers.

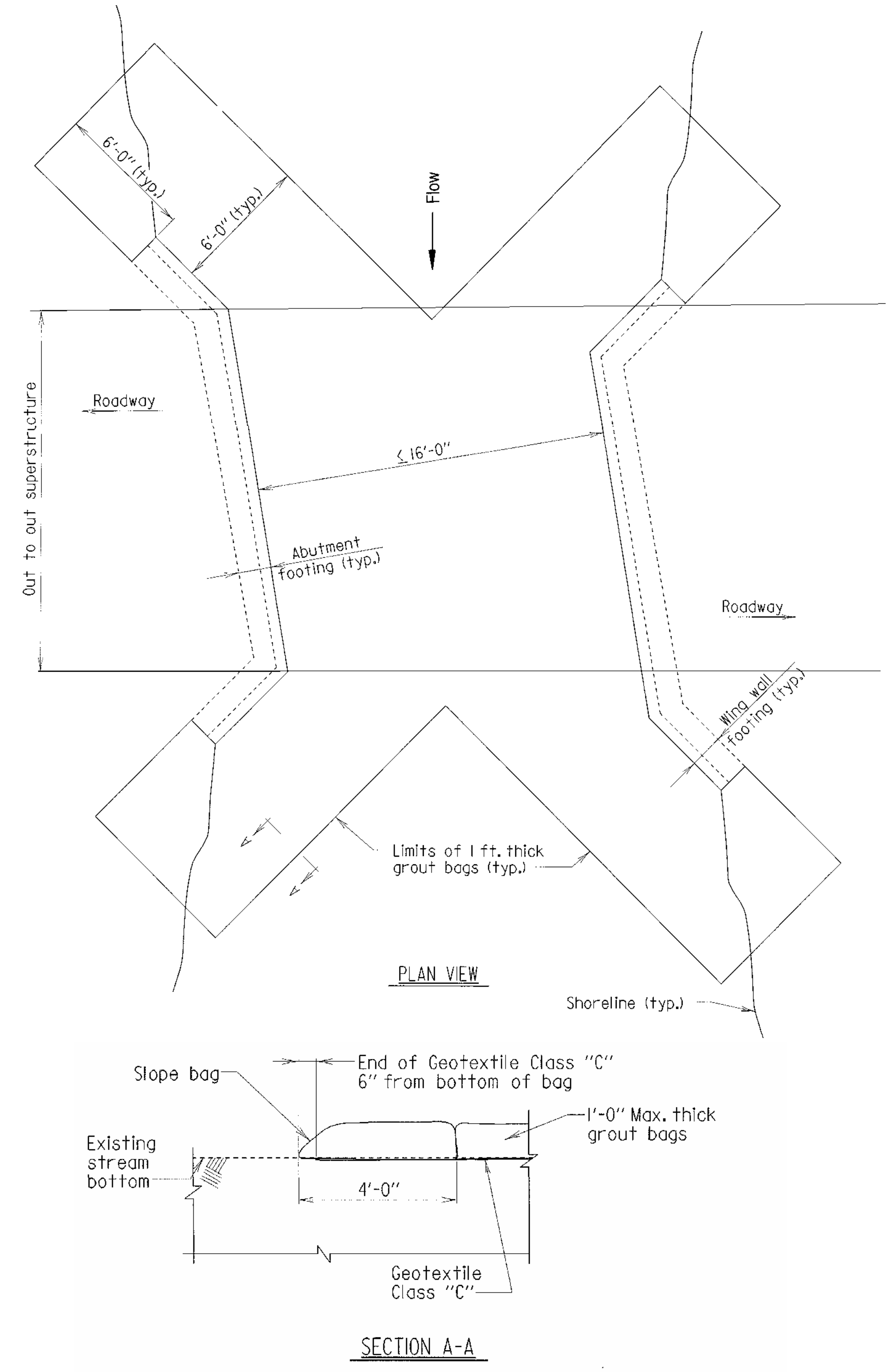


Figure 10.2. Plan View of Grout Bags (Case Where Scour Potential Exists for Full Channel Width) N.T.S

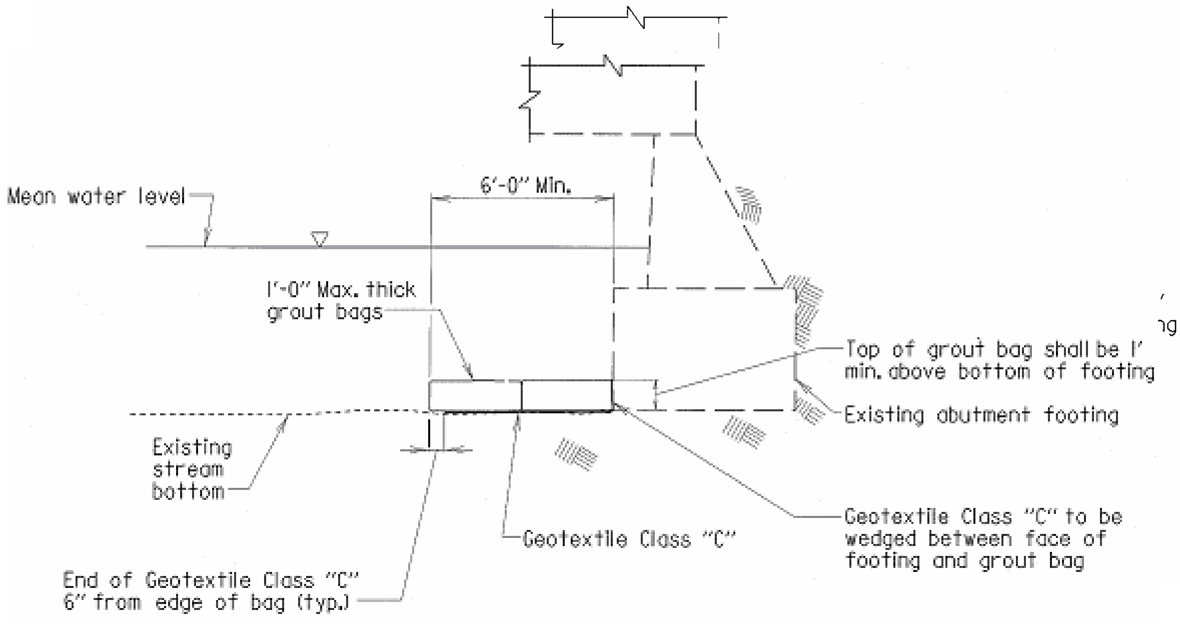


Figure 10.3. Grout Bag Section (Section Thru Abutment) Case Where Scour Potential Exists at Abutment N.T.S

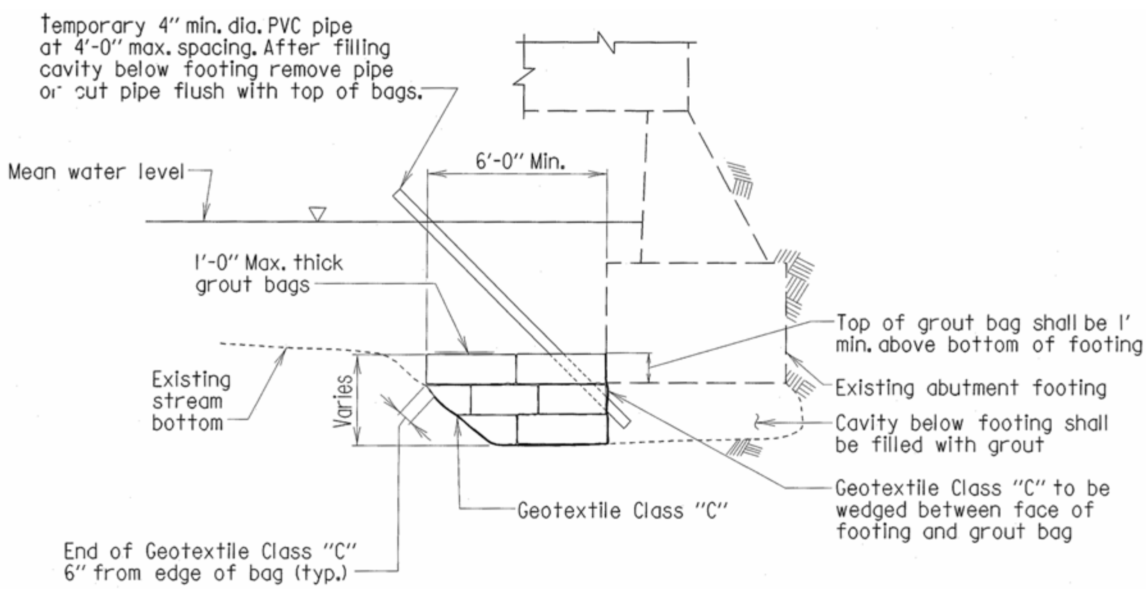


Figure 10.4. Grout Bag Section (Section Thru Abutment) Case Where Scour and Undermining Has Occurred at Abutment N.T.S

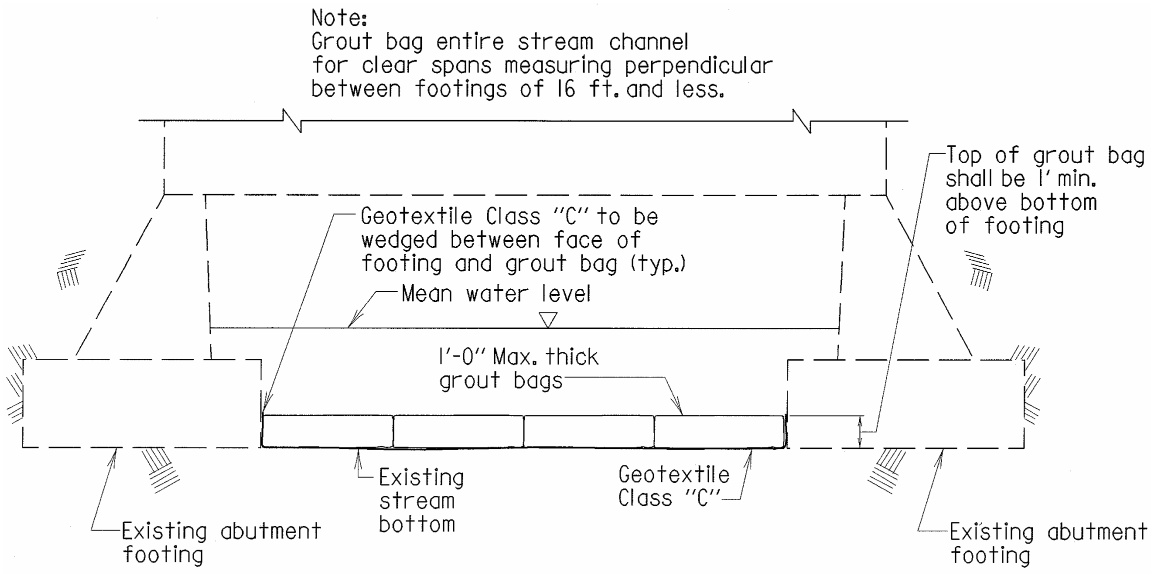


Figure 10.5. Section View of Grout Bag (Thru Abutments and Channel) Case where Scour Potential Exists for Full Channel Width N.T.S.

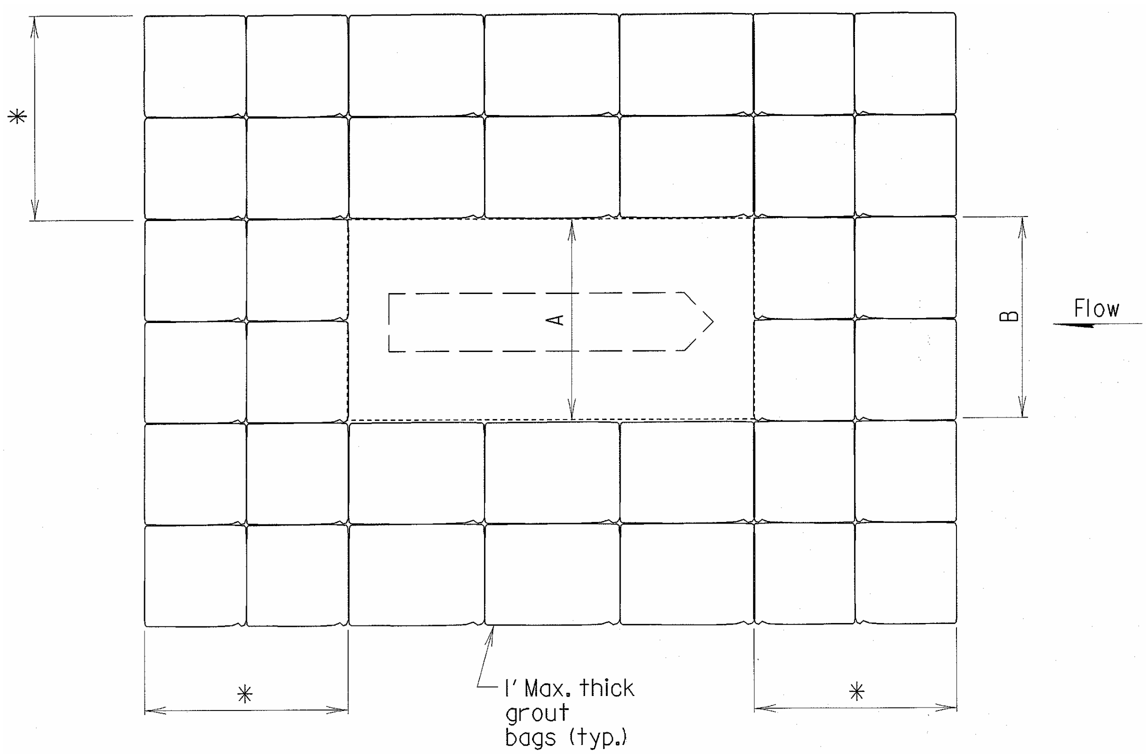


Figure 10.6. Plan View of Grout Bag Installation at Pier N.T.S.

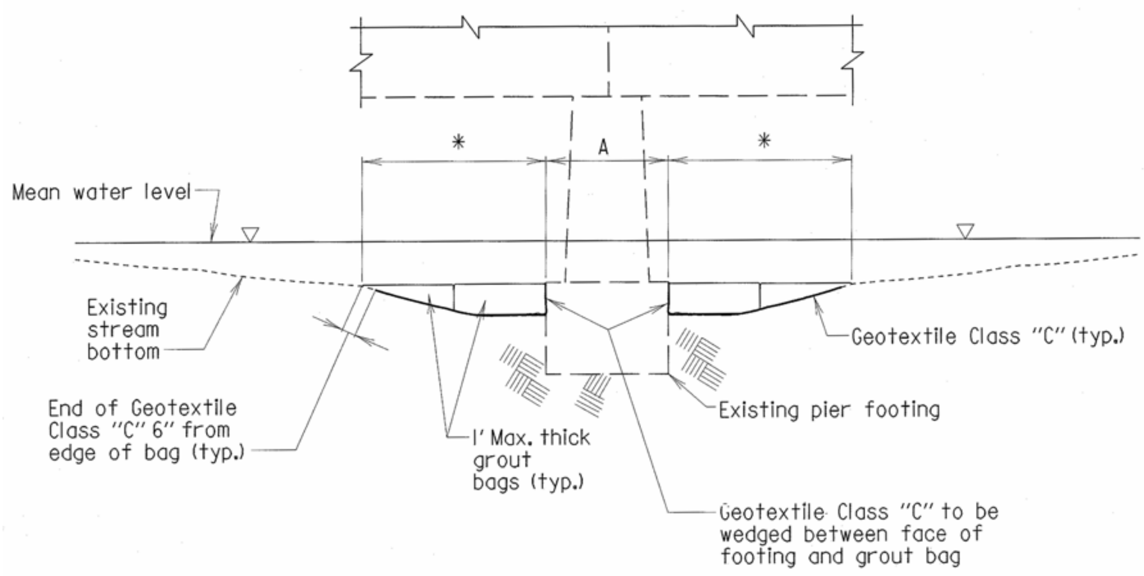


Figure 10.7. Grout Bag Section (Section Thru Pier) Case Where Scour Potential Exists at Pier N.T.S.

10.4 FAILURE OF GROUT BAGS

1. Parker et al (1998) performed a series of hydraulic tests on grout filled bags and observed that grout filled bags did not perform as well as riprap or cable tied blocks. Their lack of angularity resulted in poor interlocking, and their relatively smooth surfaces resulted in failure by sliding at relatively low velocities.

2. The grout bags were prone to catastrophic failure if they were too small. Otherwise the mode of failure was a gradual erosion process similar to that in riprap.

3. Long grout filled bags are not only prone to sliding but also can be undermined because of leeching of sand. Thus, countermeasure may not settle, but a significant scour hole can develop beneath it.

10.5 DESIGN PROCEDURES

The design size of a bag or depth of a layer depends upon the following

1. A design flood velocity of 5 to 10 ft/sec

2. A computed scour depth for contraction and local scour of 3 to 6 ft.

3. When hydrostatic pressure builds up, the dead weight of bags should exceed the uplift pressure. The mattresses should be provided with filter drains or drain holes for pressure relief.

4. Depending upon the application, bags may vary in capacity, from standard cement bag size, to about 5 ft^3, while mattresses are larger in size up to 15 ft^3 in volume.

5. Mats must be bound firmly to the pier itself for a good performance. Mats should be installed with their top surfaces flush to the bed.

6. Grout bags should be sized and placed in a manner similar to riprap, and underlain by a geotextile filter with a partial cover or filter layer. Any means to render the surface of bags rough and angular will aid to performance.

7. Properly sized bags are more effective when they extend a single layer of protection laterally, rather than if they were stacked. Efforts should be made to avoid stacking of grout bags.

8. Flexible bags of sand may be preferable to grout-filled bags.

10.6 CONSTRUCTABILITY ISSUES AND CONSTRUCTION EQUIPMENT

1. Cofferdams are not used with grout or sand bags, which make them more economical.

2. Bags or mattresses are usually connected by

    a. Straps

    b. Ties or

    c. Proprietary connectors

3. In addition, the edges should be secured through the following

    a. They are secured at the edges with anchors

    b. Their edges are toed into the underlying material

    c. Gaps between structure and bags are grouted.

4. The filled sacks should be placed in horizontal rows like brick mortar layered construction. When used on slopes, the angle should not be greater than 45 degrees.

10.7 UNDERWATER CONSTRUCTION

If construction is underwater, the bags may be filled up using tremie concrete, after empty bags are placed in position. Divers may be required for deeper water to ensure proper placement and end anchorage.

Alternatively, bags may be transported by barges and deposited in position. They may be placed in two or more layers.

10.8 FILTER LAYERS

On a sand river bed, a geotextile filter may be placed underneath the bags to prevent sand leaching. The geotextile filter may be sealed to the pier by a flexible tube containing a cable that can be tightened around the pier. More details on filter are provided in Chapter 11.

10.9 CONSTRUCTION PERMITS

Stream encroachment and other applicable permits will be required. Refer to Guidelines given for permit application in NJDEP Stream Encroachment Technical Manual.

10.10 DURABILITY AND MAINTENANCE

The following types of failures may occur and should be avoided:

1. Undermining
2. Gradual collapse
3. Settlement of ground

Weep holes should be installed to allow drainage of ground water from behind the revetment to prevent pressure build up that could cause pressure failure.

10.11 COST

Grout bags are more expensive than riprap. Costs in 2005 are $25 to $30 per sq. ft for each layer of grout bags.

CHAPTER 11

TEXTILE AND GRANULAR FILTERS AS SECONDARY ARMORING

11.1 DESCRIPTION

Filters are a secondary countermeasure but when combined with primary a countermeasure such as rock riprap, gabions or artificial riprap, it acts as an additional armoring.

The use of a filter is to ensure that underlying fine sediment particles do no leach through the voids of the individual stones that compose the riprap. Filter prevents migration of subsoil particles through the protection. It allows water flow across the soil boundary and avoids build-up of an unacceptable head across it.

At deep placement depths within the bed, the dominant failure mode for the riprap layer becomes leaching. To prevent this loss of material, presence of some form of filter beneath the stone layer would be of benefit.

In the case of interconnected gabions on a sand bed river, a geotextile filter placed underneath the baskets should be used to prevent sand leaching. The fabric should provide drainage and filtration and

1. Keep fine particles underneath in place
2. Allow for release of pore pressure, and
3. Help reinforce the armoring.

11.2 TESTING PROGRAM FOR GEOTEXTILE QUALITY

1. Impact test (to determine punching resistance, e.g., when large stone is dropped on the geotextile)
2. Abrasion test
3. Permeability, clay clogging, and sand clogging tests; and
4. Tests of material characteristics such as elongation and strength.

Through this testing program, geotextile materials have been developed that permit innovative approaches to filter placement for riprap and other countermeasures.

11.3 LIMITATIONS

Both geotextile and granular filters are not required for gravel riverbed. This is due both to the abrasive nature of gravel and its low potential for leaching. A gravel bed stream has a surface median bed material size D_{50} > 2 mm but suitable for sand bed stream for which 0.06 mm < D_{50} < 2 mm.

On steep slopes, highly erodible soils, loose sand, or with high water velocities, a filter should be used or riprap thickness should be increased beyond the minimum.

11.4 TYPES OF FILTERS

Since the hydraulic response of both synthetic and granular filters can be similar, the choice of materials is usually based on practical considerations.

The following types of filters are recommended for use in New Jersey:

1. Geotextile Filter

This covers a wide range of synthetic grids, meshes and textiles. A synthetic filter fabric is manufactured for specific applications.

a. Advantages

1.) Relatively low cost

2.) Large areas can be laid quickly

3.) Small construction thickness. Volume of riprap is therefore reduced considerably compared to no filter armoring.

4.) Generally provides a broad band of failure threshold.

5.) Non-woven types can cope with soil variations and allow for settlement.

b. Disadvantages

1.) Long-term behavior is less certain than granular filters

2.) Difficult to lay in deep water

3.) Difficult to place in high currents

4.) Needs to be pressed evenly against the subgrade by the armor layer

5.) Damage can be difficult to repair

6.) Difficult to identify exact location of geotextile failure.

7.) Careful laying of geotextile and subsequent placing of armor layer needed to avoid damage. A bedding layer between geotextile and armor layer may be needed

8.) Openings can become blocked

9.) It cannot be easily sealed to pile bents. If piles are already exposed, a slurry of riverbed material may be placed underneath the pile cap or Tremie concrete may be used.

Two forms of geotextile are used in scour protection, Woven and non-woven.

c. Woven

1.) They are formed using regularly placed fibers orientated at right angles to give uniform hole sizes.

2.) Woven geotextiles are generally stronger than non-woven and can be used as filters for soils of a particular size.

3.) They can also be appropriate where very high porosities are required.

d. Non-woven:

1.) Non-woven geotextiles are formed using randomly placed fibers, giving a range of hole sizes.

2.) They are generally considered more useful as filters in scour protection situations, because the hole opening sizes available cover a wider range of soil types.

3.) They can also stretch more before failure. By maintaining contact with the subsoil and the armor layer when stretched its filter function is not compromised.

2. Granular Filter

a. Advantages

1.) Deforms, so good contact is maintained between subsoil and armor layer

2.) Repairs are relatively easy and damage is sometimes self-healing

3.) Durable

b. Disadvantages

1.) Excavation may be required to lay it

2.) Accurate placing difficult in deep water

3.) Difficult to place in high currents

4.) Careful placing needed to achieve required thickness

5.) Grading needs to be carefully controlled

6.) Multiple layers may be needed to meet filter requirements

7.) Required grading may be difficult to obtain locally

3. Composite Filter

Advantages:

a. Can be useful in deep water and high current conditions where mattresses or sacks can be more readily placed than loose granular material or light geo-textiles.

b. Can be useful in protecting a geotextile from damage by large riprap.

4. Sand filled mats

These are geotextile mats filled with sand or fine gravel. The weight of sand ensures that mats can be laid without movement by currents, while the mat provides the required filter properties. The sand can act as a secondary filter.

11.5 DESIGN PROCEDURES FOR GEOTEXTILES

A filter is required unless the riprap lining has a thickness of at least 3 times the D_{50} size of the riprap.

1. **Functions**: The following functions should be considered in the design

a. Soil retention,

b. Permeability and

c. Strength

d. Soil retention is related to the size of pores or holes in geotextile (characteristic opening size).

e. For design guidelines see FHWA Publication HI-95-038 by Holtz D.H., Christopher B.R. and Berg R.R., 1995 “Geosynthetic Design and Construction Guidelines”, FHWA, Washington D.C.

2. **Design considerations**

   a. Anchorage: The area of the filter should be sufficient to allow for anchoring of the edges of filter.

   b. Open fabric: The selection of a relatively open fabric, which retains all sizes finer than the median size D_{50}, should be preferred. It will encourage the formation of a natural granular filter layer below the geotextile filter. It will increase the permittivity to allow release of pore pressure under flood conditions, without causing uplift of fabric.

   c. Layer Thickness: For large depths of armor a protection granular separation layer is required between the geotextile and armor layer. This will prevent tearing when stones are dropped.

   d. Material: It should be fabricated from ultraviolet light resistant material. The filter should have a life of at least 100 years without decay. The filter should be resistant to tearing or puncturing during armoring placement or settling.

   e. The Center for Civil Engineering Research & Codes 1995, CUR/RWS Report No. 169 suggests the following:

      1.) For Geo-textiles laid against non-cohesive, uniform soils

      2.) O_{95}/ D_{85} (base) < 1 where O_{95} is opening size where 95% of pores are smaller.

      3.) For Geotextiles laid against cohesive soils

      4.) O_{90}/ D_{10} (base) < 1.5 D_{60} (base)/ D_{10} (base)

      5.) O_{90}/ D_{50} (base) < 1

      6.) O_{90} < 0.5 mm

      7.) Where D_{15}, D_{50}, and D_{85} are the diameters of riprap and filter material of which 15, 50 and 85% are finer by weight. The base material may be used as the filter if it meets the above criteria.

      8.) The minimum sand gravel or stone filter thickness shall be 6 inches or 3 times the D_{50} size of the filter, whichever is greater.

      9.) Strength criteria: Reference shall be made to vendor's literature to determine the strength of geotextile to withstand the largest size of stone dropped onto it.

11.6 DESIGN PROCEDURES FOR GRANULAR FILTERS

1. Design considerations:

   a. Granular filters are normally designed using grading criteria derived from Terzaghi's filter rules. Various criteria have been developed. An important criterion is for the grading envelope to be approximately parallel to that of the soil.

   b. The thickness of each filter layer should be greater than 100 mm and 150 mm or D_{100} or 1.5 D_{50}, where one layer is used.

   c. Normally a thickness > 200 mm is required.

   d. The following criteria are recommended, based on CIRIA and CUR (1991) and CUR/RWS Report 169 (Center for Civil Engineering Research & Codes, 1995):

      1.) For uniformly graded material:

      For retention: $$\frac{d_{50\,filter}}{d_{50base}} \leq 5$$

      2.) For well-graded material:

      For retention: $$5 \leq \frac{d_{50\,filter}}{d_{50base}} \leq 20$$

      For retention: $$\frac{d_{15\,filter}}{d_{85base}} \leq 5$$

      3.) The criteria should be applied to the interface between the armor layer and the filter as well as to that between the filter and the base soil.

      4.) If the base material is gap-graded, then it should be considered as a mixture of two sub grading and the piping criterion should be based on the D_{85} of the finer of the two sub grading. This can be approximated to the D_{30} of the base material, so the piping criterion becomes as follows.

      5.) For gap-graded base material:

      For piping: $$\frac{d_{15\,filter}}{d_{30base}} \leq 5$$

      For all types of material to ensure adequate permeability:

For permeability: D15 (Coarser Layer) / D15 (finer Layer) < 40.

In addition, a uniformity criterion (also called a geometrically tight criterion) for the filter itself is to ensure that the finer particles of the filter are not removed through the voids between the coarser particles. This is particularly important where hydraulic loadings are high; such as in turbulent flow conditions.

a.) For uniformity: D_{15} (Coarser Layer) / D_{15} (finer Layer) < 40.

b.) CUR/RWS Report 169 (1995) suggests the use of a more rigorous uniformity criterion developed by Kennedy and Lau (1985):

$$(\frac{F_{4d}}{F_d} - 1)_{\min} > 2.3$$

where F_d is the percentage (by weight) of the filter finer than a particle size d and F_{4d} is the percentage (by weight) of the filter finer than a particle size of 4d. Different values of particle size d along the grain size distribution curve give different values of $((F_{4d}/F_d)-1)$. The minimum value of $((F_{4d}/F_d)-1)$ is at the flattest part of the grain size distribution curve.

6.) The thickness of each filter layer should be greater than 100 mm and should be at least 5.91 inches (150 mm) where only one layer is required. Normally, a thickness of at least 7.87 inches (200 mm) to 9.84 inches (250 mm) should be used. The layer thickness should also not be less than the D_{100} size or 1.5 times the D_{50} size of the filter layer. Where placed underwater or in high currents the layer thickness should be increased by about 50 per cent.

11.7 ALTERNATE TO GEOTEXTILE FILTER

1. A filter layer may be used in place of a geotextile filter, but in such case special care should be given to installation of both the filter layer and the gabion placement around the edges of the pier.

2. The granular filter layer may have the same cover as the baskets.

   D_{50} (Coarser Layer) / D_{50} (finer Layer) < 40. The ratio ensures uniformity criteria.

D_{15} (Coarser Layer) / D_{85} (finer Layer) < 5. The ratio ensures no erosion or piping through the filter

D_{15} (Coarser Layer) / D_{15} (finer Layer) < 40. The ratio ensures adequate permeability for structural bedding.

To satisfy permeability requirements the following criterion is suggested:

$$\kappa_g \geq M\kappa_s$$

where κ_s (in m/s) is the permeability of the soil, κ_g (in m/s) is the permeability of the geotextile and *M* is a coefficient which depends on the type of geotextile:

M= 10 for woven.

M= 50 for non-woven.

3. The strength criterion is based on the need to avoid damage from the armor layer being placed onto the geotextile. Where the armor layer is particularly large, it is normally preferable to lay a granular separation layer between the geotextile and armor layer to provide the dual function of protecting the geotextile and acting as a filter. Reference should be made to manufacturers' literature to determine the appropriate strength of a geotextile to cope with different sized stone being dropped onto it.

4. In the case of gravel riverbed these criterion are satisfied by gravel and filter is not required.

11.8 CONSTRUCTABILITY OF GEOTEXTILE FILTER

1. Construction conditions:

   It may be done in difficult site conditions, such as

   a. Limited access

   b. Fast flowing water

   c. Environmental hazards from synthetic material to the growth of vegetation, microorganisms and invertebrates.

   d. Difficulty of underwater construction such as limited working hours, construction in non-flood seasons, and use of divers

e. Expensive dewatering

2. Sealing of Geotextile Filter:

   a. It is recommended that the cover of the filter be somewhat less than the cover of riprap or gabion baskets. The cover of filter should extend a width D from each face of pier.

   b. The best performance may be obtained by sealing the filter to the pier. Sealing can be implemented by means of a flexible tube containing a cable that can be tightened around the pier by hooks. The hooks are manipulated from the bridge deck by a crane. The tube may be filled with ballast to hold the outer edge of the filter down. The flexible tube is attached to the geotextile.

   c. Alternately, sealing can be implemented by installing a granular filter layer around any gap between the geotextile filter and the pier itself.

   d. Good contact is required between riverbed and the filter, free from stones or holes.

   e. Geotextile should be fixed with stakes or pins to avoid movement of the layer while placing armoring.

   f. For underwater construction the geotextile layer should be submerged and the leading edge weighted down with sandbags or stones. Divers can then unroll the layer. If the depth of water is high a pontoon may be used for unrolling the layer in the direction of flow.

3. Storage:

Geotextile suffer ultra-violet deterioration. It should be covered when stored and when laid should not be left exposed for long periods.

4. Joints:

The joints should be lapped by at least 12 inches, with the lap laid so that it closes under the action of predominant flow.

5. Alkalinity:

In high pH conditions >10, poly propylene is preferred to polyester, owing to high resistance to chemical attack.

11.9 CONSTRUCTABILITY OF GRANULAR FILTER

1. Armoring should be placed carefully on granular filters to avoid displacement of material.

2. Underwater installation of granular filter layers can be difficult

3. Dunes inside the riverbed may make this type of filter ineffective.

11.10 COST

1. At a small additional cost the armoring countermeasure system can be made very effective and durable.

2. The cost of material and installation for geotextile is two to three times for providing granular filter.

3. An average cost for providing a geotextile layer is $10,000 to $15,000 for a pier size 4 feet wide and 20 feet long. The cost may vary for each county and water depth.

11.11 CONSTRUCTION SCHEDULE

Construction should be scheduled during off peak flood season.

11.12 VENDOR DETAILS

Details from Macafferri and other manufacturers are provided in Appendix.

An approved equivalent product may also be used.

11.13 MONITORING AND INSPECTION

1. Geotextile requires monitoring more than the granular filter.

2. After each flood and for each two-year cycle, any rupture of textile layer should be investigated. This may become obvious at the edges if riprap have moved.

SECTION 3

SELECTED RIVER TRAINING

AND

STRUCTURAL COUNTERMEASURES

CHAPTER 12

RIVER TRAINING COUNTERMEASURES

12.1 GENERAL

River training and flow altering countermeasures are recommended as secondary scour countermeasures when used in combination with primary armoring countermeasures.

River Training Countermeasures

River training structures are those which modify a river's 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. Some of the common types of river training countermeasures are:

1. Retard (earth, timber and steel sheet piles)
2. Channel Improvements (channelization)
3. Guide Banks/ Guide walls
4. Groyne (spur/dike/deflector)
5. Grade Control Structure/Check dams
6. Collars

River training structures are described as transverse, longitudinal or areal depending on their orientation to the stream flow.

1. Transverse river training structures are countermeasures which project into the flow field at an angle or perpendicular to the direction of flow. Groynes are transverse river training structures constructed from stone, earth, sheet piling or timber cribwork and extend out into the channel from a bank that is at risk of erosion. They are most commonly used on wide braided or meandering channels. They are less suitable for use where the channel is less than 131-164 ft (40-50 m) wide and where bend radii are less than 328 ft.

2. Longitudinal river training structures are countermeasures that are oriented parallel to the flow field or along a bank line. They use erosion protection systems that include riprap, gabion mattresses, concrete blocks (interlocking or articulated), sheet piling and bioengineering solutions using soil reinforcement and vegetation cover.

3. 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. Example of Areal River Training countermeasures are vertical (bed elevation control) countermeasures; such as, sills or weirs.

Flow Altering Countermeasures

These types of countermeasures are recommended for diverting scour away from bridge piers and should be used in combination with other scour countermeasures, such as riprap, gabions, etc. Sacrificial Piles, Upstream Sheet Piles, Collars and Horizontal Plates, Flow Deflecting Vanes or Plates, Modified Pier Shape or Texture and Slots in Piers and Pier Groups are examples of flow altering countermeasures. Among these flow altering countermeasures, only sacrificial piles and upstream sheet piles have been found to be effective in reducing scour at bridge piers and are recommended for applications in New Jersey.

1. Sacrificial Piles: Sacrificial piles are only recommended where the flow is likely to remain aligned with the pile or pier arrangement and for relatively low flow intensities (that is, under clear-water scour conditions). A recommended configuration in which piles are placed in rectangular configuration upstream of piers is shown in Figure 12.1.

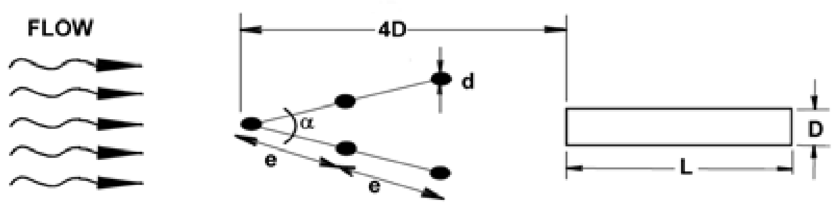


Figure 12.1a. Plan of Upstream Sacrificial Piles

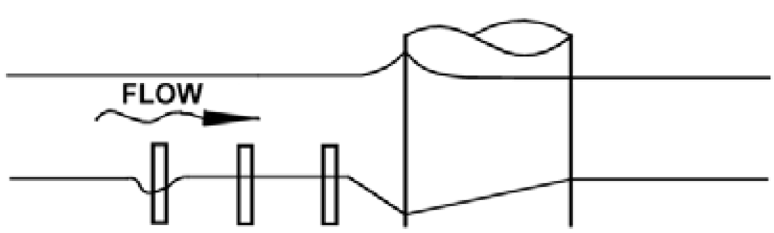


Figure 12.1b. Elevation of Upstream Sacrificial Piles

2. Upstream Sheet Piles: Upstream sheet piles are placed upstream of bridge piers to arrest scour in the lee of sheet pile. Figure 12.2 shows the recommended configuration of sheet piles. The width of sheet piles should be equal to the width of bridge pier and they should protrude only one third of the depth above the river bed.

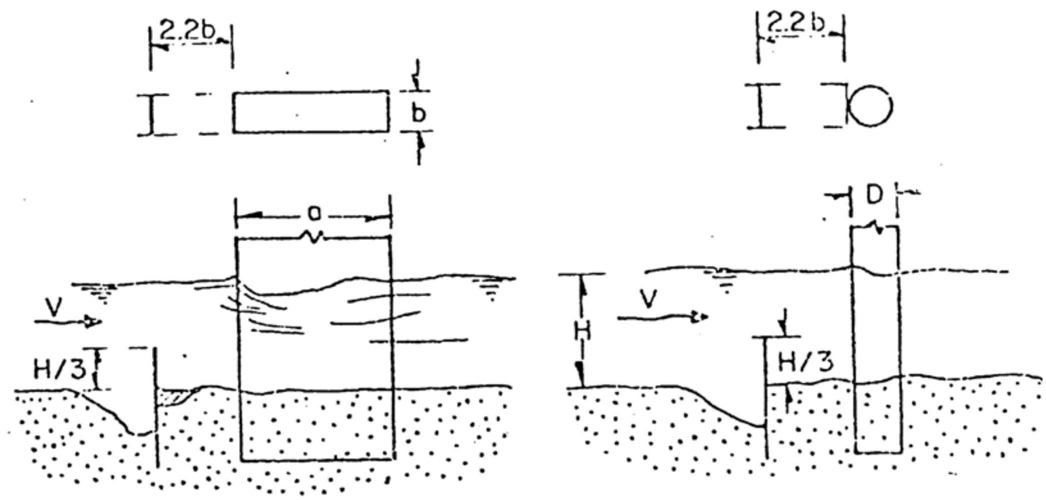


Figure 12.2. Installation Layout of Sheet piles upstream of rectangular and circular piers.

12.2 SELECTION OF RIVER TRAINING COUNTERMEASURES

The selection of types of river training measures will be based on the following considerations:

1. Flood velocity, medium or high
2. Flow conditions, overtopping or over bank
3. Perennial or seasonal
4. Type of scour, local or contraction, aggradation or degradation
5. Width of waterway, narrow or wide
6. Span length, medium or long
7. Stream alignment, straight, meandering or braided

8. Environmental requirements

9. Past experience of successful applications

Descriptions of river training countermeasures, their use on the basis of scour types, advantages and disadvantages are discussed in Table 12.1. Recommendations for suitability to various conditions that are listed above are discussed under the “Remarks” column.

12.3 RECOMMENDED RIVER TRAINING COUNTERMEASURES FOR NEW JERSEY

Depending on flood conditions, the following types are recommended for application in New Jersey,

1. Retard (earth , timber and steel sheet piles)

2. Channel Improvements (channelization)

3. Guide Banks/ Guide walls

The final selection will be made on project specific conditions.

Table 12.1. Comparison of River Training Measures

Countermeasure	Scour Type	Description	Advantages	Disadvantages	Remarks
Retard (earth, timber and steel sheet piles)	1. Local scour 2. Meandering stream or shifting, thalweg	Permeable or impermeable structure parallel to banks, to reduce flow velocity and induce deposition	Suitable for maintaining channel alignment. Induce deposition.	Minimum disadvantages since piles are buried below river bed	Suitable for high flood Velocities.
Channel Improvements (channelization)	1. Contraction and local scour 2. Aggradation	Channel modifications to increase flow capacity and sediment transport, including dredging, channel clearing	Suitable for aggradation or if upstream / down-stream of bridge is clogged	Minimum disadvantages	River encroachment permit requirements apply.
Guide Banks / Guide walls	1. Local scour at abutments / channel braiding	Straight or outward curving earth structure /fill to form embankments upstream to align flow through the bridge opening and reduce abutment scour	Improves flow conditions, Moves point of local scour away from abutment. Prevents erosion by eddy action.	Minimum disadvantages	Suitable for wide rivers with high flood velocity, River encroachment and other permit requirements apply.
Groyne (spur / dike / / deflector)	1. Local scour 2. Upstream Lateral erosion and degradation	Impermeable or permeable structure, which projects into flow, to alter flow direction, reduce velocity and induce deposition	Suitable for containment of over bank flow and for braided streams. Proven effective	Does not prevent downstream lateral erosion of banks or degradation of channel Projects above river bed	Suitable for wide rivers with high flood velocities. River encroachment permit requirements apply.
Grade Control Structure / Check dams	1. Contraction and local scour 2. Degradation Aggradation and lateral erosion of banks	Low dam or weir made of concrete, sheet pile, mats, gabions constructed across the channel to form debris basin and provide vertical stability of stream bed	Suitable for high flood velocities.	Expensive to install since riprap is required downstream of grade control structure.	Difficult to meet environmental requirements in NJ Since fish passage is adversely affected
Collars	1. Local scour	Thin horizontal plate attached to base of pier to deflect flow away from sediment bed	Suitable for high velocity rivers & for long span bridges. Low cost and maintenance	Debris accumulation for small spans. Does not eliminate scour, not much experience	Not easy to construct

12.4 DESCRIPTION OF SELECTED TYPES

Retard (Earth, Timber and Steel Sheet Piles)

Retards are permeable or impermeable structures made of earth, timber or steel and placed parallel to banks to reduce flow velocity and induce deposition. They are suitable for rivers with high flood velocities for maintaining channel alignment and inducing deposition. Retards are suitable for reducing local scour in meandering streams or shifting thalwegs.

Channelization

Channelization involves Channel modifications to increase flow capacity and sediment transport, including dredging, channel cleaning (as discussed in Figure 12.3). Also, it is suitable for reducing contraction scour, local scour and aggradation. A detailed plan for Channelization should be prepared on the basis of river flow conditions, environmental considerations and costs.

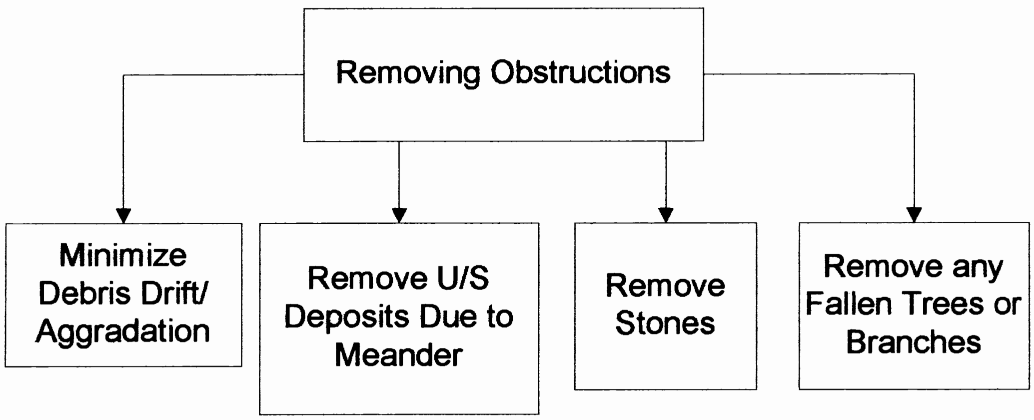


Figure 12.3. Procedures for Cleaning Up Channels

Guide Banks

A guide bank provides a smooth transition for flow on the flood plain to the main channel. Guide banks are earth or rock embankments placed at abutments to improve the flow alignment and move the local scour away from the embankment and bridge abutment.

When embankments span wide flood plains, the flows from high waters must be aligned so that they flow smoothly through the bridge opening. Over bank flows on a flood plain can severely erode the approach embankment and could increase the scour depth at a bridge abutment. Guide banks can be used to redirect flow from the embankment and to transfer scour away from an abutment. They can serve to reduce the separation of flow at the upstream abutment face, maximize the total bridge waterway area and reduce abutment scour by reducing turbulence at the abutment face [HEC-23].

Typically the length of a guide bank will be longer than the width of the bridge opening. The plan shape is usually elliptical, and is designed to provide acceptable flow alignment without flow separation. This requires long radius curves. The important factors for guide bank design are orientation relative to the bridge opening, plan shape, length (upstream and downstream of the abutment), cross-sectional shape, crest elevation, and protection of the structure from scour as shown in Figure 12.4. Protection from scour, usually by using riprap stone protection, on the flow facing side of guide banks is critical. The equations for sizing riprap for guide banks are [NJDOT Design Manual]:

$$d_r = 0.0282V^2 \qquad \text{for 1V : 2H side slope}$$

$$d_r = 0.0216V^2 \qquad \text{for 1V : 3H side slope}$$

$$d_r = 0.0418V^2 \qquad \text{for launching apron}$$

where V is the mean velocity of the approach flow.

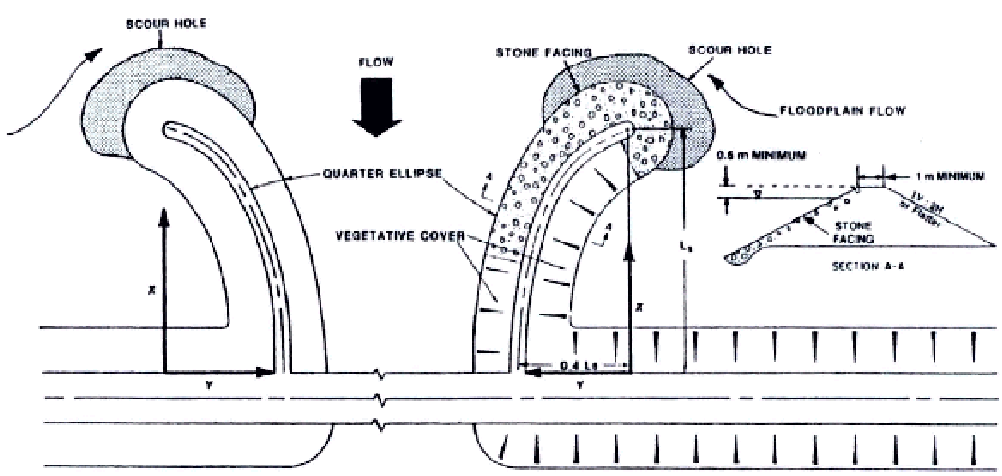


Figure 12.4. Layout and Configuration of Guide Banks.

GUIDE WALLS

Guide walls are similar to guide banks. Typical examples of Guide Walls are shown in Figures 12.5.

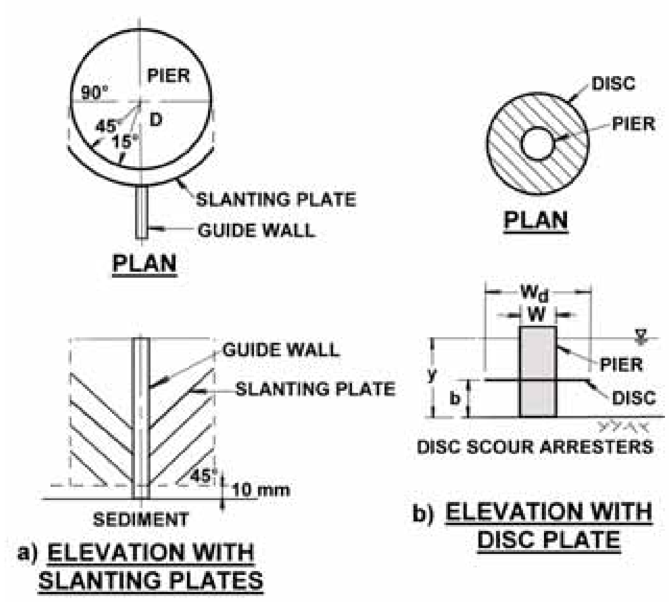


Figure 12.5. Plan & Elevation of (a) Guide Wall with Slanting Plate, (b) Disc Scour Arrestors.

CHAPTER 13

STRUCTURAL COUNTERMEAURES

13.1 INTRODUCTION

Structural countermeasures involve modification of the bridge structure (foundation) to prevent failure from scour. Typically, a bridge foundation is modified to increase its stability after scour has occurred or when a bridge is classified as scour critical. These modifications involve foundation strengthening, conversion from a simple span to continuous span configuration or pier geometry modifications.

1. Foundation strengthening includes additions to the original foundation, which will reinforce and/or extend the original foundation of the bridge. These countermeasures are designed to prevent failure when a channel bed is lowered to an expected scour elevation, or to restore structural integrity after scour has occurred.

2. Design and construction of bridges with continuous spans provide redundancy against catastrophic failure that is due to substructure displacement as a result of scour. Retrofitting a simple span bridge with continuous spans can also serve as a countermeasure after scour has occurred or when a bridge is classified as scour critical.

3. 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.

Structural countermeasures can also involve use of armoring countermeasures or use of no armoring countermeasures. This is generally used when scour is of a high magnitude. However, structural repairs such as grouting holes and cracks are generally required.   Commonly used structural countermeasures for existing bridges are:

1. Foundation Shielding such as constructing concrete apron/curtain walls

2. Sheet Piling local to foundations to act as shielding

3. Extended Footings

4. Mini piles driven through spread footings

5. Underpinning under the footing

6. Use of open parapet or railings to permit deck drainage of flood water.

General descriptions, advantages, disadvantages and applicability for a particular scour type for different structural countermeasures are presented in Table 13.1.

Recommendations for the application of a particular structural countermeasure for a given condition are presented in the "Remarks" column in Table 13.1.

13.2 STREAMLINING STRUCTURAL ELEMENTS FOR NEW BRIDGES

Streamlining structural elements is an effective approach to reduce scour at bridge piers and abutments by preventing vortex induced turbulent flows. However, this approach is suitable for new bridges only and is discussed in detail in Chapter 15. Following are examples of streamlining structural elements.

1. A sloping ("spill-through") abutment causes significantly less scour than a vertical wall abutment. In addition, angled wing walls (typically set at 30-75°to the longitudinal flow direction) or curved wing walls improve the hydraulic performance of vertical wall abutments.

2. Angled wing walls under normal circumstances are adequate and, where turbulence due to separation of flow is unlikely to be a significant problem, wing walls at 90° to the longitudinal flow direction are also acceptable.

3. Pier shapes: When a river may change its angle of approach over the life of the structure, the best hydraulic performance is given by rectangular piers having a wedged-shaped nose (known as "cutwaters"). Circular piers or a series of circular piles with a pile cap above water that support the piers may be more appropriate. Where debris accumulation is likely to be a problem, debris deflectors can be used

4. Overall structure alignment and alignment of elements: When a bridge deck could become submerged by an extreme flood (in excess of the design event), it may be appropriate to streamline the underside of the bridge deck by rounding the upstream and downstream faces to encourage passage of debris.

13.3 RECOMMENDED STRUCTURAL COUNTERMEASURES IN NEW JERSEY

Depending on flood conditions and overall site conditions, the following types of structural countermeasures are recommended:

1. Concrete apron/curtain walls

2. Local sheet piles

3. Extended Footings

4. Driving mini piles

5. Open parapets or railings

The final selection of a particular structural countermeasure must consider environmental considerations and cost. Figure 13.1 shows a flowchart for substructure repairs prior to installation of structural countermeasures. Illustrations of different structural countermeasures are shown in Figures 13.2 to 13.17.

13.4 EVALUATING (SCOURED) UNSUPPORTED PILE LENGTHS

Scour around piles leads to exposed pile lengths. This reduces axial capacity. The higher the scour, the lower the axial capacity. After a peak flood has exposed pile lengths, it is necessary to compute the axial capacity, the reduced factor of safety and the safety of the foundation. A pile program such as L-Pile may be used.

Table 13.1. Comparison of Structural Countermeasures

Countermeasure	Scour Type	Description	Advantages	Disadvantages	Remarks
Concrete Apron / curtain wall	Contraction and local scour	Concrete walls precast or cast in place against the sides of footing	New wall can rest on hard strata/rock.	Cofferdam is required for construction	Recommended
Local Sheet piles	Degradation	Piles driven as shields adjacent to bridge foundations to deflect flow	Suitable for high flood velocities. Stops flow, helpful in dewatering	Scour can occur near sheet piling, construction difficult, rust	Recommended for high scour situations with riprap protection
Extended footing	Local scour	Cast wider concrete slab footing to prevent settlement	Suitable for low scour depths. Acts as curtain wall/ apron on side of spread footing	Not suitable for masonry Footings. Bridge may be closed to traffic during construction	Recommended for concrete Spread footings
Constructing mini piles through spread footings	Degradation	Piles of small lengths driven through	Commonly used for footing strengthening	Expensive. Not suitable for old masonry footings	Not recommended for high traffic volume bridges
Under-pinning	Contraction scour Local scour	Lowering the bottom of footing elevation below scour depth	Commonly used for extensive repair or footing strengthening	Expensive. Not suitable for old masonry Footings. Bridge needs to be closed to traffic. Disturbance of streambed during construction.	Not recommended for high traffic volume bridges
Use of open parapets or railings	Contraction scour / pressure flow	Increases flow area and prevents overtopping of flood water	Effective for small openings or where vertical alignment is limited	Additional overflow downstream needs to be checked	Recommended only for overtopping flood situation
Relief bridge	Contraction and local scour	Approach bridge to increase size of waterway opening	Flood water will be discharged rapidly	Expensive. The key scour problem at main bridge may still remain unchanged	Not recommended since utilities need to be relocated

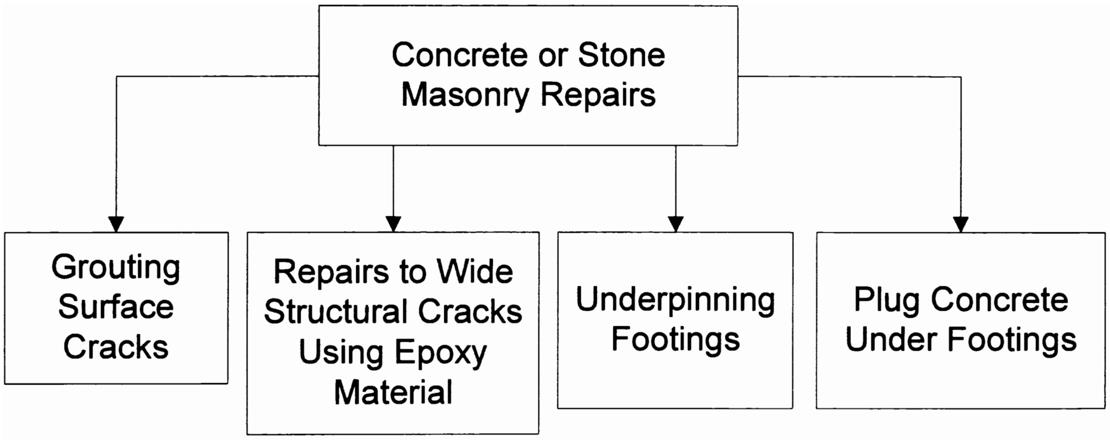


Fig. 13.1. Substructure Repairs Prior to Installing Structural Countermeasures.

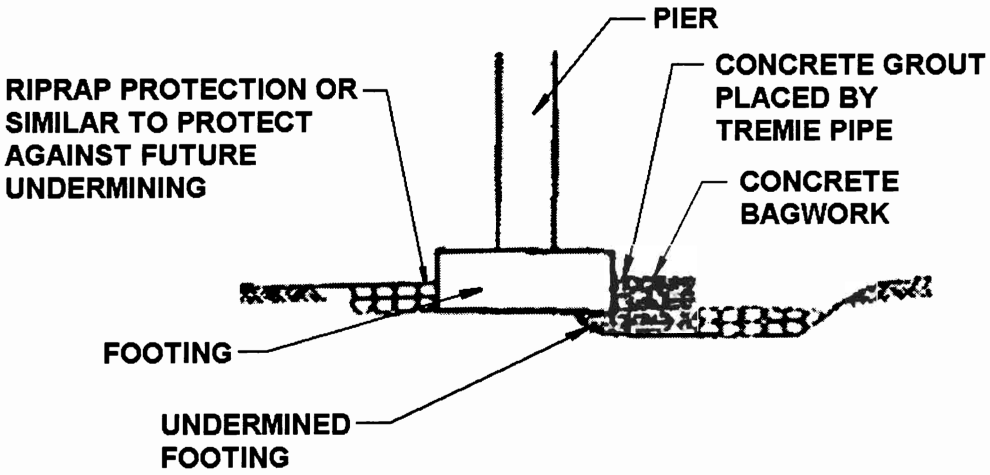


Figure 13.2. Structural Repairs – Grouting with Tremie Concrete
N.T.S

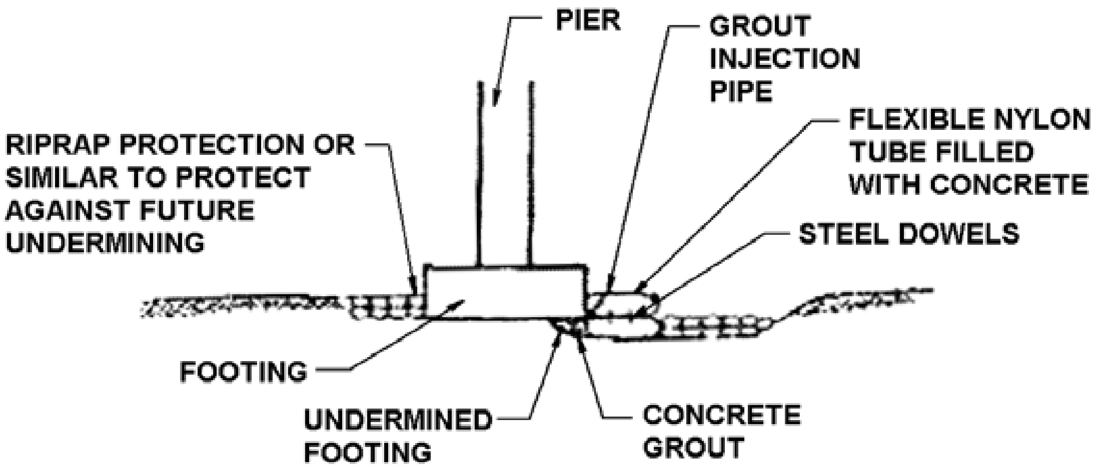


Figure 13.3. Structural Repairs – Grouting with pipe injection of concrete.

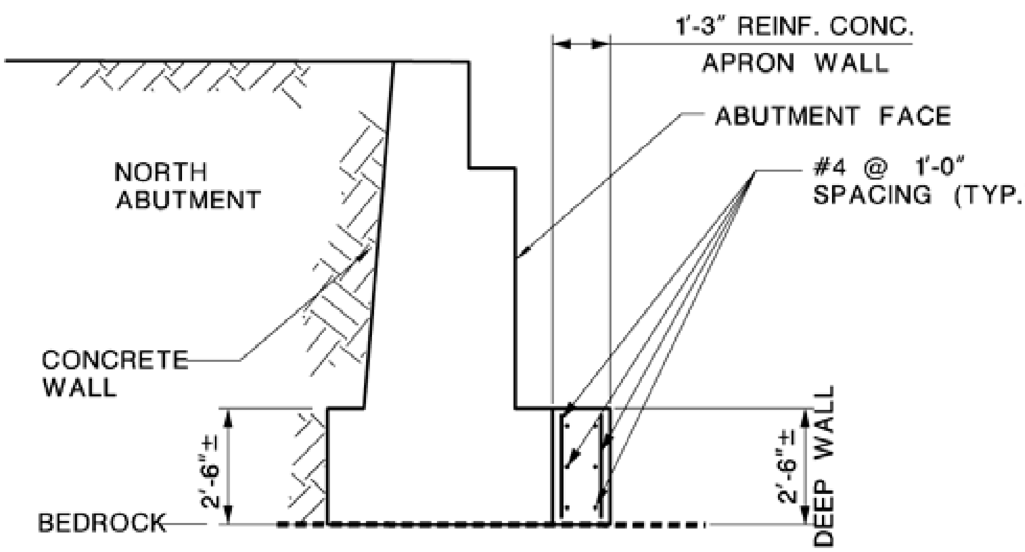


Figure 13.4. Concrete Apron Wall, Details at Abutment
N.T.S

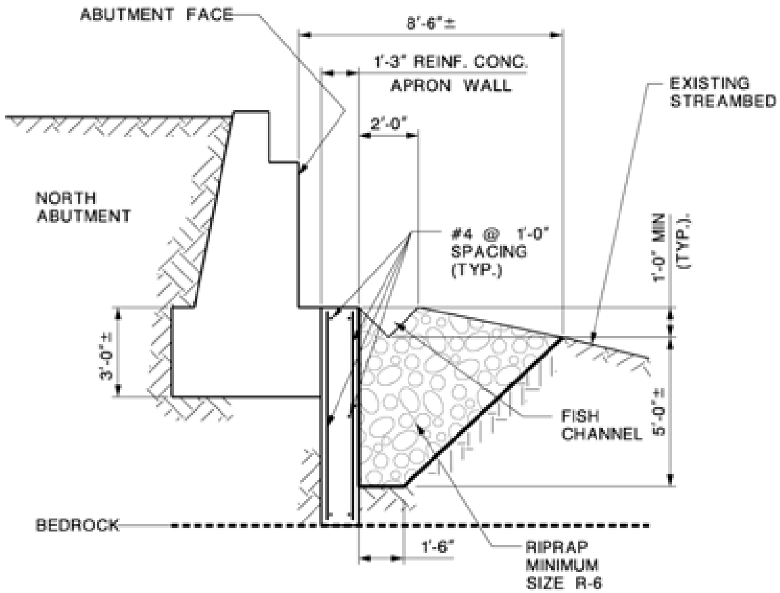


Figure 13.5. Concrete Apron Wall and Riprap, Details at Abutment
N.T.S

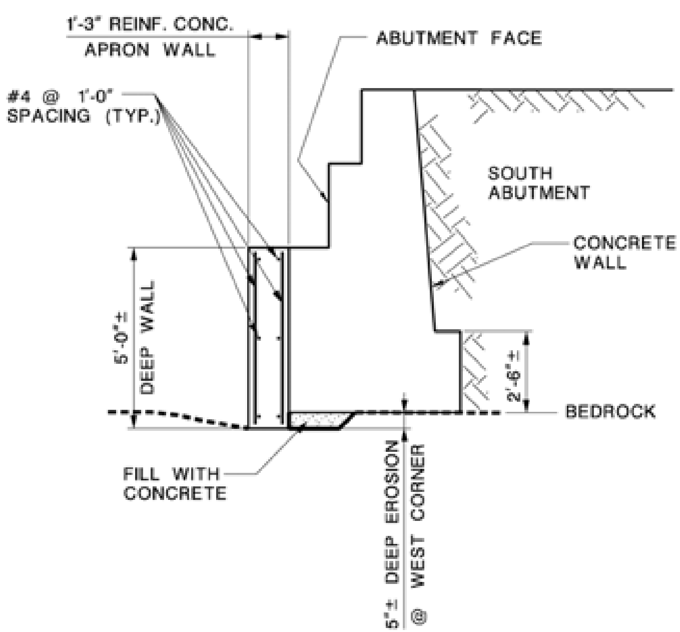


Figure 13.6. Concrete Apron Wall and Grouting of Scour Hole
Details at Abutment (N.T.S)

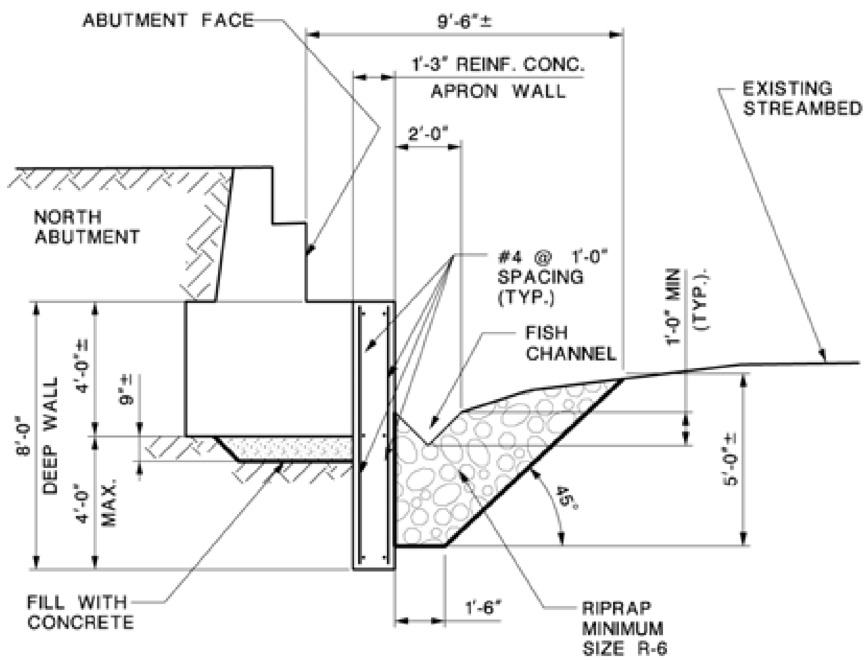


Figure 13.7. Concrete Apron Wall, Grouting Scour Holes with Armoring (N.T.S)

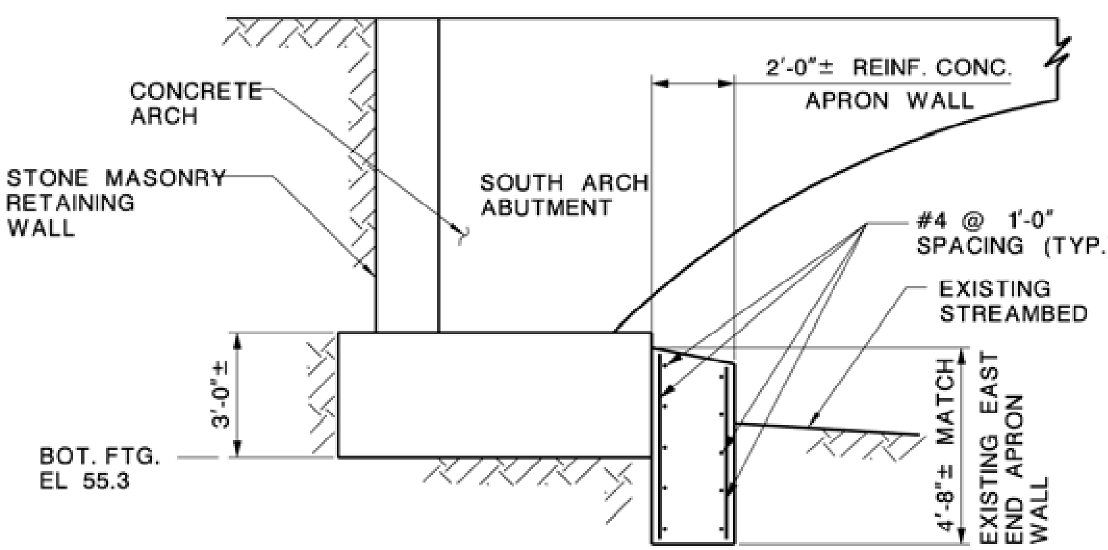


Figure 13.8. Concrete Apron Wall, Details at Abutment (N.T.S)

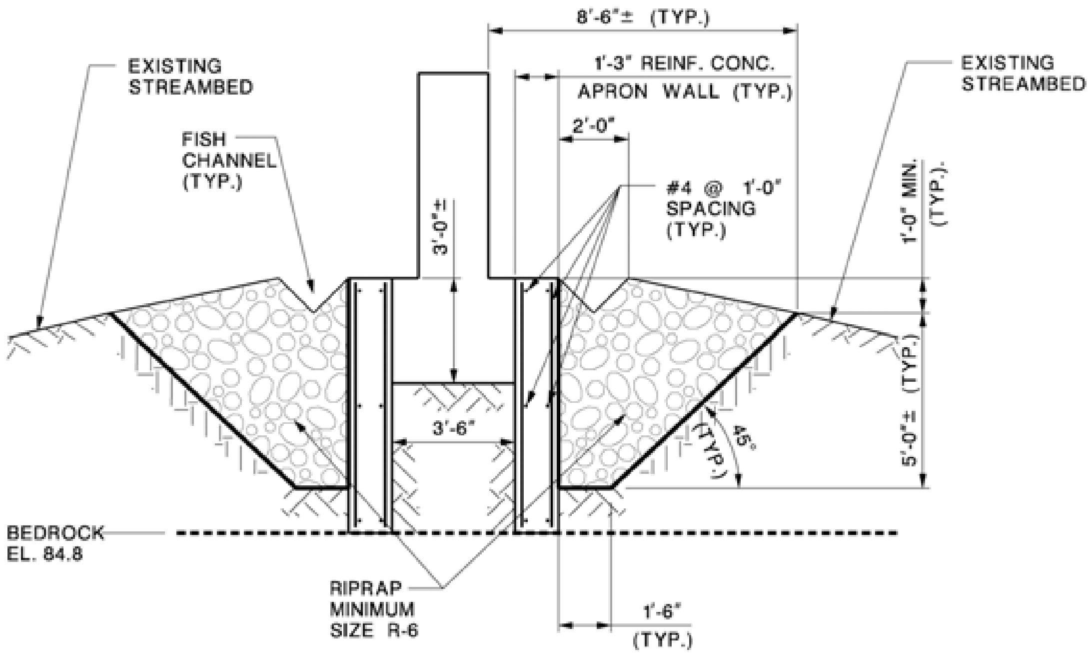


Figure 13.9. Concrete Apron Wall and Riprap, Details at Piers (N.T.S)

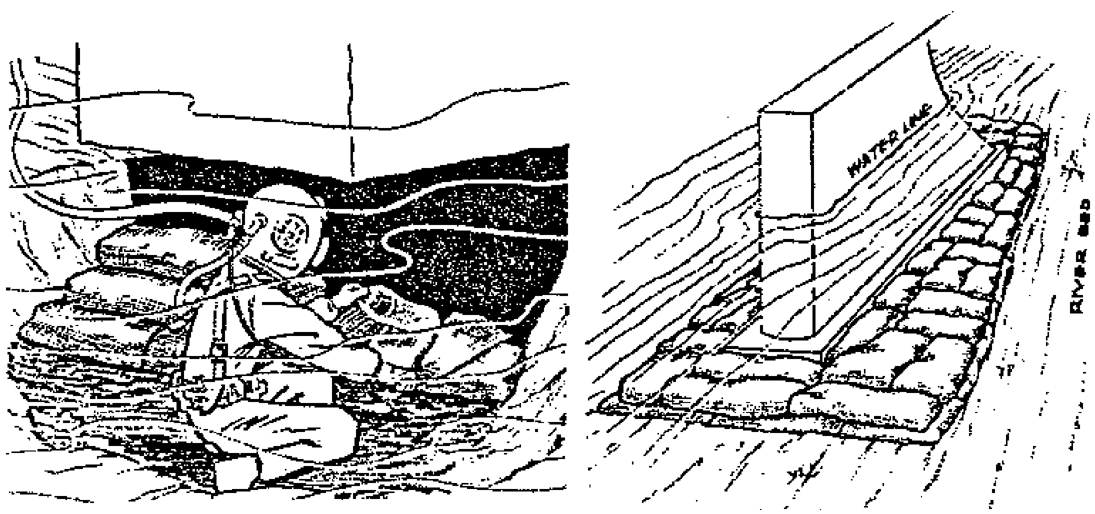


Figure 13.10. Underpinning With Replaced Aggregate and Pressure Grouting, Cast-in-Place Concrete or Concrete Filled Fiber Bags. (N.T.S.)

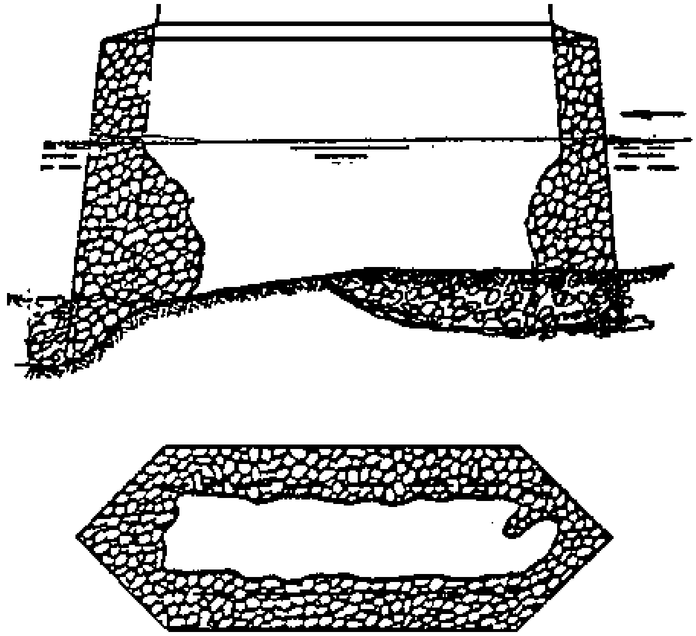


Figure 13.11. Jacketing the Foundation By Deepening and Jacketing With Concrete (N.T.S.)

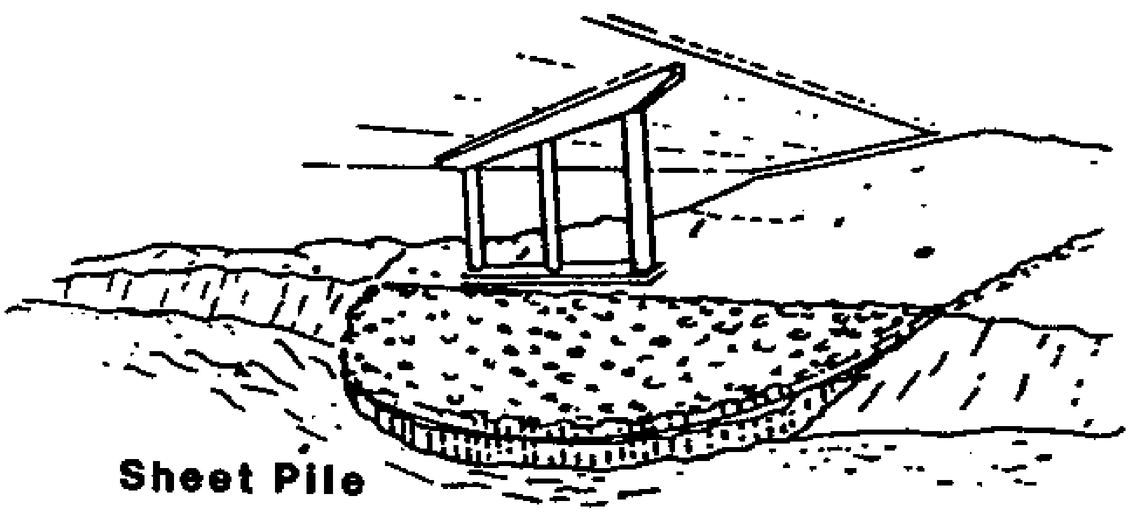
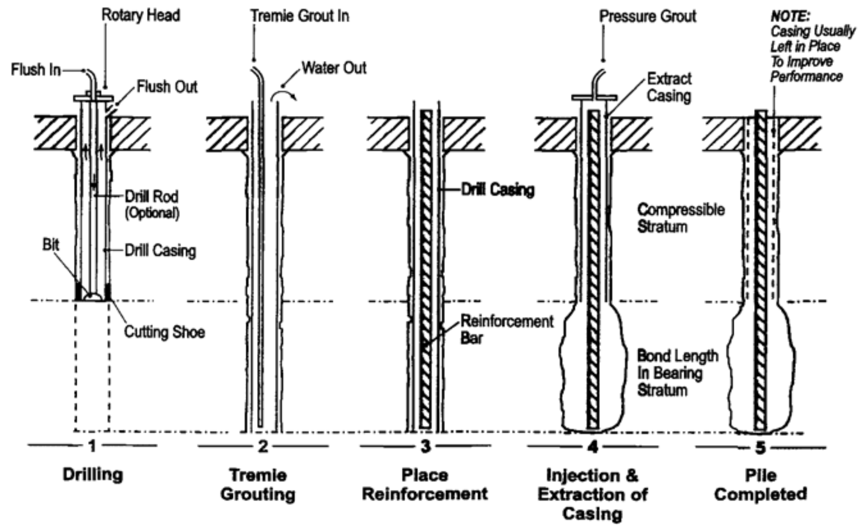


Figure 13.12. J Sheet-pile Cofferdam Enclosures and Cutoff Wall (N.T.S.)

PIN PILES℠



Typical construction steps for PIN PILES℠ in soil

Common Uses of PIN PILES℠

- To replace deteriorating foundation systems.
- To provide extra support for structures during renovation.
- To provide pile foundations where access, geology or environment prevent the use of other methods.
- To support structures affected by adjacent excavation, tunneling or dewatering activities.
- To provide a fast, effective alternative to more traditional underpinning methods.

Benefits of PIN PILES℠

- Can be installed through virtually any ground condition, obstruction and foundation and at any inclination.
- Ensure minimum vibration or other damage to foundation and subsoil.
- Can be installed in as little headroom as 6′ and close to existing walls.
- Allow facility operations to be maintained during construction.
- Resist compressive, tensile or lateral loads, or combinations of all three.
- Provide impressively high load capacity at extremely low total and permanent settlement.
- Simple and economical connection to existing and new structures.
- Can be preloaded to working load before connecting to particularly sensitive structures.

Figure 13.13. Uses and Advantages of Pinpiles as Structural Countermeasure.

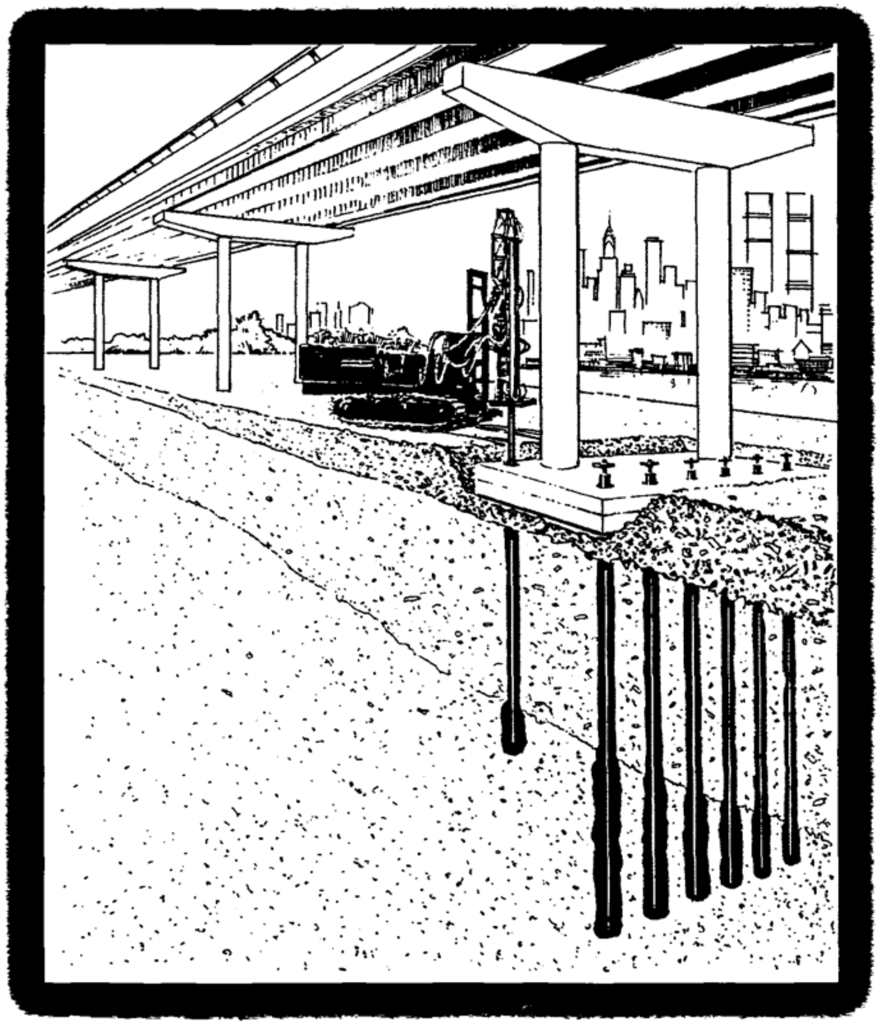


Figure 13.14. Restoration of Bridge Footings using Pinpiles .

Figure 13.15. Exposed Pile Bents at Peckman's River Bridge on Route 46.

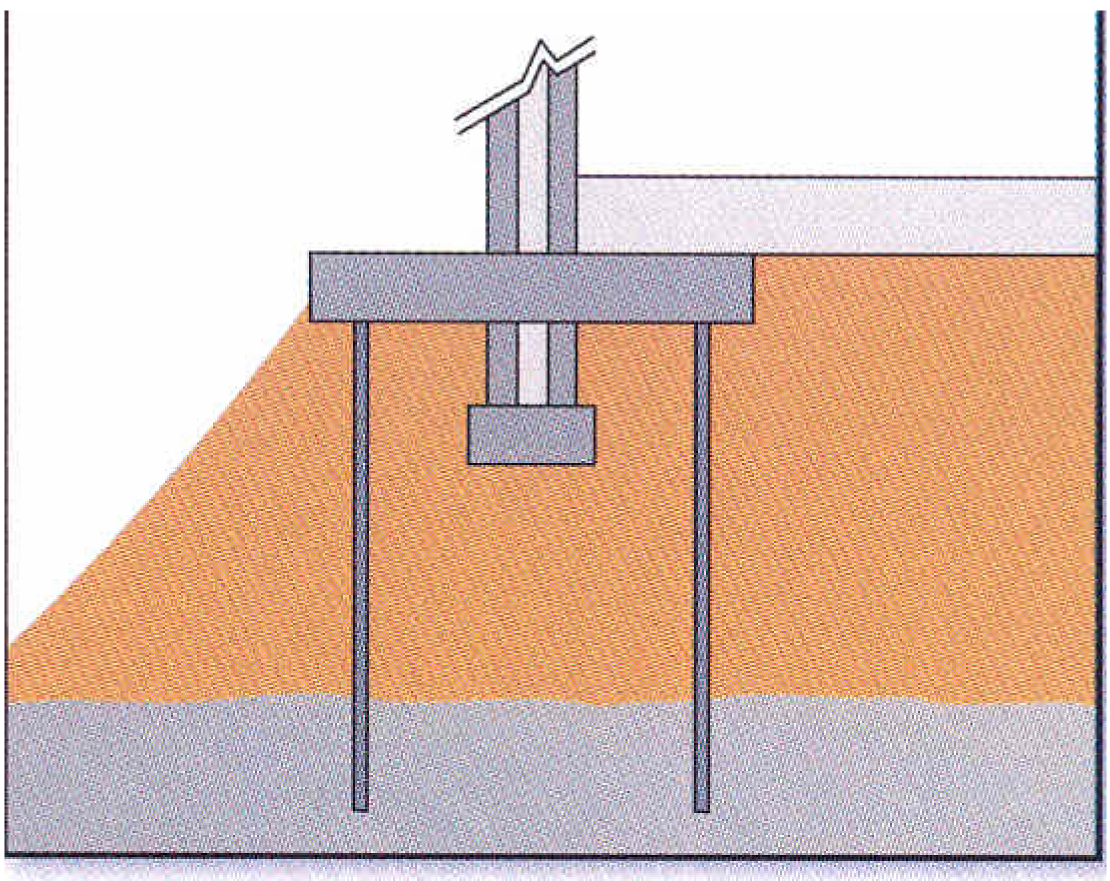


Figure 13.16. Underpinning method using mini piles and additional pile cap.

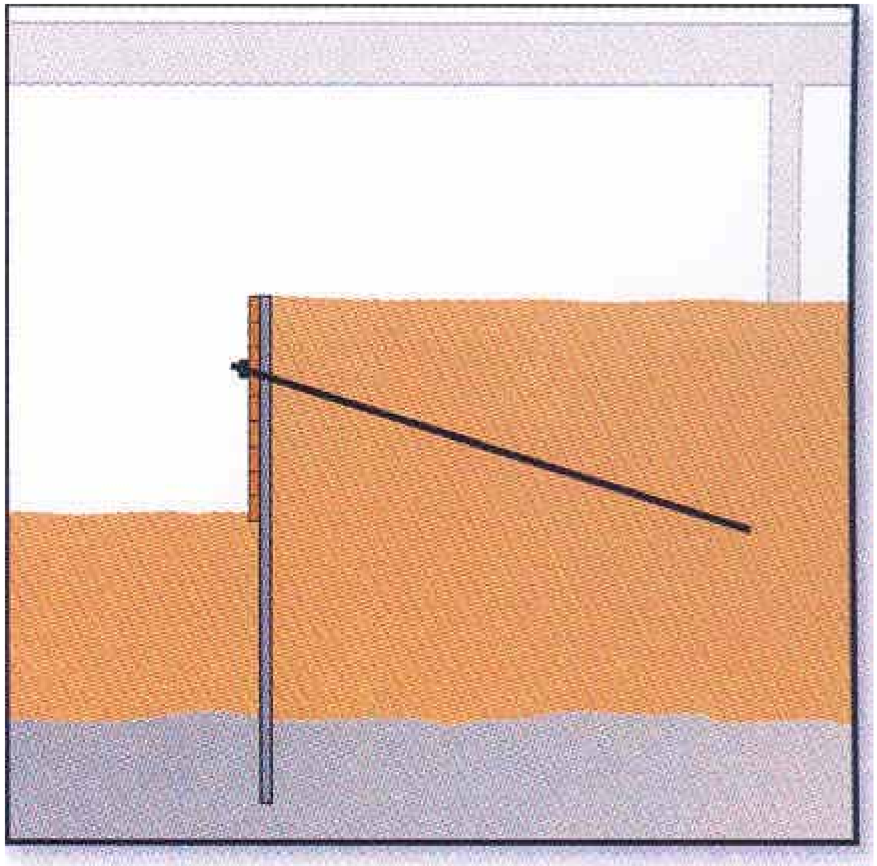


Figure 13.17. Timber boards spanning across driven minipiles to stabilize soil against future erosion

SECTION 5 – POST DESIGN REQIREMENTS

CHAPTER 14

MONITORING AS EFFECTIVE COUNTERMEASURE

14.1 MONITORING AND INSPECTION

Scour critical bridges typically require maintenance. Since streams are dynamic, and many bridge protection measures include living plants and biodegradable material, the potential for stabilization measures to change or deteriorate over time and through flood events is high. Such changes can only be corrected through an adaptive management program that is based on monitoring. Annual or biannual monitoring should be developed for scour-critical bridges to identify any potential bridge scour problems before they develop. Typical issues that require monitoring include:

1. Erosion to bridge footings
2. Performance assessment of countermeasures
3. Migrating meander forms
4. Adjustments to water and/or sediment supply from upstream
5. Impacts to vegetation survival from on-site land use
6. Head-cuts from downstream activities.

A well designed scour monitoring program will provide the following advantages:

1. It can be used as an effective tool to facilitate early identification of potential scour problems and provide a continuous survey of the progression of scour around bridge foundations.
2. Remedial action can be taken to offset a scour event.
3. If monitoring indicates that a bridge protection countermeasure is no longer functioning, then adjustments can be made to ensure the continued long-term function of the countermeasure. Such maintenance is called "adaptive management" because it is geared to identifying, over time, what countermeasures are best at providing functional applications for bridge scour, while minimizing impacts to fish and wild life.
4. Monitoring of scour countermeasures, such as dumped riprap, gabions, grout bags, etc. on regular intervals ensures that they are functioning as designed and facilitates the development of detailed field knowledge of their long-term performance.

14.2 SCOUR MONITORING PROGRAM

After installation of a scour countermeasure, a monitoring and inspection program, that includes obtaining scour depth measurements, an inspection process and traffic closures should be developed. An effective monitoring program should include the following considerations.

1. A monitoring plan may have some of the following objectives:

    a. Developing a database of photographic records of one or more constant points above and below flow depths for scour critical bridges

    b. Measuring bank and channel cross-sections and bed elevations

    c. Measuring scour depths regularly

    d. Measuring lateral migration

    e. Identifying eroded areas around the footings after major floods.

    f. Measuring plant densities and species composition

    g. Estimating fish use.

2. A project site should be monitored periodically (two or more times a year) during the first few years to ensure the success of the countermeasures and any compensatory mitigation. During this period, vegetation is re-establishing and the protection measures are less tested. This is especially important where the bridge protection measures rely heavily on plants to provide long-term stabilization. After vegetation has been established, monitoring once a year, or every other year may be adequate to ensure that the selected countermeasure functions as designed identifies any “obvious or apparent” channel response impacts and addresses any potential risks.

3. Upstream development should be monitored to ensure that bridge protection measures do not fail. Bridge scour problem should be managed through planned, integrated responses. This will avoid or minimize the need for “reactive” bridge repair projects.

14.3 TYPES OF MONITORING

Monitoring can be accomplished using instrumentation or visual inspection. A well-designed instrumentation program for scour monitoring can be a very cost-effective countermeasure. Two types of instrumentation can be used to monitor bridge scour: fixed instruments and portable instruments.

Fixed Instrumentation

In this approach, monitoring devices are attached to a bridge structure to detect scour at a particular location. Typically, sensors capable of measuring quantities, such as scour depth are located at piers and abutments. The number and location of piers to be instrumented by these sensors should be determined on the basis of scour and flood conditions, past visual inspection records, and relative importance of the bridge. Advanced sensors such as sonar monitors can be used to provide a timeline of scour, whereas magnetic sliding collars can only be used to monitor the maximum scour depth. Data from sensors can be stored in a datalogger on site and can be retrieved manually at fixed intervals or they can be transmitted to a remote computer system through wireless network.

Installation of instrumentation to monitor scour are discussed in "Instrumentation for Measuring Scour at Bridge Piers and Abutments", by Richardson E.V. & Lagasse, P.F., Final Report, Phase III, NCHRP Project No. 21-3, Transportation Research Board, Washington, DC, 1994.

Newly developed sensors capable of measuring different quantities related to bridge scour progression should be investigated through field tests. These may include sonar devices, sounding devices, buried electro-mechanical devices or tethered sensors.

Portable Instrumentation

In this approach, monitoring devices are carried manually and used along a bridge and transported from one bridge to another. Portable instruments are more cost effective than fixed instruments in monitoring an entire bridge. However, they do not offer a continuous monitoring over a span of several years.

The frequency of monitoring using portable instrumentation depends on the permitted level of risk for a particular bridge. Ground Penetrating Radar (GPR) is one of the portable instruments that have been used frequently to map river bed during peak floods.

Newly developed portable instrumentation can be used for scour monitoring, provided that the reliability of the instrumentation is verified by using visual monitoring or laboratory tests.

Visual Inspection

Visual Monitoring is a standard monitoring practice for inspecting bridges on a regular interval during low flow events and on a smaller interval (i.e., increased monitoring efforts) during high flow events. Typically, bridges are inspected on a biennial schedule where channel bed elevations at each pier location are measured. Periodic inspections, especially after major floods or coastal storm surges, should be carried out.

The normal 2-year inspection cycle with soundings (where required) for all scour-critical bridges should be performed.

Underwater inspections may be necessary for bridges whose substructure foundations cannot be visually inspected. Underwater inspections should be planned once every four years. The guideline on "Underwater Inspection and Evaluation of New Jersey Bridges" should be followed for underwater inspections. Figure 14.1 below shows the flow chart for visual inspection of bridges.

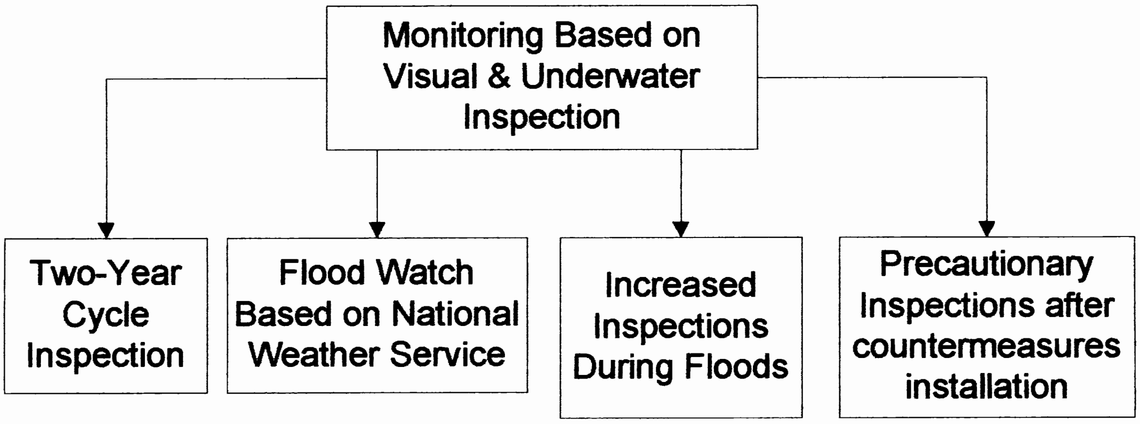


Figure 14.1. Flow Diagram for Visual and Underwater Monitoring Scour

In a visual inspection approach, 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. The channel bed elevations should be compared with historical cross sections to identify changes due to scour.

Flood Watch List

The flow diagram in Figure 14.2 shows the various steps leading to preparing a flood watch list for monitoring high risk (scour critical) bridges prior to floods.

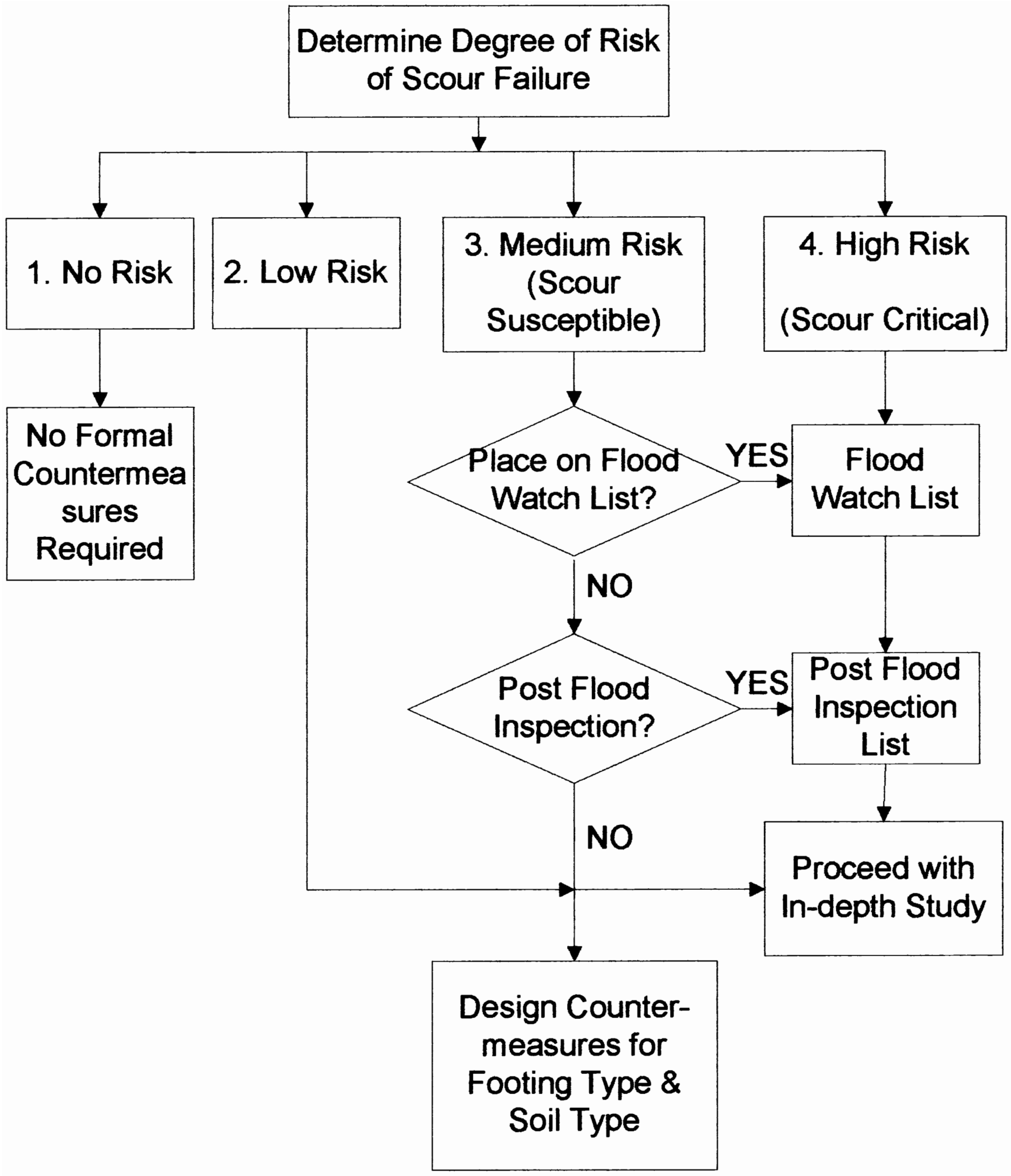


Figure 14.2. Flow diagram for Preparing Flood Watch List

CHAPTER 15

PLANNING FOR NEW BRIDGES TO PREVENT SCOUR

15.1 GENERAL

Since major costs are involved in planning new bridges, the following factors should be considered in the preliminary planning of a new bridge construction.

1. Carrying out a cost benefit analysis based on the importance of a bridge. (Analysis is to access what type of countermeasure?)

2. Deciding on a design return period other than 100 years and check return period of 500 years (super flood). (How could this be justified?)

3. Risk assessment (how could this be justified?)

4. Right of way and construction issues. (related to proposed countermeasures?)

Countermeasures are basically rehabilitation tools for foundations. Their use for new foundations should be limited. AASHTO and NJDOT bridge design codes do not recommend countermeasures over sound structural planning. Supplementary countermeasures, such as riprap, gabions, etc., are however optional. They may be used as a second line of defense against scour. To ensure the long-term safety of bridge foundations, scour countermeasures should be incorporated at the design stage as an integral part of a new structure. Hence, scour countermeasures for new foundations should be addressed through the following four planning approaches [CIRIA (2000)].

Hydraulic planning: The goal of scour reduction measures is to actively reduce scour around bridge foundations by improving flow conditions around the structure. Such hydraulic measures include:

1. Location of structure

2. Scour depth calculations using hydrologic and hydraulic analysis to design scour resistant foundations.

3. Streamlining structural elements

4. River training and deflectors to control stream instability. The measures include guide banks, check dams and spurs etc. and would reduce flood velocities or control river migration.

5. Avoiding the use of riprap for pier footings.

Structural Planning: Structural measures involve designing bridge foundations that will not fail during design flood events. This is the most important, safest and most reliable part of a scour countermeasure design for new bridges. The following factors should be considered in the structural planning of new bridges.

1. Locate bridges to avoid adverse flood flow patterns

2. Streamlining bridge elements to minimize obstructions to flow.

3. Designing foundations to resist scour.

4. Designing bridge pier foundations to resist scour without relying on the use of riprap or other countermeasures

5. Designing abutment foundations on deep foundations or on rock, where practicable;

6. For spread footings on soil, placing the footing to minimize the scour hazard.

Scour Reduction Planning: The goal of scour reduction measures is to limit scour around the structure by shielding foundations by sheet piling and/or by other suitable countermeasures. This is done without relying on the use of riprap or other countermeasures. If used countermeasures are applied to structures after they are constructed, use of countermeasures is not of primary importance and should be considered as a supplement to structural planning.

Monitoring Countermeasure Planning: Monitoring allows for action to be taken before the safety of the public is threatened by the potential failure of a bridge. A well-designed monitoring program can be a very cost-effective countermeasure. Important factors to be considered in monitoring countermeasure planning are:

1. Regular maintenance based on inspections of foundations and after each major flood.

2. Scour evaluation and repairs by providing and maintaining as-built drawings, foundation details and hydraulic and geotechnical information.

3. Availability of as-built plans (depicting bridge layout, foundations, pile tip elevations, etc.), bridge soils and scour reports and other documented hydrologic and hydraulic design information in a permanent file for the use of bridge maintenance and inspection units. The information on design assumptions and site conditions can serve as base line data to evaluate future changes in a river channel and to determine if the changes could affect the safety of the bridge.

15.2 SCOUR REDUCTION PLANNING

Table 15.1 presents a brief description of several important factors that should be considered during the hydraulic design phase of the bridge to minimize the risk of scour. More detailed information on scour reduction measures can be found in CIRIA (2000), Lagasse, et al (2001), Przedwojski et al (1995) and May et al (2000).

Table 15.1. Scour Reduction Measures

Type	Issues to consider and measures to reduce scour
Location related	Avoid locating structures at a confluence of two or more channels Avoid locating structures at or near sharp bends; locations on straight reaches or gentle bends are preferred Locate bridge crossings at the head or apex of the alluvial fan Check channel stability using aerial and satellite photography, historic maps, changes in flow direction can increase scour significantly Consider the stability of the channel vertically and laterally. Physical modeling should be considered for major structures on alluvial rivers or channels, major tidal crossings and barrages and complex flow conditions that cannot be modeled using simple 1-D models or don't match with well researched scour problems. Consider river morphology: channel widening, realignment, and changes in agricultural practices that may reduce scour Analyze the effect of existing nearby structures or effects of removal of old structures on flow conditions around the bridge
Hydraulic design related Refer to Neil (1975), Farraday and Charlton (1983), Hamill (1999), USDOT (1970), Brown (1987) for detailed Hydraulic Design of Bridges	Size of waterway opening: consider construction cost versus scour risk Size and number of relief openings: Lack of relief openings may result in larger bridge crossing, increased water level on floodplain, scour of embankments parallel to flow, scour of abutments from turbulence and vortices generated by the interaction of main channel flow with returning floodplain flow. Determine the number of relief openings by hydraulic design. Consider scour countermeasures at relief openings. Consider scour due to changes in flow direction at low flows and flood flows Consider effect of operational requirements causing concentration of flow on one side of structure (e.g., opening of weir gate on one side) Overtopping of approach embankments: Lower level of approach embankment than bridge deck will avoid washout of the bridge, reducing scour and risk of failure. Consider effects of removal of downstream or upstream bed control Channel improvements
Streamlining structural elements	Abutments: Sloping ("spill-through") abutment causes significantly less scour than a vertical wall abutment. In addition, angled wing walls (typically set at 30-75° to the longitudinal flow direction) or curved wing walls improve the hydraulic performance of vertical wall abutments. Under normal circumstances angled wing walls are adequate and, where turbulence due to separation of flow is unlikely to be a significant problem, wing walls at 90°to the longitudinal flow direction are also acceptable. Pier groups

Pier shapes: The best hydraulic performance is given by rectangular piers having a wedged-shaped nose (known as "cutwaters"). Where the river may change its angle of approach over the life of the structure. Circular pier, or a series of circular piles with a pile cap above water supporting the piers, may be more appropriate. Where debris accumulation is likely to be a problem, debris deflectors can be used
Overall structure alignment and alignment of elements: Where the bridge deck could become submerged by an extreme flood (in excess of the design event) it may be appropriate to streamline the underside of the bridge deck by rounding the upstream and downstream faces to encourage passage of debris.

15.3 DESIGN OF FOUNDATIONS FOR NEW BRIDGES

For the design of new bridges, bridge foundation should be designed for potential scour by assuming that all streambed material in the computed scour prism has been removed and is not available for bearing or lateral support. Bridge foundation should be designed to withstand scour during floods equal to or less than 100-year flood, and should be checked to ensure that they will not fail during a superflood (of the order of 500 year flood event). All foundations should have a minimum factor of safety of 1.0 under the superflood conditions [HEC-18].

The procedure to compute the scour prism, which represents calculated scour conditions, should be based on procedures outlined in HEC-18. All foundations should be designed in accordance with the AASHTO LRFD Bridge Design Specifications. In case of pile foundation, the piling should be designed for additional lateral restraint and column action because of unsupported pile length after scour. In areas where local scour is confined to the proximity of the footing, the lateral ground stresses on the pile length, which remains embedded, may not be significantly reduced from the pre-local scour conditions.

Shallow Foundation

1. **Spread Footings On Soil, Sand and Silt**

    a. Insure that the top of the footing is below the sum of the long-term degradation, contraction scour, and lateral migration.

    b. Place the bottom of the footing below the total scour line.

    c. The top of the footing can act as a local scour arrester.

2. **Spread Footings on Hard Rock**

    a. Place the bottom of the footing directly on the cleaned rock surface. The rock surface should be highly scour resistant massive rock formations (such as granite).

b. Small embedment (Keying) should be avoided since blasting to achieve keying frequently damages the sub-footing rock structure and makes it more susceptible to scour.

c. If footings on smooth massive rock surfaces require lateral constraint, steel dowels should be drilled and grouted into the rock below the footing.

3. **Spread Footings on Erodible Rock**

   a. Weathered or other potentially erodible rock formations need to be carefully assessed for scour. An engineering geologist familiar with the area geology should be consulted to determine if rock, soil or other criteria should be used to calculate the support for the spread footing foundation. The decision should be based on an analysis of intact rock cores, including rock quality designations and local geology, as well as hydraulic data and anticipated structure life.

   b. An important consideration in the analysis would be to determine the existence of a high quality rock formation below a thin weathered zone. For deep deposits of weathered rock, the potential scour depth should be estimated and the footing base placed below that depth. Excavation into weathered rock should be made with care.

   c. If blasting is required, light, closely spaced charges should be used to minimize the break up beneath the planned footing level. Loose rock pieces should be removed and the zone filled with clean concrete. The final footing should be poured in contact with the sides of the excavation to the fully designed footing thickness to minimize water intrusion below the footing level.

   d. Guidance on scourability of rock formations is given in the FHWA memorandum “Scourability of Rock Formations” dated July 19, 1991.

4. **Spread Footings Placed on Tremie Seals and Supported on Soil**

   a. Insure that the top of the footing is below the sum of the long-term degradation, contraction scour, and lateral migration

   b. Place the bottom of the footing below the total scour line.

Deep Foundations

1. **For Deep Foundations (Drilled Shaft And Driven Piling) with Footings or Caps**

Placing the top of the footing or pile cap below the streambed to a depth that is equal to the estimated long-term degradation and contraction scour depth will minimize obstruction to flood flows and resulting local scour. Lower footing elevations may be desirable for pile supported footings when the piles could be damaged by erosion and corrosion from exposure to river or tidal currents.

2. **Stub Abutments on Piling**

Stub abutments positioned in an embankment should be founded on piling that is driven below the elevation of the thalweg. Long term degradation and contraction scour in the bridge waterway to assure structural integrity in the event the thalweg shifts and the bed material around the piling scours to the thalweg elevation should be considered.

15.4 PLANNING OF SUBSTRUCTURES AND SUPERSTRUCTURES

A brief description of various factors, including advantages and disadvantages, in the planning of substructures and superstructures of the bridge are presented in Tables 15.2 and 15.3, respectively. Some important guidelines for planning and design of superstructures and substructures (piers and abutments) on the basis of HEC-23 guidance are described in the following.

Superstructure

1. Raise the bridge superstructure elevation above the general elevation of the approach roadways wherever practicable. This provides for overtopping of approach embankments and relief from the hydraulic forces acting at the bridge. This is particularly important for streams carrying large amounts of debris which could clog the waterway at the bridge.

2. It is recommended that the elevation of the lower chord of the bridge be increased a minimum of 0.9 m (3 ft) above the normal freeboard for the 100-year flood for streams that carry a large amount of debris.

3. Superstructures should be securely anchored to the substructure if buoyant, or if debris and ice forces are probable. Further, the superstructure should be shallow and open to minimize resistance to the flow where overtopping is likely.

4. Continuous span bridges withstand forces due to scour and resultant foundation movement better than simple span bridges. Continuous spans provide alternate load paths (redundancy) for unbalanced forces caused by settlement and/or rotation of the foundations. This type of structural design is recommended for bridges where there is a significant scour potential.

5. At some bridge sites, hydraulics and traffic conditions may necessitate consideration of a bridge that will be partially or even totally inundated during high flows. This consideration results in pressure flow through the bridge waterway [HEC-23, Chapter 6].

Piers (Substructure)

1. Local scour holes at piers and abutments may overlap one another in some instances. If local scour holes do overlap, the scour depth becomes indeterminate and possibly deeper. The top width of a local scour hole on each side of a pier ranges from 1.0 to 2.8 times the depth of local scour. A top width value of 2.0 times the depth of local scour on each side of a pier is suggested for practical applications.

2. For pile and drilled shaft supported substructures subjected to scour, a reevaluation of the foundation design may require a change in the pile or shaft length, number, cross-sectional dimension and type based on the loading and performance requirements and site-specific conditions.

3. Pier foundations on floodplains should be designed to the same elevation as pier foundations in the stream channel if there is likelihood that the channel will shift its location over the life of the bridge.

4. Align piers with the direction of flood flows. Assess the hydraulic advantages of round piers, particularly where there are complex flow patterns during flood events.

5. Streamline piers to decrease scour and minimize potential for buildup of ice and debris. Use ice and debris deflectors where appropriate.

6. Evaluate the hazards of ice and debris buildup when considering use of multiple pile bents in stream channels. Where ice and debris buildup is a problem, consider the bent a solid pier for purposes of estimating scour. Consider use of other pier types where clogging of the waterway area could be a major problem.

7. Scour analyses of piers near abutments should consider the potential of larger velocities and skew angles from the flow coming around the abutment.

Abutments (Substructures)

1. Equations to calculate abutment scour in Chapter 7 of HEC-18 may tend to over estimate the scour depth because of lack of verification of field conditions. Recognizing this, the abutment scour equations are used to develop insight as to the scour potential at an abutment. Engineering judgment must be used to determine if the abutment foundation should be designed to resist the computed local scour.

2. As an alternate, abutment foundations should be designed for the estimated long-term degradation and contraction scour. Riprap and/or guide banks

should be used to protect the abutment for this alternative. In summary, riprap or some other protection should always be used to protect the abutment from erosion.

3. Relief bridges, guide banks, and river training works should be used, where needed, to minimize the effects of adverse flow conditions at abutments.

4. Where ice build-up is likely to be a problem, set the toe of spill-through slopes or vertical abutments back from the edge of the channel bank to facilitate passage of the ice.

5. Wherever possible, use spill-through (sloping) abutments. Scour at spill-through abutments is about 50 percent of that of vertical wall abutments.

6. Riprap or a guide bank 50 ft or longer or other bank protection methods should be used on the downstream side of an abutment and approach embankment to protect them from erosion by the wake vortex.

Guidelines for new foundations suggest the use of riprap as a supplementary countermeasure for the protection of new foundations (piers/abutment). Since riprap is considered as temporary countermeasure and is not recommended, alternatives to riprap should be considered for the protection of new foundations. If riprap is the preferred countermeasure, it should be used with prior excavation and with filter layer and only as a supplemental countermeasure.

15.5 DESIGN PROCEDURES TO MINIMIZE DIFFERENT TYPES OF SCOUR

The following guidelines may be used to minimize different types of scour in the design phase of new bridges.

Contraction scour

1. Use larger openings to allow for debris height accumulation.

2. Use longer span bridges, elevated decks, and crest vertical curves.

3. Use profiles for overtopping during floods and relief bridges

4. Use reduced superstructure depths. Use open spandrel parapets.

5. Place piers away from the thalweg of a river

6. Excavate waterways to remove debris from smaller floods

7. Use guide banks on upstream side to align flow in bridge opening

8. Use revetments on channel banks fill slopes at bridge abutments.

Local scour at abutments

1. Place foundations on sound rock
2. Use deep piling
3. Use stub abutments in lieu of full height
4. Use sloping walls in place of vertical walls
5. Use revetments (pervious rock or rigid concrete)
6. Use riprap on spill slopes
7. Use guide banks at abutment
8. Monitor and inspect after flood events

Local scour at piers

1. Place foundations in sound rock or below the total scour line
2. Use deep piling as foundations
3. Streamline pier noses (rounded shape)
4. Use pile bents or multiple columns with curtain walls to prevent debris deposit
5. Use riprap as a temporary measure
6. Cut cofferdams below contraction scour depths
7. Monitor and inspect after flood events

Aggradation

1. Use Debris Basins
2. Continual Maintenance Planning

Degradation

1. Use check dams or drop structures on small to medium streams
2. Channel lining
3. Use deeper foundations
4. Adequate setback of abutments

5. Use rock and wire mattress for small channels

River meander

Locate bridges on straight reaches of streams between bends.

Braided channels

Build one long bridge, more than one bridge or a relief bridge

Table 15.2. Substructure Planning for New Bridges

Substructure Components	Action	Advantages	Disadvantages	Supplementary Countermeasures
Bridge Location	On straight segments. Avoid bends and downstream of dam	Lateral meander of river is avoided. Increased scour if dam is breached.	Bridge may have sharp skew in plan to fit in the straight segment	Spurs on upstream side to retard flood flow, Guide banks on upstream side to align flow in bridge opening, Monitoring
Flow Direction	Align abutment and pier walls parallel to flow direction	Angle of attack is minimized. Local scour is minimum.	Bridge plan has sharp skew.	Monitoring
Abutment Location	Adequate setback of abutments	Degradation is minimum.	Increased cost of bridge	Bed armoring or sheet piles. Monitoring
Abutment Type	Use stub or Integral in place of full height.	Local scour is minimum. K2 = 0.5	None	Use sheet piles Monitoring
Vertical wall or Sloping Wall Abutment	Use sloping wall in place of vertical wall	Local scour is minimum.	Increased Cost of formwork	Monitoring
Abutment Foundation	Use deep piles in place of short piles	Minimum degradation, Loss of soil due to scour compensated by additional pile lengths.	Cost of piles increases	Sheet piles with riprap to protect scour of top of piles, Monitoring
Wing walls	Align with direction of flow	Local scour is minimum.	None	Sheet piles with riprap to protect scour of top of piles, Monitoring
Pier Location	Place piers way from thalweg of river.	Contraction scour is minimum. Pier height is reduced.	Survey of river profile is required	Use bed armoring or sheet piles. Monitoring
Pier Type	Use pile bents or multiple columns with curtain wall	To prevent debris deposit	None	Monitoring
Pier Shape	Use round or pointed shapes.	Local scour is minimum. K1 is lower.	Formwork cost increases.	Monitoring
Pier Foundation	Use deep piles in place of short piles	Loss of soil due to scour is compensated by additional pile lengths. Minimum degradation	Cost of piles increases	Use sheet piles with riprap to protect scour of top of piles, Monitoring
Spread Footing on	Place bottom of footing below total	Footing settlement is avoided	None	Bed armoring such as riprap or gabion

Soil	scour line.			required, Monitoring
Spread Footing on Weathered Rock	Determine RQD to estimate erodibility of rock	Allowance is made for scour of rock	None	Bed armoring such as riprap or gabion required, Monitoring
Spread Footing on Rock	Place bottom of footing on rock	Footing settlement is avoided	None	Monitoring
Bearings	Place bearings above M.W.L.	Rusting or damage to bearings is avoided	Height of bridge increases	Monitoring
Embankment	Protect embankment at upstream and downstream of bridge with armoring	Controlled flood plain width since bank erosion is avoided. Contraction scour is minimum.	Costly Protection of Sloping embankment	Revetments required. Monitoring

Table 15.3. Superstructure Planning for New Bridges

Superstructure Item / Component	Action	Significant Advantages	Significant Disadvantages	Supplementary Countermeasures
Length of Bridge	Bridge to span full flood plain width	Opening area increased. Relief bridge is not required. Contraction scour is minimum	Costs are high. Right of way issues to be resolved.	Use bed armoring or sheet piles. Monitoring
Number of spans	Prefer multiple spans to single span	Redundant load path. Safer bridge	Foundation cost is higher for multiple spans.	Use bed armoring or sheet piles. Monitoring
Length of Single span	Increasing the span	Opening area increased. Contraction scour is minimum	Bridge cost increases	Use bed armoring or sheet piles. Monitoring
Parapet Wall	Use open spandrel. Avoid solid parapet.	Increases flow during peak flood. Contraction scour is minimum	None	None
Parapet Railing	Avoid fence.	Increases flow during peak flood. Contraction scour is minimum	None	None
Deck Profile	Use sag vertical curve	Increased vertical clearance	Cost of approaches is increased	None
Girders	Use shallow depth	Increased vertical clearance	More girders required since girder spacing reduced.	N/A

15.6 GEOTECHNICAL CONSIDERATIONS

In addition to structural planning, supplementary scour countermeasures should be selected based on geotechnical considerations. Soil types are broadly classified as Non-cohesive (e.g., Gravels, Sands and Silts) and cohesive (Silts, Clays) materials. Non-cohesive sediments have a granular structure, with individual particles being susceptible to erosion when the applied fluid forces (drag and lift) are greater than the stabilizing forces due to gravity and cohesion with adjacent bed particles. The threshold of movement of particles of non-cohesive materials depends on particle size, density, shape, packing and orientation of bed material. Most fine-grained sediments possess some cohesion, the clay content being of great importance. Cohesive sediments typically require relatively large forces to detach the particles and initiate movement, but relatively small forces to transport the particles away. The bed material comprises of sediments (alluvial deposits) or other erodible material. If bed materials are stratified, a conservative approach needs to be adopted regarding the risks of the scour breaking through the more resistant layer into the less resistant layer.

Selection of supplementary countermeasures should be based on the applicability of supplementary countermeasures (countermeasures for existing bridges) to a particular soil type.

15.7 REQUIREMENTS OF NJDOT BRIDGE DESIGN MANUAL

Section 1.46.5 Subsection 4 lists guidelines for the design of bridge foundations for 3 types of soil conditions. These guidelines should be applied in combination with design guidelines provided in Section 15.3 herein.

Spread Footings on Soil

Place bottom of footings 3 feet below the total scour line. Although any minor surface erosion of soil will not cause a danger to the footings, soil cover or protection to the concrete footing or piles should still be provided for the following reasons:

1. Will provide frost resistance (minimum frost depth requirement)

2. Will maintain as-built cosmetic appearance

3. Will guard against any unforeseen error in the scour analysis data or computations.

A minimum 3 feet depth of riprap or an alternative countermeasure should be provided adjacent to the footings.

Spread Footings on Erodible Rock

Place the bottom of footing 6 inches below the scour depth. This provision is conservative compared to that for soil conditions in which scour depth may be reduced by 50%.

Spread Footings on Non-erodible Rock

This condition is less common in New Jersey. Guidelines in Section 15.3 herein should be followed for this case.

CHAPTER 16

ENVIRONMENTAL ASPECTS OF COUNTERMEASURE DESIGN

16.1 ENVIRONMENT ASSESSMENT

The following steps should be performed to minimize adverse environmental impacts:

1. A baseline survey to define the current environment
2. Assessment of impact of proposed countermeasures on the current river environment
3. Considerations of measures to avoid or mitigate the adverse impacts.

16.2 ENVIRONMENTAL CONCERNS

Environmental concerns that must be addressed include:

1. Stream Encroachment
2. Open Water Impacts / wetlands
3. Ecology (flora and fauna / fish passage)
4. Landscape / soil erosion, sediment transport, drainage
5. Minimizing impacts to natural vegetation by controlling construction access points.
6. Re-vegetation of disturbed areas with species.
7. Use of a 6 inches minimum layer thickness of native substrate cover over the proposed armoring countermeasure
8. Minimizing the erosion of native substrate due to sediment transport after the installation of countermeasures
9. Reactions with acid producing soils and air quality (contamination, pollution)
10. Noise, aesthetics and traffic disruption
11. Historical and cultural aspects
12. Socio-economic aspects, job creation

The flowchart in Figure 16.1 illustrates the NJDEP requirements on stream encroachment.

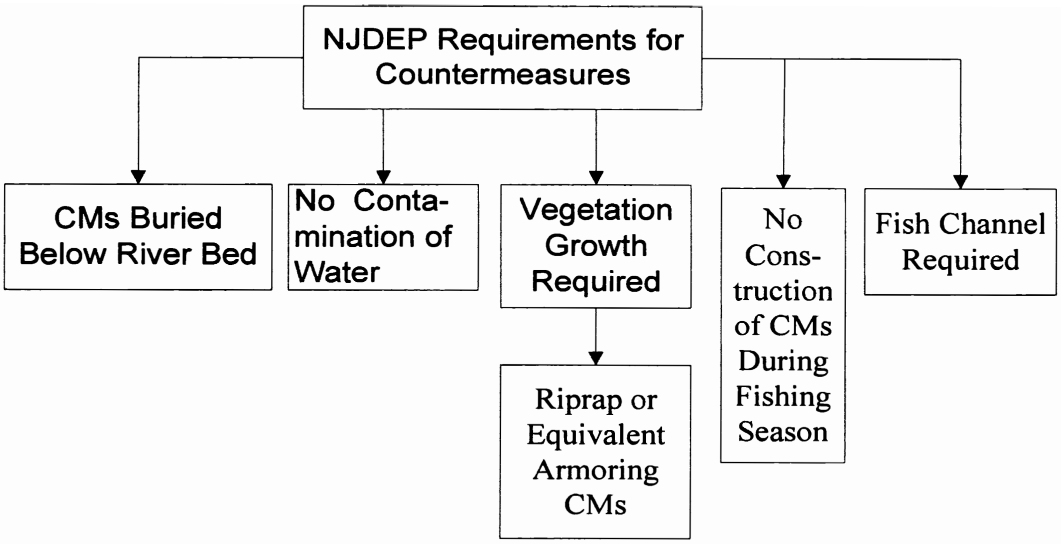


Figure 16.1. NJDEP Requirements for Stream Encroachment Permit.

Table 16.1 lists potential environmental impacts of scour countermeasures.

16.3 AGENCIES THAT MAY BE AFFECTED BY SCOUR COUNTERMEASURE INSTALLATIONS

1. Department of Environment Protection
2. Wildlife Conservation Groups
3. Angling Clubs
4. Coastguard and Other Navigation Organizations
5. Riparian Owners
6. Recreation Bodies

16.4 PERMITTING CONSIDERATIONS

The following permitting considerations are required before the installation of scour countermeasures:

1. Acquisition of Regulatory Permits
2. Obtaining a waiver
3. Requirements for Freshwater Wetlands General Permit Authorization

The following regulations in New Jersey may affect the permitting of scour countermeasures at a bridge site:

1. Watercourse Cleaning (NJAC 7:13-2.5): The channel will be restored to its original configuration after installation of countermeasures.

2. Fish Protection (NJAC 7:13-3.5) & Low Fish Passage (NJAC 3.6 and NJAC 7:13 -4.1(j)4): Applicable to trout associated water. A low flow channel will be provided in the center portion of opening. A minimum six-inch layer of native streambed material over the armoring will be used to help the passage of fish.

3. Change in opening size of bridge (NJAC 7:13-2.9 & 3.6): Applicable to new bridges only.

4. Channel Modification (NJAC 7:13-2.9 & 3.6(c)): There will be no change in cross sectional area and the hydraulic characteristics of the waterway will not be affected.

5. Acid Soils (NJAC 13-3.7 & NJAC 7:13-4.1 9(j) 6): Soil contents do not have clay plus silt content exceeding 30%.

6. Timing Restrictions (NJAC 7:13-3.5 & 3.6): The applicable timing restrictions will be adhered to.

7. Near Watercourse Vegetation Protection (NJAC 7:13-3.2): The footprint of the construction access road will not impact vegetation within 25 feet of the stream bank.

8. After construction is completed any disturbed areas will be restored to their original grade and re-vegetated with native species. Any vegetative debris created by the project will be removed from the site and not disposed of in the flood plain.

9. Soil Erosion & Sediment Control (NJAC 7:13-3.3): Soil erosion and sediment control measures will be provided in accordance with NJDOT Standards for Soil Erosion and Sediment Control. The pumping of sediment-laden water directly into the stream will be prohibited.

10. Storm water Management & Water Quality (NJAC 7:13-3.8): Any modification in the storm water discharge of the site will be kept to a minimum.

11. Disposal of Soils (NJAC 7-13-2.7): All excavated soils, which are not to be used, as part of the native streambed material cover over armoring will be removed from the site. Those native streambed materials which are to be reused, as a cover will be stock piled outside of the floodplain and beyond 25 feet from the top of the stream bank.

12. Wetlands (NJAC 7:13-3.8): No impact to wetlands.

13. Endangered Species (NJAC 7:13-3.9 & 7:13-4.1(j) 7): Field observations will be carried out. NJ Natural Heritage Program will investigate if federally protected species are present.

Table 16.1. Environmental Impacts of Recommended Countermeasures

Selected Countermeasure	Advantages	Disadvantages	Remarks	Type of Permit Required
Riprap	Stone is not a pollution risk. It blends well with cobbles and boulder beds and provides a good habitat.	Stone may not blend with soil, it may not be locally available and new quarry adversely affects landscape	Place below river bed and provide one foot overlay	Stream Encroachment
Gabions	Excavated soil can be replaced over gabions	Steel wires rust or may break	Place below river bed and provide one foot overlay	Stream Encroachment
Concrete Blocks	Suitable where stone is not available	May prevent growth of fauna, flora, vegetation	Place below river bed and provide one foot overlay	Stream Encroachment
Grout Filled Bags	Suitable for filling scour holes under or adjacent to footings	May prevent growth of fauna, flora, vegetation Risk of pollution from cement	Place below river bed.	Stream Encroachment
Concrete Aprons	More stable than riprap	Expensive. Prevents growth of fauna, flora, vegetation		Stream Encroachment
Sheet Piling	Very effective for long term scour protection	Expensive. May affect marine environment and fish passage		

CHAPTER 17

SCOUR PROTECTION AT CULVERTS

17.1 HYDRAULIC DESIGN

Chapter 9 of the AASHTO Model Drainage Manual describes design procedures for the hydraulic design of highway culverts. Included are design examples, tables and charts that provide a basis for determining the selection of a culvert opening. However, no scour analysis method is recommended for computing soil erosion under the culvert floor slab. Instead, scour at both the inlet and outlet of the culvert and at the wingwalls must be evaluated.

17.2 TYPES OF CULVERTS

Based on material and shape, culverts are of the following 4 types

1. Reinforced Concrete Box Culvert
2. Reinforced Concrete Pipe Culvert
3. Metal Pipe Culvert
4. Stone and Brick Arch Culverts
5. Precast

17.3 FACTORS AFFECTING SCOUR AT CULVERTS

The following factors must be considered for evaluating long term scour at culverts:

1. Area of opening
2. Flood velocity
3. Angle of flow
4. Longitudinal slope
5. Head water and tail water elevations
6. Invert elevation

17.4 SCOUR AT INLET

If a waterway opening is too small, poorly located due to skew or a culvert barrel is choked with sediment (Figure 17.1), debris or brushes aggradation will result at the inlet. Turbulence of water is likely to occur, sometimes leading to culvert failure.

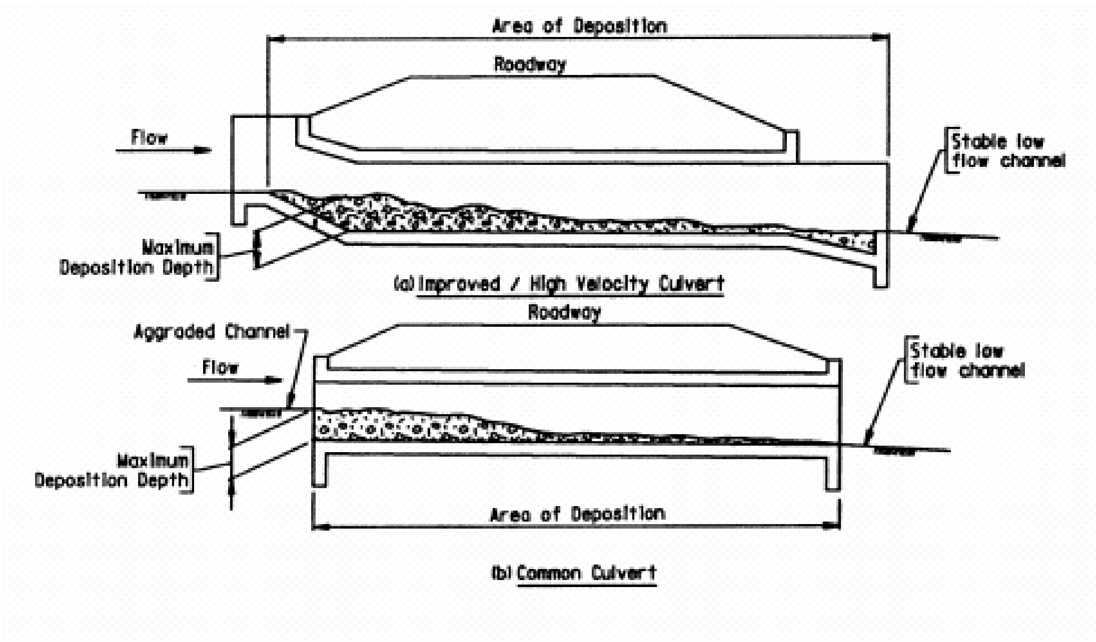


Figure 17.1. Culvert Sediment Deposition

17.5 SCOUR AT OUTLET

If water is discharged under pressure and the longitudinal slope is high, a large volume of water is discharged at a high velocity and soil degradation will be high. Local scour can occur at the wingwall footings. Also, if a longitudinal slope is small, clogging of culverts by silt or debris can occur.

17.6 COUNTERMEASURE DESIGN

The following countermeasures are recommended:

1. Debris control devices
2. Channel protection at upstream and downstream, such as riprap or gabions
3. Energy dissipator transition slab
4. Improved inlet and outlet design with headwall
5. Use of stilling basin or apron slab

Footings for any flared wing walls at the entry and the exit of culverts should be protected by riprap or alternate armoring countermeasures.

For high velocities exceeding 10 ft/sec , riprap at wing walls should be replaced by a concrete apron, which is to extend between the opposite wing walls and to the edge of the culvert.

Regular visual monitoring should be carried out if riprap has been installed at the entry and exit of culverts.

CHAPTER 18

CONSTRUCTION AND INSTALLATION OF COUNTERMEASURES

18.1 GENERAL CONSTRUCTIBILITY ISSUES

Before embarking on underwater construction, the following planning issues shall be evaluated:

1. **Duration of construction:** The available flow width may be reduced due to construction of cofferdams, embankment, and countermeasures.

   Flow velocities through the reduced channel opening will increase, thereby increasing scour in the channel and around the structure. Hence, construction of the above items shall be done during off-flood season.

2. **Maintenance and Protection of Traffic:** During installation of countermeasures, small cranes or pile driving equipment may be parked on a lane or shoulders. A lane closure would then be required. Coordination with traffic police and local officials would be necessary.

3. **Underwater work:** Health and safety of construction personnel may be of concern if depth of water is high. Trained divers will be required.

4. **Access to site:** Temporary road for transportation of materials and equipment adjacent to the channel bank may be difficult to construct. Wooden mats as shown in Figure 18.1 should be used when lane width is restricted.

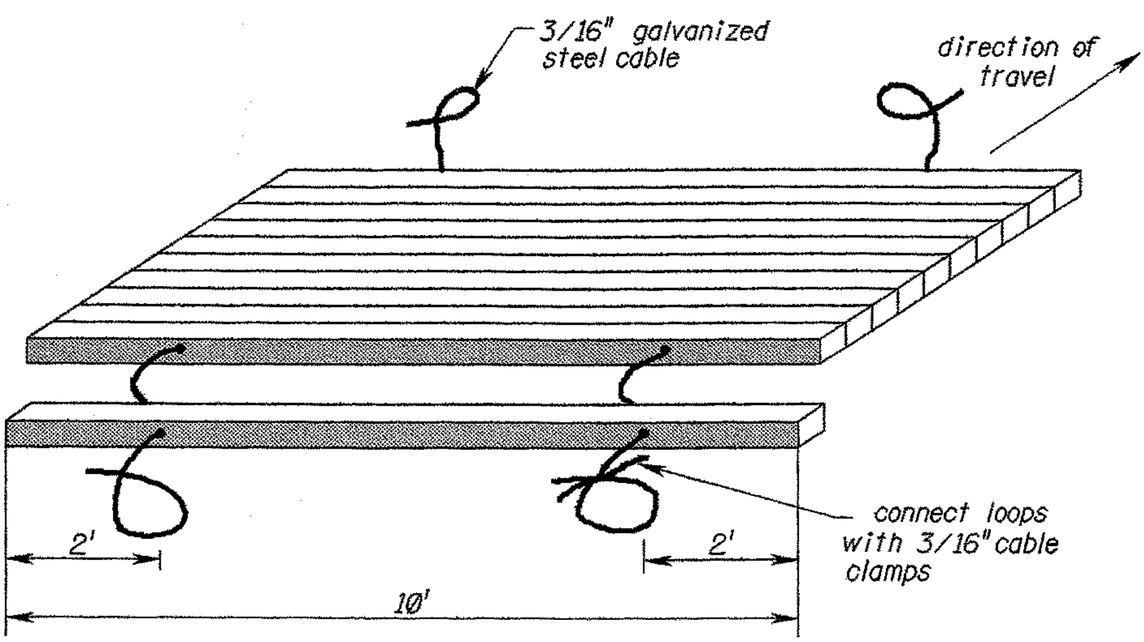


Figure 18.1. Construction Driveway Using Wood Mats.

5. **Temporary Works**: Temporary construction works may be required. More economical alternatives implementing quick construction and safety needs to be carefully evaluated.

6. **Safety of personnel**: Due to instability of banks because of recent floods (for banks with slopes steeper than 1:1), sudden collapse of bank may occur. OSHA safety standards must be followed.

7. **Environmental risks**: Pollution of river from construction material may occur. Channel needs to be cleaned. Approvals for stream encroachment permits would be necessary.

8. **Impact on existing utilities**: The effect of driving sheeting or bed armoring on existing utilities needs to be evaluated. Utilities may be relocated in such cases. Coordination and approval from utility company would be required.

9. **Impact on right of way**: Countermeasures may extend into adjacent property limits. Right of way needs to be purchased in such cases. Similarly, encroachment of adjacent property during construction may occur. A construction easement needs to be determined and permits obtained.

10. **Specialized work**: Modern countermeasures require new construction techniques. The contractor performing such tasks needs to train his construction crew for such techniques.

11. **Availability of labor and plant:** Some types of countermeasures such as gabions, interlocking blocks and stone pitching require experienced labor. Since local labor may not be familiar with the work, bringing labor from long distance may be expensive.

12. **Limited vertical clearance under the bridge**: It will be difficult to drive cofferdam and sheeting under the bridge if restricted vertical clearance is available. Placement of countermeasures will also be difficult.

18.2 EMERGENCY BRIDGE PROTECTION MEASURES

1. Any design and installation of bridge protection measures during high water can be difficult, if not impossible. A planned response for bridge scour is much preferred over a reactive response.

2. Emergency installation is typically much more costly than low water conditions because of better access, less immediate timing, many more countermeasure alternatives, and more effective use of materials.

3. There is usually an increased cost of mitigation since damage during an emergency project can be greater and equipment remobilization may be required for post-project mitigation. Project impacts (i.e., damages to trees

and vegetation) in carrying out emergency work have to be mitigated in the same way as for projects with normal timing.

4. Impacts of carrying out emergency work should be minimized. Under emergency scenarios, the tendency is to take actions to protect a bridge at the expense of existing trees and other vegetations. However, these trees and vegetations may be providing protection or may eventually protect bridge abutment or approach. The trees and vegetation also provide important riparian habitat, and should be protected even if they don't offer any direct stabilization of bridge countermeasures.

18.3 SAFETY CONSIDERATIONS

1. Working adjacent to fast, unpredictable currents and rapidly rising water levels can be extremely dangerous. Safety of construction workers is very important aspect of emergency scour countermeasure work.

2. Floating (or subsurface) debris and woody materials contribute to hazard during emergency work.

3. Weather conditions (rain, snow, or darkness) may further endanger safety.

18.4 DETAILED CONSTRUCTIBILITY ISSUES

1. Scour depth should be measured from a reference line1'-0" above the top of footing. If eroded elevation is located at a higher elevation than 1'-0" above the top of footing, the higher elevation will be considered.

2. For placing riprap, excavation to design depth will be carried out.

3. The depth of riprap should be at least below contraction scour depth.

4. If considerable erosion has already taken place and the riverbed elevation is below the top of footing, hydraulic analysis shall be based on new channel profile by considering the new opening size.

5. Embedment of footings for new bridge foundations:

Footing on non-erodible rock -	Minimum 6" into bedrock.
Footing on erodible rock -	Minimum 3'-0" in erodible rock
Footing on soil -	Minimum 6'-0" in soil (For existing footings minimum 3' depth of riprap shall be used).

6. The same type of armoring countermeasure shall be used for abutments and piers for economy and ease of construction. Armoring countermeasures will not be mixed; i.e., if gabions are selected, then they should be used for the whole bridge site. However, armoring countermeasures can be combined with structural countermeasures or river training measures.

18.5 OSHA RECOMMENDATIONS FOR SLOPES OF EXCAVATIONS IN SOILS

The following maximum values of slopes shall be for used for excavation of sloping structures. The angle of repose shall be flattened when an excavation has water conditions.

1. Solid rock - 90 degrees
2. Compacted angular gravels - 0.5:1 (63 deg. 26')
3. Average soil 1:1 - (45 deg)
4. Compacted sand 1.5:1 - (33 deg. 41')
5. Loose sand 2:1 - (26 deg 34')

18.6 UNDERWATER CONSTRUCTION AND INSPECTION

1. Standard procedures for underwater inspection shall be followed so that observations are accurate and safety of divers is not jeopardized.
2. Trained underwater inspectors may be required. Refer to NJDOT, "Underwater Inspection and Evaluation of New Jersey Bridges", May, 1994. When divers are used. Visibility under water would be limited.
3. If a large plan area is required to be dewatered, dewatering can be problematic and expensive.
4. Flow conditions in the river may make it difficult to place filter layers.

18.7 COFFERDAMS

If the water depth is not high, temporary cofferdams may be required for construction in dry conditions. For water depth less than 5 feet, cofferdams are not used. However, without dry conditions the quality of placement of countermeasures will be difficult to monitor or maintain. Figure 18.1 shows elevation view of typical Cofferdam.

Cofferdam Construction Notes

1. Cofferdam sheet piles shall be at least of Profile Section AZ18 (this may not be available. Write in minimum section modulus) and shall meet the requirements of ASTM A752.
2. Sheet piles may be braced with perimeter walls and knee braces as required for improving the stability of sheet piles.
3. Drive cofferdam sheet piles below grade.

4. Dewater within cofferdam.

5. Excavate to the design depth of riprap layer or gabion mat.

6. Install riprap/alternate countermeasures within cofferdam.

7. Backfill to original bank/river bed.

8. Remove bracing system, complete backfill and remove sheet piles.

9. Repeat steps 1 to 8 at another pier/abutment reusing the removed sheet piles.

10. If water depth is less than 3 feet, sandbags may be used in place of sheet piles.

Cofferdam Design Procedure: The following procedures should be followed:

1. Design calculations and working drawings shall be submitted according to NJDOT Standard English Specifications.

2. Cofferdams used in the preparation and protection of the foundation shall be carried below the bottom of the footings, shall be braced in all directions, and shall be of such construction as to permit them to be pumped and maintained free of water until the construction therein has been completed.

3. Cofferdams shall be so constructed as to protect the foundation and the construction against damage from a rise in the water elevation.

4. Timber or bracing of a cofferdam may extend into or through the substructure masonry only with written authorization.

5. Cofferdams with all falsework, sheeting, and bracing shall be removed after the completion of the substructure therein, except where sheeting is designated to remain.

6. Where sheeting interferes with batter piles, the depth of penetration of the interfering sheets may be reduced or the sheeting may be moved out to provide clearance between the sheeting and the batter piles as authorized.

7. Working drawings shall be submitted showing proposed construction and approval shall be obtained before proceeding with the work.

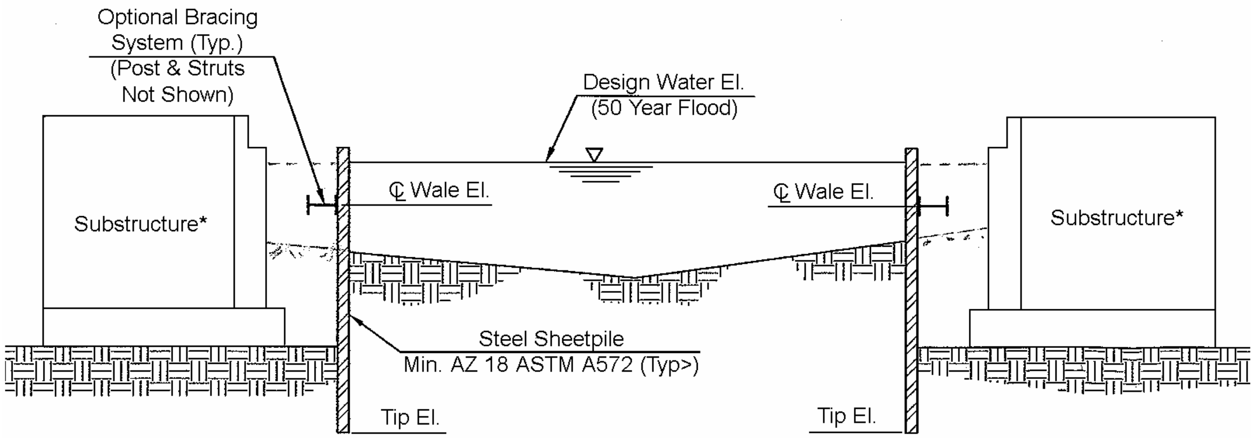


Figure 18.2. Cofferdam Elevation

18.8 SHEET PILING LEFT IN POSITION

For underwater construction, temporary sheeting on the stream side is required for installing countermeasures at the sides of spread footings/pile caps for the following abutment and pier types:

Abutment Types:

a. Full Height

b. Stub

c. Spill Through

d. Integral

Pier Types:

a. Wall

b. Hammerhead

c. Column Bents

d. Pile Bents

To prevent long term scour, temporary sheeting may be left in place after countermeasures installation is completed.

18.9 TRAFFIC AND UTILITIES ISSUES

1. Site access: Adequate access to the site shall be provided for trucks to deliver riprap.

2. Right of Way: Construction easement and right of way may be purchased, for the duration of construction.

3. Possible detours: Detour, lane closure or night time work may be necessary. Coordination with Traffic Control department would be required. Emergency vehicles and school bus services shall not be affected by lane closures.

4. Utilities: Relocation of any utilities at the sides of abutment or pier may be necessary for the duration of construction. Coordination with utility company would be required.

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