

# Handbook of Scour Countermeasures Designs

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Submitted by

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## LIST OF ABBREVIATIONS AND SYMBOLS

C	=	Coefficient
D <sub>50</sub>	=	Median stone diameter, ft
F <sub>r</sub>	=	Upstream Fraud Number
K	=	Velocity magnification factor
K	=	Coefficient for pier shape
O <sub>95</sub>	=	Opening size where 95% of pores are smaller, ft
Q	=	Discharge, ft <sup>3</sup> /s
S <sub>s</sub>	=	Specific gravity of riprap
V	=	Flow velocity, ft/s
W	=	Width of pier at base, ft
W	=	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,
d	=	Thickness of riprap
y	=	Scour depth, ft
y <sub>0</sub>	=	Flow depth, ft
$\alpha$	=	Location factor
$\beta$	=	Flood factor
$\zeta$	=	Scaling Factor for application to the geology of soil
$\eta$	=	Scaling Factor for application to width of the bridge opening
$\xi$	=	Scaling Factor for application to river training measures
$\chi$	=	Scaling Factor for application to the remaining bridge life assessed
$\psi$	=	Scaling Factor for application to underwater inspection
$\Psi_{CR}$	=	Stability Factor
$\theta$	=	Side slope angle with respect to the horizontal plane
$\theta$	=	Angle between the impinging flow direction and the vertical wall
$\rho$	=	Density of water, lb/ft <sup>3</sup>
$\rho_{cob}$	=	Density of block material, lb/ft <sup>3</sup>

## FOREWORD

This handbook has been prepared to provide comprehensive and detailed guidelines on bridge scour countermeasures in New Jersey. The handbook can be used by bridge engineers and designers for designing countermeasures for foundations of abutments and piers against scour. It can also be used for planning of a new bridge site to reduce scour risk. Understanding of theoretical and practical requirements for the concept, need, selection and application of countermeasures as based on erosion type, substructure component, and cost considerations are the pre-requisites for a detailed design. The handbook deals with all such aspects of countermeasure design in a comprehensive manner. Detailed specifications for several countermeasures that are proprietary in nature may be obtained directly from the vendors, if not available in NJDOT Standard Specifications. Guidelines recommended in this handbook can be supplemented with design data provided by the manufacturers.

The handbook is based on Hydraulic Engineering Circulars (HEC-18, HEC-23), recent NCHRP research reports, CIRIA (2002) Manual on Scour used extensively in Britain, NJDOT Phase II in-depth scour evaluation studies and consultations with various State DOTs. The handbook is prepared with a goal to supplement HEC-18 and HEC-23 so that a bridge engineer/consultant can address aspects of countermeasure design in New Jersey effectively, including environmental, constructability and geotechnical aspects.

This handbook has been prepared through extensive review of numerous literature resources, including but not limited to 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 (2002).
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.

Other references used in developing this handbook are presented in Appendix I.



## DEFINITIONS OF SCOUR RELATED TERMINOLOGY

**AASHTO (LRFD) Requirements on Scour Design:** The Extreme-Event Limit States relate to events with return periods in excess of the design life of the bridge. A flood event exceeding a 100-year flood is generally considered as extreme event for bridges. A 500-year flood is recommended as the check flood for scour. Other conditions relating to scour that the designer may determine to be significant for a specified watershed may be ice loads or debris logging operations, etc. The AASHTO LRFD specifications required that scour at bridge foundations be designed for 100-year flood storm surge tide or for the overtopping flood of lesser recurrence.

**Aggradations:** Progressive buildup of longitudinal profile of channel bed due to sediment deposition.

**Apron:** Protective material placed on a streambed to resist scour.

**Armor (armoring):** Surfacing of channel bed, banks, or embankment slope to resist erosion and scour using riprap, gabions, and articulated concrete blocks, etc.

**Bank:** The sides of a channel between which the flow is normally confined.

**Base floodplain:** The floodplain associated with the flood with a 100-year recurrence interval.

**Bed:** The bottom of a channel bounded by banks.

**Bed material:** Material found in and on the bed of a stream.

**Bedrock:** The solid rock exposed at the surface of the earth or overlain by soils and unconsolidated material.

**Bed slope:** The inclination of the channel bottom.

**Blanket:** Material covering all or a portion of a stream bank to prevent erosion.

**Bridge opening:** The cross-sectional area beneath a bridge that is available for conveyance of water.

**Causeway:** Rock or earth embankment carrying a roadway across water.

**Cellular-block:** Interconnected concrete blocks with regular cavities placed directly on a stream bank to resist erosion.

**Channel:** The bed and banks that confine the surface flow of a stream.

**Channelization:** Straightening or deepening of a natural channel artificially.

**Check dam:** A low dam or weir across a channel used to control stage or degradation.

**Clear-water scour:** Scour at a pier or abutment (or contraction scour) when there is no movement of the bed material upstream of the bridge crossing at the flow causing bridge scour.

**Concrete revetment:** Unreinforced or reinforced concrete slabs placed on the channel bed or banks for the protection from erosion.

**Confluence:** The junction of two or more streams.

**Constriction:** A natural or artificial control section, such as a bridge crossing, channel or dam, with limited flow capacity.

**Contraction scour:** This component of scour results from a contraction of the flow area at the bridge, which causes an increase in velocity and shear stress on the bed at the bridge. The contraction can be caused by the bridge or from a natural narrowing of the stream channel. Contraction scour, in a natural channel or at a bridge crossing, involves

the removal of material from the bed and banks across all or most of the channel width. Contraction scour occurs when the flow area of a stream at flood stage is reduced, either by a natural contraction or bridge. It also occurs when overbank flow is forced back to the channel by roadway embankments at the approaches to the bridge. For continuity, a decrease in flow area results in an increase in average velocity and bed shear stress through the contraction. Hence, there is an increase in erosive forces in the contracted area and more bed material is removed from the contracted waterway than is transported into the waterway. This increase in transport of bed material from the waterway lowers the natural bed elevation.

**Countermeasure:** A measure to prevent, delay or reduce severity of hydraulic problems.

**Crib:** A frame structure filled with earth or stone ballast, designed to reduce energy.

**Critical shear stress:** Minimum amount of shear stress required to initiate soil particle motion.

**Current meter:** An instrument used to measure flow velocity.

**Debris:** Floating or submerged material, such as logs, or vegetation transported by stream.

**Degradation (bed):** A general and progressive (long-term) lowering of the channel bed due to erosion, over a relatively long channel length.

**Depth of scour:** Vertical distance a streambed is lowered by scour below a reference elevation.

**Design flow (design flood):** The discharge that is selected as the basis for the design or evaluation of a hydraulic structure.

**Dike (groin, spur, and jetty):** A structure extending from a bank into a channel that is designed to reduce the stream velocity

**Drift:** Alternative term for vegetative "debris."

**Eddy current:** A vortex-type motion of a fluid flowing contrary to the main current, such as the circular water movement, that occurs when the main flow becomes separated from the bank.

**Fabric mattress:** Grout-filled mattress used for stream bank protection.

**Fascine:** A matrix of willow or other natural material woven in bundles and used as a filter.

**Filter:** Layer of fabric (geotextile) or granular material (sand, gravel, or graded rock) placed between bank revetment (or bed protection) and soil filter blanket: For example, a layer of graded sand and gravel is laid between fine-grained material and riprap to serve as a filter.

**Floodplain:** Nearly flat, alluvial lowland bordering a stream that is subject to frequent inundation by floods.

**Flow-control structure:** A structure either within or outside a channel that acts as a countermeasure by controlling the direction, depth, or velocity of flowing water.

**Froude Number:** A dimensionless number that represents the ratio of inertial to gravitational forces in open channel flow.

**Gabion:** A basket or compartmented rectangular container made of wire mesh. When filled with cobbles or other rock of suitable size, the gabion becomes a flexible and permeable unit with which flow and erosion control structures can be built.

**General scour:** General scour is a lowering of the streambed across the stream or waterway at the bridge. This lowering may be uniform across the bed or non-uniform. i.e., the depth of scour may be deeper in some parts of the cross section. General scour may result from contraction of the flow or other general scour conditions such as flow around a bend.

**Grout:** A fluid mixture of cement, sand, and water used to fill joints and voids.

**Guide bank:** A dike extending upstream from the approach embankment at either or both sides of the bridge opening to direct the flow through the opening.

**Hardpoint:** A stream bank protection structure whereby "soft" or erodible materials are removed from a bank and replaced by stone or compacted clay. Some hard points protrude a short distance into the channel to direct erosive currents away from the bank.

**Hydrograph:** The graph of stage or discharge against time.

**Hydrology:** Science concerned with occurrence, distribution, and circulation of water on earth.

**Jetty:** An obstruction built of piles, rock, or other material extending from a bank into a stream, so placed as to induce bank building, or to protect against erosion

**Lateral erosion:** Erosion in which the removal of material is extended horizontally as contrasted with degradation and scour in a vertical direction.

**Launching:** Release of undercut material (stone riprap, rubble, slag, etc.) down slope or into a scoured area.

**Levee:** An embankment, generally landward of top bank that confines flow during high-water periods, thus preventing overflow into lowlands.

**Local scour:** Removal of material from around piers, abutments, spurs, and embankments caused by an acceleration of flow and by vortices induced by obstructions to the flow.

**Mattress:** A blanket or revetment of materials interwoven or otherwise lashed together and placed to cover an area subject to scour.

**Meander or full meander:** A meander in a river consists of two consecutive loops, one flowing clockwise and the other counter-clockwise.

**Migration:** Change in position of a channel by lateral erosion of one bank.

**Pile:** An elongated member usually made of timber, concrete, or steel that serves as a structural component of a river-training structure.

**Reinforced-earth:** A retaining structure consisting of vertical panels and bulkhead: attached to reinforcing elements embedded in compacted backfill for supporting a stream bank.

**Relief bridge:** An opening in an embankment on a floodplain to permit passage of over bank flow.

**Retard** (Retarder): A permeable or impermeable linear structure in a channel structure parallel to the bank, intended to reduce flow velocity, induce deposition, or deflect flow from the bank.

**Revetment:** Rigid or flexible armor placed to inhibit scour and lateral erosion.

**Riprap:** Layer of rock or broken concrete dumped or placed to protect footing of bridge piers and abutments, or embankment from erosion.

**River training:** Measures taken to direct or to lead a stream flow into a prescribed channel.

**Rock-and-wire mattress:** A flat wire cage or basket filled with stone or other suitable material and placed as protection against erosion.

**Rubble:** Rough, irregular fragments of materials of random size used to retard erosion. The fragments may consist of broken concrete slabs, masonry, or other suitable refuse.

**Runoff:** Part of precipitation, which appears in surface streams of either perennial or intermittent form.

**Sack revetment:** Sacks (e.g., burlap, paper, or nylon) filled with mortar, concrete, sand, stone or other available material used as protection against erosion.

**Scour:** Localized erosion of streambed or bank material due to flowing water.

**Seepage:** The slow movement of water through small cracks and pores of the bank material.

**Sill:** A structure built under water with the aim of raising the depth and slope of streambed.

**Slope protection:** Any measure such as riprap, paving, vegetation, revetment, brush or other material intended to protect a slope from erosion, slipping or caving.

**Soil-cement:** A designed mixture of soil and Portland cement compacted at a proper water content to form a blanket or structure that can resist erosion.

**Spill-through abutment:** A bridge abutment having a fill slope on the stream ward side.

**Spur:** A permeable or impermeable linear structure that projects into a channel from the bank to alter flow direction, induce deposition, or reduce flow velocity along the bank.

**Stone riprap:** Natural cobbles, boulders, or rock dumped or placed as protection against erosion.

**Stream:** A body of water ranging in size from a large to a small river flowing in a channel.

**Stream bank protection:** Any technique used to prevent erosion or failure of a stream bank.

**Stream instability:** Changes in river cross-section due to horizontal and vertical erosion of riverbanks from hydraulic and geomorphic factors.

**Stream stability:** No appreciable changes in cross section of river from year to year due to conditions of flow.

**Tetra pod:** Bank protection component of precast concrete consisting of four legs joined at a central joint, with each leg making an angle of 109.5° with the other three.

**Thalweg:** The line extending down a channel that follows the lowest elevation of the bed.

**Timber or brush mattress:** A revetment made of brush, poles, logs, or lumber interwoven or otherwise lashed together.

**Toe protection:** Loose stones laid or dumped at the toe of an embankment, groin; masonry or concrete wall built at the junction of the bank and the channel bed or at extremities of hydraulic structures to counteract erosion.

**Total scour:** The sum of long-term degradation, general (contraction) scour and local scour.

**Turbulence:** Motion of fluids in which local velocities and pressures fluctuate irregularly in a random manner as opposed to laminar flow where particles of fluid move in distinct and separate lines.

**Ultimate scour:** Maximum depth of scour attained for a given flow condition.

Development of ultimate scour may require multiple flow events. In cemented or cohesive soils, ultimate scour may be achieved over a long time period.

**Uniform flow:** Flow of constant cross section and velocity through a channel at a given time.

**Vortex:** Turbulent eddy in the flow caused by an obstruction such as a bridge pier or abutment.

**Weephole:** A hole in an impermeable wall or revetment to relieve the neutral stress or pore pressure in soil.

**Wire mesh:** Wire woven to form a mesh.

## CHAPTER 1

### INTRODUCTION TO SCOUR OF BRIDGE FOUNDATIONS

#### Background

Scour is the result of erosive action of running water, excavations and erosion of material from bed and banks of stream. Over thirty six thousand bridges in the 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".

Over thirty six thousand bridges in USA are either scour critical or scour susceptible. In New Jersey, there are 467 scour critical bridges in New Jersey. These bridges are susceptible to damage because of the erosion of streambed material during severe floods that could cause damage.

Bridge abutments and their approach embankments are the most common bridge components that may be damaged during floods. Many of the failures due to scour have been observed in older bridges. Provisions for scour analysis were not developed or fully understood at the time of planning of these bridges. AASHTO LRFD Bridge Design Specifications in 1994 introduced scour as an extreme event in the load combinations for design of bridges. With the publications of HEC18, HEC 20 and HEC 23 by FHWA, formal hydraulic and scour analyses are now being carried out, leading to a rational design of bridge footings against scour.

Some examples of recent scour-related bridge failures are:

- Ovilla Road Bridge located in Ellis County in TX in 2004.
- Route 46 Bridge on Peckman's River Bridge in Passaic County in NJ in 1998.
- Schoharie Creek Bridge located on NY State Thruway in April 1987
- US 51 Bridge over Hatchie River in TN in 1989
- Damages to bridges located on Mississippi River in 1993
- Interstate 5 NB and SB Bridges over Los Gatos Creek in CA in 1995

Figure 1.1 shows Ovilla Road Bridge in Ellis County, Texas, that collapse because of foundation settlement during heavy flooding on July 30, 2004. Figure 1.2 shows failure of approach at New Jersey Route 46 Peckman's River Bridge after Hurricane Floyd because of settlement of spread footing due to contraction scour. Figure 1.3 shows severe damages to a major bridge because of overtopping flood.



Figure 1.1: Scour Failure of Ovilla Road Bridge in Ellis County, TX on July 20, 2004.



Figure 1.2: Failure of approach at New Jersey Route 46 Peckman's River Bridge after Hurricane Floyd had struck.



Figure 1.3: Severe damage to a major bridge due to Overtopping of floods.

## Scours at Abutments and Piers

Hydraulic conditions and rates of erosion are vastly different at abutments and piers at any bridge site. Extent of erosion at abutments may be minimized by placing them away from the river banks. Piers are located in the middle of peak flood zones, 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. Hence, foundation all around a pier may be scoured. On the other hand, the foundation only on side exposed to the flow in case of an abutment may be scoured.

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. Total scour at bridge footings is primarily sum of degradations and aggradations, local scour and contraction scour. Degradation is a general and progressive (long-term) lowering of the channel bed due to erosion over a relatively long channel length. Local scour is due to increase in local flow velocities and turbulence levels because of obstruction caused by bridge piers and abutments to the water flow. Contraction scour is because of increased water velocity in the bridge opening as a result of decrease in cross-sectional area of waterway at the bridge crossing.

Scour analysis of bridge footings should be carried out on the basis of guidelines and equations presented in HEC-18. Scour depth should be measured from a reference line 1'-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.

Scour analysis of bridge piers and abutments in cohesive soils can be carried out on the basis of NCHRP 516 report entitled "Pier and Contraction Scour in Cohesive Soils" by Briaud et al (2004). Countermeasures for bridge footings can be designed by guidelines presented in this handbook based on scour analysis data, such as total scour depth, peak flow velocities, etc.

A detailed scour analysis at a bridge site may require the following steps:

- Studying bridge hydraulics to screen bridges as scour critical and non-scour critical.
- Studying past floods and obtaining flood data from any gages installed near bridge sites.
- Performing field survey to obtain topography, river boundaries, obstructions to flow, catchment area, vegetation and terrain.
- Performing hydrologic analysis.
- Performing geotechnical Investigation for types of soil and foundations.
- Examine aggradation, degradation, contraction and local scour.
- Setting up a hydraulic model to calculate scour depths because of degradation, local scour and contraction scour.

Design of countermeasures at a site may require the following steps:

- Performing a feasibility study for using appropriate countermeasure based on alternatives, economics, site conditions, permit considerations and constructability.
- Considering alternatives for river training, structural planning and deep foundations



for new bridge designs.

- Selecting preliminary design of countermeasures and preparing cost estimates.
- Reviewing countermeasure design for compliance with structural and environmental permit requirements.
- Developing technical specifications, construction methods and construction schedule.

### **Site-Specific Data for Scour Analysis and Design**

The following is the summary of site-specific factors, which directly contribute to soil erosion. These factors need to be identified for scour analysis and countermeasure design:

1. Geology and types of soils – Geotechnical investigation to be performed.
2. Hydrology – Meteorological data required.
3. Hydraulics – Manning's Coefficient to model waterway for HEC-RAS software.
4. Location and width of waterways – Field survey required.
5. Longitudinal slope - Field survey required
6. Environmental conditions – Data for marine and plant life, and NJDEP permit approval.
7. Bridge Characteristics, size of opening, site conditions
8. Footing type and depth – Unknown foundations require boreholes.
9. If considerable erosion has already taken place and the riverbed elevation is below the top of footing, hydraulic analysis should be based on the new channel profile by considering the new opening size.

### **Scour Studies in New Jersey**

In New Jersey, Stage I Scour Investigation (Screening and Prioritization) was completed in 1992 for all state owned, toll agency and majority of county bridges. This investigation identified a priority list of scour critical bridges in New Jersey. Detailed information on this work may be obtained from Baig et al. (1993, 2002), Wojcik (1993), and Anella and Olinger (1993).

Stage 2 In-depth scour studies by NJDOT have identified several hundred scour critical bridges in New Jersey. Records of existing results from hydraulic analysis, scour analysis and countermeasure design of such bridges are available. However, scour conditions are likely to change with demography, hydrology, aggradation and degradation every few years, and a fresh scour analysis may be required. Hence, studies done in the past can only be used as initial information to carry out in-depth scour study.

### **Rivers in New Jersey**

New Jersey has a large network of brooks, creeks, streams and rivers as shown in Figure 1.4. Examples of perennial rivers with high flood velocity are the Raritan and Passaic Rivers. On the West side of the state is the Delaware River (not shown) and on the East side is the Atlantic Ocean with tidal inlets (not shown). There are numerous streams, which are seasonal and which become active during storms. Names of only major rivers are shown in the map. Morris and Hunterdon Counties have the largest

number of bridges on rivers subjected to scour.

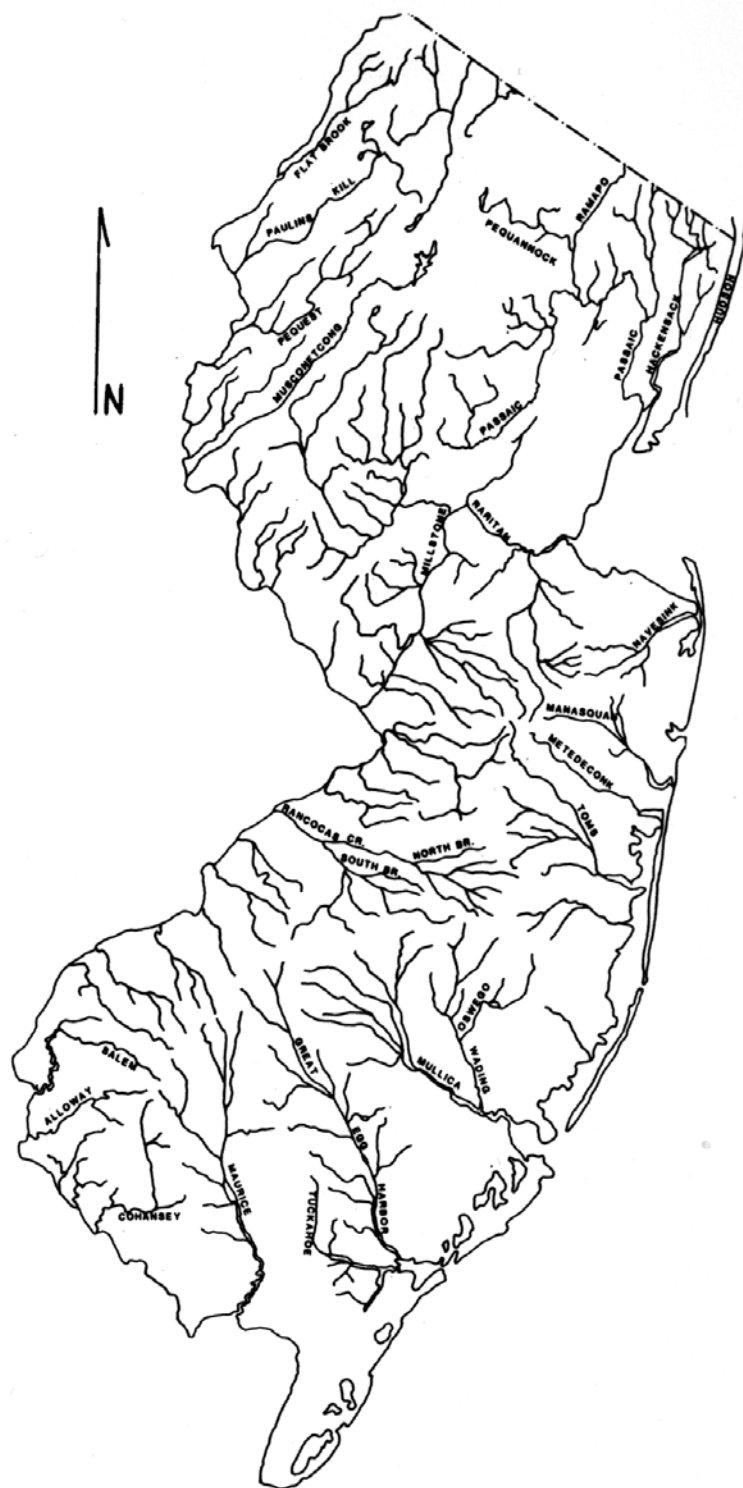


Figure 1.4: Drainage Map of New Jersey Containing Scour Critical Rivers.

## **Applications of Computer Software for Scour Analysis.**

Construction drawings for scour countermeasures must be based on detailed designs using approved commercial software. A spreadsheet may be developed and used in lieu of software. Following software may be used for detailed scour.

### **a. Hydrology**

1. *TR55*
2. *PENNSSTATE Program*

### **b. Hydraulics**

1. HEC-RAS
2. WSPRO
3. UNET

### **c. Scour Analysis**

1. HEC-RAS
2. Excel Spread Sheets Based On Hec-18 Equations

## **Organization of the handbook**

The handbook consists of 15 chapters. A brief overview of organization of these chapters is as follows:

**Chapters 1 to 3:** These chapters present brief introduction on pre-countermeasure design issues, including geotechnical and environmental issues. A user is suggested to read these chapters to prepare for scour countermeasure designs.

**Chapters 4 to 7:** These chapters pertain to selection of countermeasures for existing bridges. These chapters present detailed information of countermeasures based on scour type and costs, recommended countermeasures for piers and abutments, scour countermeasures for culverts, selection of countermeasures and monitoring countermeasures.

**Chapter 8:** This chapter presents detailed guidelines of planning of new bridges to reduce or minimize effects of scour.

**Chapter 9:** This chapter presents information on constructibility of scour countermeasures for existing bridges.

**Chapter 10 to 11:** These chapters present detailed guidelines on design of armoring scour countermeasures for existing bridges. Chapter 10 presents design guidelines for filters, which are essential for effectiveness of countermeasures, such as riprap, gabions, etc. Chapter 11 presents detailed guidelines for design of armoring countermeasures for existing bridges.

**Chapter 12:** This chapter presents guidelines on river training measures that may be used with out without armoring countermeasures.

**Chapter 13:** This chapter presents detailed guidelines on structural countermeasures for existing bridges.

**Chapter 14:** This chapter presents detailed guidelines on design of riprap for existing brides. Although riprap is an armoring countermeasure, its guidelines are presented in a separate chapter since riprap is considered to be a temporary countermeasure.

## **CHAPTER 2**

### **ENVIRONMENTAL ASPECTS OF COUNTERMEASURES INSTALLATION**

#### **Introduction**

Environmental considerations are by far the most important in the installation and construction of countermeasure. Streams are sensitive to encroachments, since water quality, marine life and vegetation are likely to be affected. Hence, volume and impact of stream encroachment should be minimized in the design of a countermeasure.

In New Jersey, it is mandatory to obtain NJDEP approval of the concept of countermeasure type, justify its use and obtain construction permits. More than one permit may be required from NJDEP, such as permits for stream encroachment and water quality, and installation of a selected countermeasure. By following the environmental permit application procedures, design and construction of a scour countermeasure can be carried out by resolving environmental issues in a realistic manner.

Meetings with the DEP engineers at an earlier stage of design are recommended, as resolution of technical issues will lead to appropriate selection of countermeasure in an efficient manner. 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. Use of a 6 inches minimum layer thickness of native substrate cover over any proposed armoring
6. Minimization of erosion of native substrate due to sediment transport after the installation
7. Reactions with acid producing soils and air quality (contamination, pollution)
8. Noise, aesthetics and traffic disruption
9. Historical and cultural aspects
10. Socio-economic aspects, job creation
11. Minimization of impacts to natural vegetation by controlling construction access points. Re-vegetation of disturbed areas with species may be required.

The following steps should be performed to minimize adverse environmental impacts:

1. Develop a baseline survey to define current environment issues

2. Develop an assessment of impact of proposed countermeasures on the current river environment
3. Develop considerations or measures to avoid or mitigate adverse impacts.

The flowchart in Figure 2.1 illustrates the basic NJDEP requirements for minimum impacts on the stream. NJDEP may be contacted for their detailed requirements for allowing construction in the waterway.

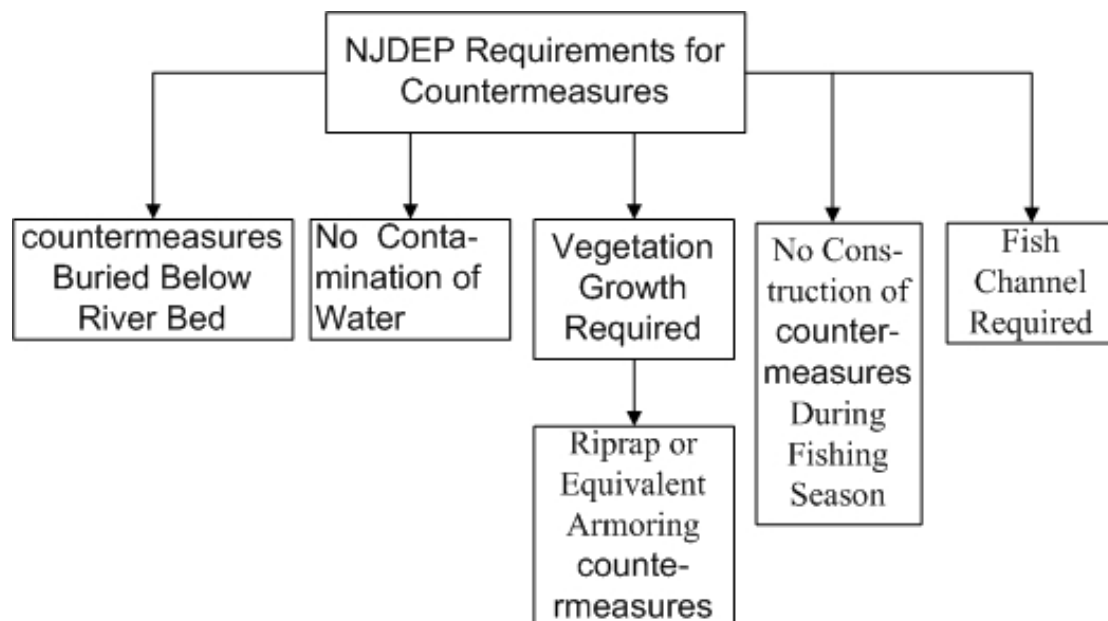


Figure 2.1: NJDEP General Requirements for Stream Encroachment Permit.

### Countermeasure Selection for Stream Beds and Banks

When selecting armoring for stream beds and banks, the following analysis of applicable countermeasure will help in meeting the environmental permit requirements at the time of permit application:

1. Selected Countermeasure Type: Vegetable planting, grasses, trees and shrubs
  - Scour Type to be addressed: Degradation, Lateral erosion
  - Description: Vegetables, grasses and trees need to be planted to prevent bank erosion both on upstream and downstream of the bridge.
  - Advantages: Low cost. Suitable for natural appearance and varied habitat. Vegetation roots bind together the soil particles and minimize their movements during flowing water.
  - Disadvantages: Difficult to plant on the steep banks of river or for soil with large size stones.

- Remarks: Highly recommended. Meets environmental requirements in NJ
2. Selected Countermeasure Type. Packed and compacted rock riprap on a geotextile or filter fabric layer
- Scour Type to be addressed: Local scour, degradation, lateral erosion
  - Description: Graded broken rock placed below river bed in position, by hand or by machine on geotextile layers and overlaid with soil.
  - Advantages: Geotextile acts as filter, preventing the bed erosion to a high degree. Familiarity with similar application and past experience in NJ. Relatively low cost and minimum maintenance. Easy to construct, ability to adjust to minor scour
  - Disadvantages: Oversize stones are not available in abundance. Smaller sizes of stone wash out easily in floods. It tends to disturb channel ecosystem, until vegetation is reestablished. Labor intensive. Low cost of riprap is offset by placement cost.
  - Remarks: Recommended for temporary use until foundations are fixed, permanent countermeasures are installed or the bridge is replaced with better planning.  
Permitted for secondary use on wingwall foundations and river banks subjected to low scour and low flood velocities.  
Meets environmental requirements of NJDEP.
3. Selected Countermeasure Type. Artificial Riprap
- Scour Type to be addressed: Local scour, Degradation, Lateral erosion.
  - Description: Alternatives to riprap such as using tetrapods / toskanes.
  - Advantages: Useful if rock riprap is not available locally from the quarry. Precast units can be mass-produced in factories at low costs. Due to precasting, cast in place and tremie concrete will not affect water quality.
  - Disadvantages: Concrete is likely to prevent vegetation growth, unless buried below river bed.
  - Remarks: Highly recommended, subject to meeting environmental permit requirements in NJ.
4. Selected Countermeasure Type. Gabions / Reno mattress on Geotextile layer
- Scour Type to be addressed: Local scour Degradation, Lateral erosion
  - Description: Galvanized woven or welded wire mesh baskets, mattress filled with loose stones. Wire should be coated with PVC.
  - Advantages: Available in various sizes as wire enclosed boxes or baskets. Can adapt to the steep slopes of banks. Rocks inside baskets do not move. Permits vegetation growth. Useful, even where large size rock is available. Relatively low cost, and has the ability to adjust to minor scour.
  - Disadvantages: Wire may break due to corrosion or cut by vandals. Tree branches /debris may get trapped in wires. Regular maintenance is required. Possible disturbance of channel ecosystem.

- Remarks: Cost effective. Highly recommended as armoring. Gabion baskets have become popular in NJ in recent years. Meets environmental permit requirements in NJ

5. Selected Countermeasure Type. Precast concrete interlocking blocks

- Scour Type to be addressed: Local scour Degradation, Lateral erosion
- Description: Concrete blocks of a cellular shape placed as revetment
- Advantages: Available in large quantities from precast concrete factories. Useful in lieu of large size rock riprap.
- Disadvantages: Likely to prevent vegetation growth or affect marine life. Weep holes for relief of hydrostatic pressure may be required. Liable to move in floods unless anchored by spikes.
- Remarks: Recommended, subject to meeting environmental requirements in NJ. Its long-term performance needs to be monitored but has been successful in other States.

6. Selected Countermeasure Type. Cable-tied blocks

- Scour Type to be addressed: Local scour, Degradation, Lateral erosion
- Description: Concrete blocks / slabs interconnected with steel cables
- Advantages: Minimum maintenance. Conforms to river shape better than concrete blocks. Suitable for piers. Will not wash out easily in floods.
- Disadvantages: Not suitable for pile bent bridges. Steel cables likely to corrode and affect water quality. Edges of revetment need to be anchored into underlying material. Expensive, since divers are required to tie blocks in deeper water. Higher maintenance and costs compared to other types of precast units.
- Remarks: Recommended subject to meeting environmental requirements in NJ

7. Selected Countermeasure Type. *Sacked concrete / grout filled bags*

- Scour Type to be addressed: Local scour, Degradation, Lateral erosion
- Description: Fabric bags filled with concrete and stacked to produce a protective layer. Sand filled bags preferred.
- Advantages: Suitable for sandy soils only and very useful for filling scour holes under footings and elsewhere.
- Disadvantages: Undermining of toe may result. Likely to prevent vegetation growth or marine life. Risk of pollution from cement washout. Catastrophic failure potential due to lack of interlocking.
- Remarks: Recommended for filling scour holes under footings subject to meeting environmental requirements in NJ.

8. Selected Countermeasure Type. Dumped riprap without geotextile or fabric filter layer

- Scour Type to be addressed: Local scour Degradation, Lateral erosion

- Description: Graded broken rock, placed below river bed in position by hand or dumped by boats and overlaid with soil
- Advantages: Familiarity and past experience in NJ. Relatively low cost and no maintenance required. Easy to construct, ability to adjust to minor scour. Oldest method in use for shielding footings.
- Disadvantages: Not reliable for stability. Except for large size stone, it can wash out easily in moderate floods; disturbs channel ecosystem until vegetation is reestablished. Maintenance and monitoring required before and after floods.
- Remarks: Not recommended for general bridge substructure use. Recommended for filling scour holes and bank erosion. Meets environmental requirements in NJ.

### **Interference from Underground Utilities**

Installation and maintenance of underground utility pipes close to a bridge foundation may cause significant disturbance to the soil. Detailed environmental investigation may be needed if utilities need to be relocated because of interference with a proposed countermeasure.



## CHAPTER 3

### GEOTECHNICAL CONSIDERATIONS

#### Introduction

The type of soil supporting a bridge foundation governs erosion potential and the long-term behavior of a foundation. Likewise, geotechnical conditions affect footing designs. Piers require deeper foundations due to a drop in the profile of a river located close to the thalweg. Hence, an abutment footing is generally located at a higher elevation than the pier footing. Existing NJDOT soil investigation data for the bridge site and US Geological Survey data may be used for an initial study of scour at an existing foundation. Special field conditions such as debris accumulation and formation of scour holes may further contribute to erosion. The condition of any 'unknown' foundation also needs to be determined. For new foundations, scour problems should be addressed through foundation design itself.

- A. The geology of the river bed and its banks:** This is one of the most important considerations when analyzing potential scour. 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.
- B. Geologic maps:** 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](#). See Figures 3.1 and 3.2 for geological subdivisions and distribution of soil types.

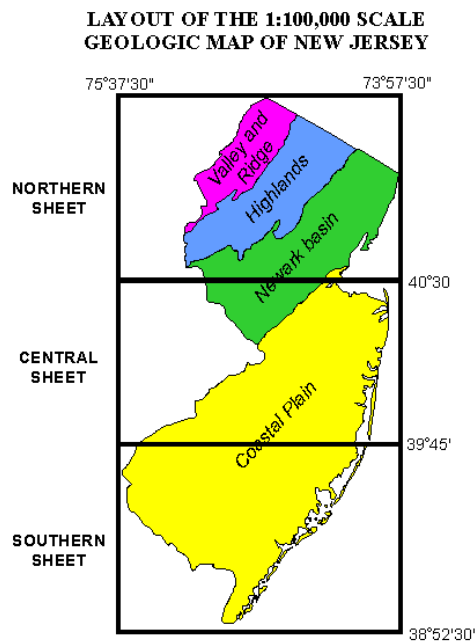


Figure 3.1: Geological subdivisions of New Jersey to identify soil types

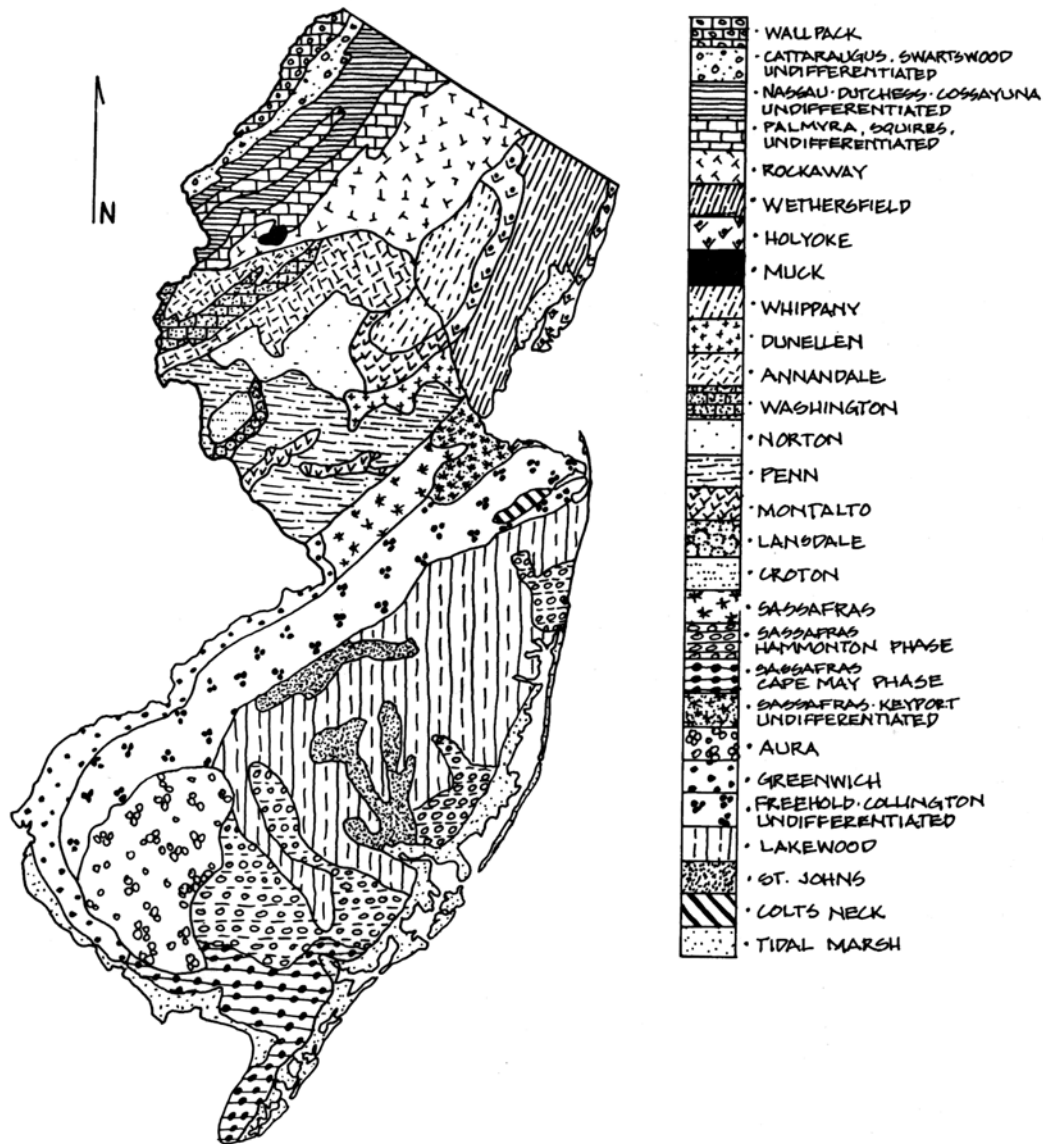


Figure 3.2: Soil Series of New Jersey As Presented By Tedrow (1986)

**C. Soil Types:** To determine grain size distribution for a scour analysis, samples from river bed need to be tested in accordance with ASTM D422.

Soil types may be broadly classified as:

a. Non-cohesive materials:

- 1.) Gravels
- 2.) Sands
- 3.) Silts

- b. Non-cohesive sediments have a granular structure. Individual particles are 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
  - 1) Particle size
  - 2) Density
  - 3) Shape
  - 4) Packing
  - 5) Orientation of bed material.
- c. Cohesive materials:

Most fine-grained sediments possess some cohesion. 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:

  - 1) Silts
  - 2) Clays

### **Scour in Cohesive Soils**

The clay content in soil increases cohesion and relatively large forces are required to erode the riverbed. Higher pulsating drag and lift forces increase dynamic action on aggregates until the bonds between aggregates are gradually destroyed. Aggregates are carried away by the flow. Briaud at Texas A & M University has proposed SRICOS method of scour measurement in cohesive soils [Briaud (2003)].

- 1. In cohesive soils such as clay, both local scour and contraction scour magnitudes may be similar. However, scour takes place considerably later than in the non-cohesive sand.
- 2. Scour analysis methods are different for cohesive and non-cohesive soils.
- 3. Bridge foundations supported by cohesive soils resist erosion for a much longer period than usually calculated, resulting in a longer life of bridge.

The bed material may be comprised of sediments (alluvial deposits) or other erodible materials. 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. Scour analysis of bridge piers and abutments in cohesive soils can be carried out on the basis of NCHRP 516 report entitled "Pier and Contraction Scour in Cohesive Soils" by Briaud et al (2004).

### **Soil Profiles for Typical Scour Critical Bridges in NJ**

The soil profile for a particular bridge site is based on boring logs. To document sample soil profiles at different bridge sites, a detailed review of NJDOT Phase II Scour inspection reports was carried out. Table 3.1 shows soil profiles for 23 selected bridges. *This data is for informational purposes only and should not be used for the design.*

Detailed geotechnical and borehole testing should be carried out to obtain site-specific information.

Table 3.1: Soil Profiles at Selected Bridge Sites in New Jersey.

Bridge Name	County	Soil type
US 322 over Hospitality Brook	Atlantic	Brown gravel little F/M sand trace silt (GP)
US 322 over Big Ditch	Atlantic	Gray F/M sand few silt (SM)
NJ 4 / Hackensack River & Access Rd	Bergen	Silt and muck (fine sand/silt)
ROUTE 63 over ROUTE 5 & Wolf Creek.	Bergen	N/A
NJ ROUTE 70 over Friendship Creek	Burlington	Gray fine sand trace silt (SP)
US206 over Assiscunk Creek	Burlington	Brown F/M sand trace silt (SP)
RT 73 over Pennsauken Creek	Camden	Gray silt little clay few fine sand (ML)
RT49 over Mill Creek	Cape May	Brown F/M/C sand and gravel trace silt (SP)
Berkeley Ave over Second River	Essex	Gray coarse/fine gravel with some coarse/fine sand (GP)
I 295 NB over Raccoon Creek	Gloucester	Brown F/M sand few silt trace gravel (SP-SM)
ROUTE 29 over Swan Creek	Hunterdon	Medium to fine Gravel and coarse to fin Sand, trace Silt
RT165 over Swan Creek	Hunterdon	Gravel and cobbles (SW)
ROUTE US 1 over Shipetaukin Creek	Mercer	Coarse to fine sand with varying proportions of silt and gravel (SM)
US ROUTE 130 over Rocky Brook	Mercer	Brown F/M sand trace silt (SP)
US9 over Deep Run Brook	Middlesex	Brown F/M sand trace silt trace gravel (SP)
US RT 9 over Milford Brook	Monmouth	Brown F/M sand trace silt (SP)
NJ ROUTE 10 over Malapardis Brook	Morris	Gray-brown c-f Gravel, little (+) c-f sand, trace (-) Silt
US 9 over Cedar Creek	Ocean	Brown gravel and F/M/C sand trace silt (GP)
Sicomac Rd over Molly Ann's Brook	Passaic	Gray coarse/medium sand with little gravel (SP)
I295 SB over Oldmans Creek	Salem	Dark grey hard gravely fine to medium sand (GP)
RT US 206 over Crusers Brook	Somerset	Medium to fine gravel and coarse to fine sand, trace silt
NJ.RTE.15 over Paulins Kill Creek	Sussex	Poorly-graded coarse to fine sand with some gravel, trace silt
RT 173 over Pohatcong Creek	Warren	Brown sandy silt

### Soil Types Affecting Scour

The percentage of soil types with scour problems are listed in Table 3.2. It is seen that sand foundations have 48% of scour problems while silt foundations do not display any scour problem.

Table 3.2: Soil types with scour problems.

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

### Soil Types Affecting Scour Depth

The intensity and duration of floods will affect the rate of scour in the soil. The following Table represents the duration of maximum scour depths in different soil conditions:

Table 3.3: Duration for maximum scour depths 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

Rocky soil is one of the most scour resistant materials; however, rock types wear down differently. The following Table 3.4 provides guidelines on assessing the erodibility of bedrock.

Table 3.4: Guidelines for Assessing the Erodibility of Bedrock.

<b>Rock property</b>	<b>Scour criteria</b>
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 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

### Countermeasures for Footings on Soil and Rock

Figures 3.3 and 3.4 provide Illustrations of Countermeasures on bridge footings on rock and soil. Theoretically, good quality bedrock will not require any countermeasure. However, irrespective of erosion, the footing should never be exposed. A layer of insulation of one-foot minimum is required to address anti-frost, thermal changes and vegetation growth. This may be in the form of soil with riprap or an alternate countermeasure at the top of a footing.

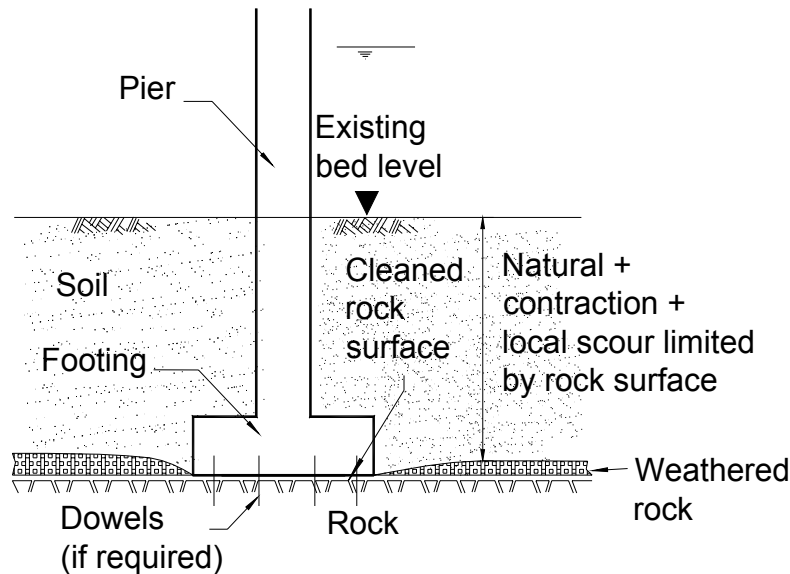


Figure 3.3: Spread footing on sound rock

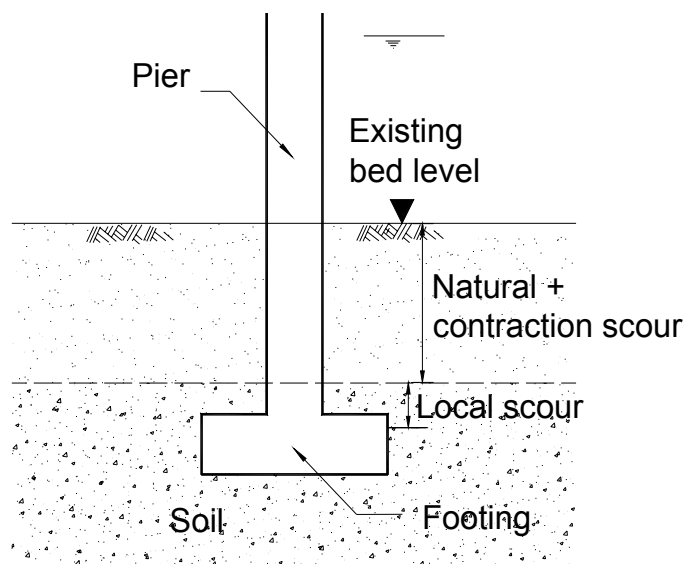


Figure 3.4: Spread footing on soil

## Debris Accumulation

Debris accumulation changes both the geotechnical and hydraulic characteristics of a bridge. Debris may accumulate at the upstream or downstream end of a bridge or under a bridge. Debris consists of indigenous material deposited at the bridge obstruction from continued floods and long-term aggradation. The material transported by water flow varies according to demography and the type of terrain.

In NJ, a great majority of bridges are located on narrow stream widths and therefore the effects of debris may be higher. It has been observed that sediments, small stones, gravel, leaves, tree branches, bushes, broken pieces of timber from furniture, boats etc. and fragments of metals may substantially block a bridge opening. Depending upon the size of debris accumulation, the waterway opening area may be reduced. Hydraulically, this reduction will increase contraction scour.

Debris accumulation in bridge openings may result in overtopping, failure of roadway embankments because of increased local scour at piers and/or abutments, and the formation of pressure flow scour. The blockage of an opening through the bridge structure causes significant increase in flow, which may increase the contraction scour at bridge footings.

Experimental studies have shown that debris accumulations cause larger and deeper scour holes. This happens as a result of the significant increase in the downward velocity below the debris and increase in both the horseshoe vortex size and the contact area of the vortex. The increase in both the contraction and local scour near the bridge structure could possibly damage or cause failure of the structure due to undermining of the pier footing or the abutment toe. Hence, removal of debris accumulations and the need for debris-control structures should be an essential part of any scour countermeasure design of a bridge.

Generally medium and large floating debris causes a significant problem at bridge structures because of their entrapment near bridge piers. Ice and debris can cause the water flow to plunge downward against the streambed and increase pier scour. Figure 3.5 shows large floating debris accumulation under a bridge along Lumberton-Vincetown road. Figure 3.6 shows debris accumulation under the Frenchtown-Uhlerstown Bridge in Frenchtown, NJ.

The effect of debris is greater at pier footings than at abutments. The effect is similar to freezing of ice around a pier and ice forces due to impact from moving ice. However, since the type and magnitude of debris vary for each bridge site, it may not be possible to develop an analysis approach similar to floating ice for the analysis of debris loads. HEC-18 based approach to calculate pier scour depths because of debris accumulation is presented in Figure 3.7.





Figure 3.5: Large Floating Debris Accumulation under Lumberton-Vincetown road during July 13-14 2004 flooding



Figure 3.6: Debris accumulation under Frenchtown-Uhlerstown Bridge in Frenchtown, NJ during June 28-30, 2006 flood.



## Interim Procedure for Estimating Pier Scour with Debris

### D.1 ASSUMPTIONS

1. Debris aligns with the flow direction and attaches to the upstream nose of a pier. The width of the accumulation,  $W$ , on each side of the pier is normal to the flow direction.
2. The trailing end of a long slender pier does not add significantly to pier scour for the portion of the length beyond 12 pier widths. This is consistent with the current guideline in HEC-18 to cut  $K_2$  at  $L/a=12$ .
3. The effect of the debris in increasing scour depths is taken into account by adding width,  $W$ , to the sides and front of the pier. Engineering judgment and experience is used to determine the width,  $W$ .

### D.2 SUGGESTED PROCEDURE

1. Use  $K_1$  and  $K_2=1.0$
2. Project the debris pile and up to twelve pier widths of the pier length normal to the flow direction as follows:

$$L' = L \text{ or } 12(a) \text{ (whichever is less)}$$

$$a_{proj} = 2W + a \cos\theta \text{ or } W + a \cos\theta + L' \sin\theta \text{ (whichever is greater)}$$

3. Use  $K_1$ ,  $K_2$ ,  $K_3$ ,  $K_4$  and  $a_{proj}$  in the HEC-18 pier scour equation as follows:

$$\frac{y_s}{y_1} = 2.0(1.0)(1.0)K_3K_4\left(\frac{a_{proj}}{y_1}\right)^{0.65}F_r^{0.43}$$

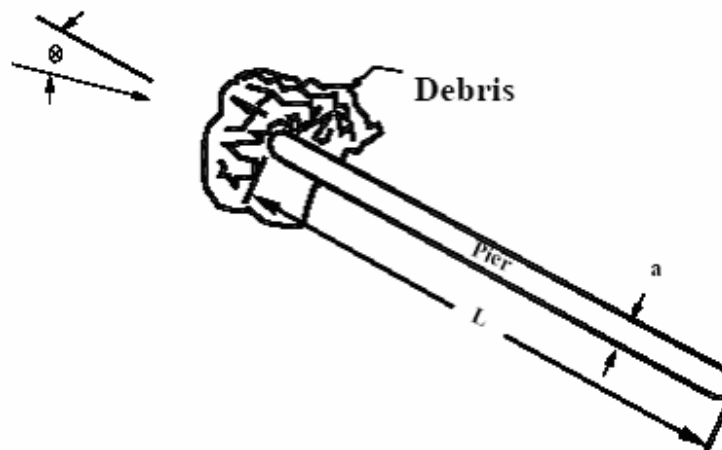


Figure 3.7: Procedure for Pier Scour with Debris.

### **Countermeasures for Debris Accumulation**

The most-effective countermeasure against debris accumulation during high-flow flood is monitoring and debris removal. A bridge monitoring crew should closely monitor signs of debris buildup, especially near the lower chord. If a large difference in the water surface elevation is noticed from the upstream to the downstream side of the bridge, it may be a sign of a severe debris blockage beneath the water surface. The structural loading caused by this situation could lead to failure of the bridge.

Removing debris from the nose of a pier, for instance, can prevent significant additional scour during that flood event. Debris removal during high flow is usually accomplished from the bridge deck using a bridge inspection truck, crane, or excavator arm. Alternative site-specific approaches may be needed at bridges with a large bridge deck overhang because of the difficulty in reaching the pier nose under the deck and in pulling the debris away from the pier against the flow.

The following measures are recommended to address debris accumulation problems in New Jersey:

- a) A flood watch and inspection immediately after a flood to evaluate the extent of debris and litter deposit at a bridge site.
- b) Cleaning up of the site and removing all such debris.
- c) Dredging of the river bed to pre-flood conditions.
- d) New bridges: The bridge superstructure and the general elevation of approach roadways should be above the maximum flood level of a 100 year or the critical flood.
- e) For streams that carry a large amount of debris, the elevation of the lower chord of the bridge should be at least 2 ft. above the free board for a 100-year flood.
- f) If there is a hazard of ice and debris buildup, multiple pile bents should be avoided. For scour estimation, bent pier should be evaluated as a solid pier.
- g) Use of debris deflectors: Specially designed debris deflectors can be installed on the upstream side of bridge.

### **Formation of Scour Holes at Bridge Sites**

Scour holes may evolve by progressive initiation, development, stabilization and equilibrium. During initiation phase, erosion is rapid and material is eroded to form scour hole. During development phase, depth of scour hole increases. During stabilization phase, maximum length of hole occurs. In equilibrium phase the dimensions of the hole are virtually fixed. These are usually created by turbulence of flow and formation of eddy currents near the piers. Debris accumulation upstream and downstream of a bridge also contributes to the formation of scour holes. In some cases, these holes may close over a long period by internal soil bed movement and aggradation. Some important aspects of the scour hole formation process are:

- They are more likely to be formed in soft silty soils.
- The location and size of any scour holes must be determined. Sizes may typically

vary between 2 to 5 feet in depths, while widths are wider and would depend upon peak flood velocity and turbulence.

- Scour holes close to the footings may continue to erode and enlarge in area and may ultimately lead to foundation instability. Several small holes may enlarge to form a single large scour hole.
- Although they are filled up with water, they need to be completely filled with a similar soil such as river bed material or riprap. DEP may not issue a permit for using concrete or a filling material due to chemical contamination and adverse effects on surrounding vegetation.

Fig. 3.8 shows an example of construction of a pier on a scour hole in the case of twin bridges. There is a concentration of flow at the upstream pier because of obstruction to flow from construction of adjacent bridges, enlarging the size of scour hole. Figure 3.9 details a scour hole formed because of lateral movement of a main channel, exposing pier foundations. Figure 3.10 shows bridge failure due to settlement of foundation in scour hole resulting from deficiencies in foundation design and soil conditions.

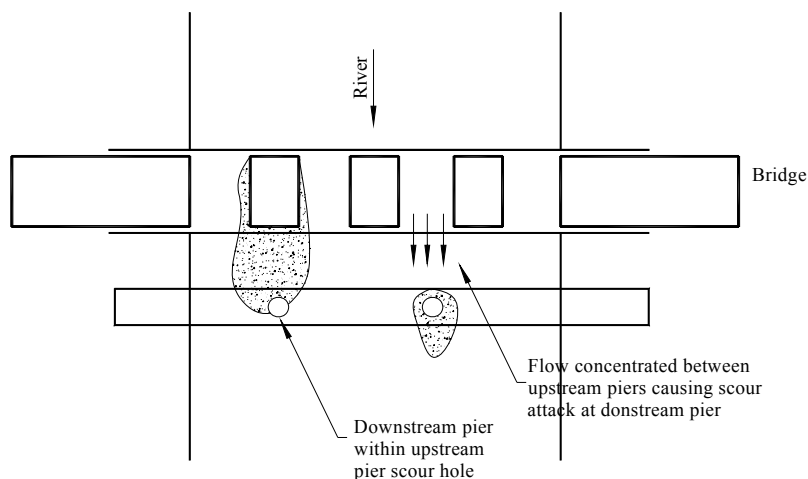


Figure 3.8: Location of downstream pier within the scour hole at upstream pier.

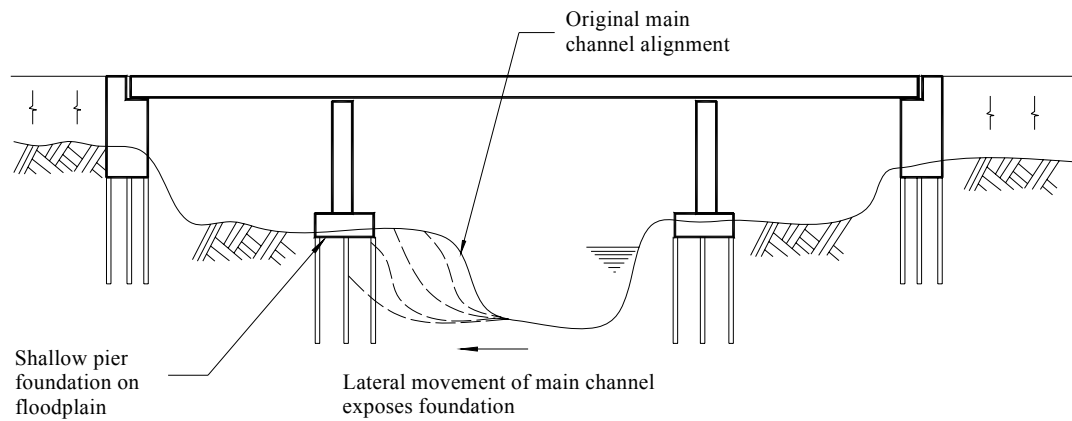


Figure 3.9: Scour hole formation and lateral movement of main channel



Figure 3.10: Bridge failure due to settlement of foundation in scour hole resulting from deficiencies in foundation design and soil conditions.

## Unknown Foundations

Unknown foundation may be defined as an existing buried foundation for which no drawings, including as-built drawings from the time of construction, are available. It may be defined as

- A foundation whose type and elevation is unknown
- A foundation whose type is known but its bottom elevation is unknown

Such foundations include

- Short one span bridge abutments on land. "Dig and see" method is cost effective for such foundations.
- Piers of multiple span bridge on land or water. Borings and NDT methods are generally used for such foundations.

For scour protection design, the physical size and type of foundation needs to be investigated. The following approaches are used to determine unknown foundations:

- Conventional methods such as probes, hand augers, power augers, rotary drilling methods and digging pits adjacent to footing.
- Deductive methods for deep foundations which rely on geotechnical analysis, obtaining samples of subsurface strata using Standard Penetration Method (SPT) ASTM D-1586, Cone Penetrometers, evaluating density of dense layers.
- NDT Methods such as Surface NDT and Borehole NDT methods.

Surface methods include

- Pulse Echo (compression wave echo from stiffness change)
- Bending Wave Method
- Ultra-Seismic Method
- Surface Waves Spectral Analysis
- Ground Penetration Radar or Impulse Radar

Borehole NDT methods include

- Parallel Seismic Method (Measurement of compression and shear wave emitted by foundation impact)
- Pulse Echo in Borehole
- Borehole Radar Method

Usually, the work is performed by specialist geotechnical contractors. In NJ, common methods are digging pits adjacent to footings or drilling boreholes through the approaches to abutments, backfill and footings and through deck slab adjacent to piers. The actual type of NDT method chosen should be project specific depending on access, foundation configuration, nature of subsurface soils, and the skills and equipment of the

practitioner, etc. It should be noted that the state of the art for NDT methods is still evolving and interpretation is still somewhat subjective based on the interpreter's judgment and experience. Hence, NJDOT project engineer and consultants should consider latest information/resources in selecting appropriate NDT approach. Additionally, the following four reference sources may be used for more information on unknown foundations:

1. FHWA Geotechnical Engineering Notebook, Geotechnical Guideline No. 16, "Summary of Interim Report for NCHRP Project 21-05 "Determination of Unknown Subsurface Bridge Foundations" by Olson, Jalinoos and Aouad."
2. "Unknown Subsurface Bridge Foundation Testing" by Olson, Larry D. and Marwan F. Aouad, unpublished draft final report for NCHRP project 21-05 & 21-05(2), National Cooperative Highway Research Program, Washington D.C., Sept. 2000.
3. Guideline for "Nondestructive Evaluation of Unknown Subsurface Bridge Foundation Depths", NCHRP Project 21-5(2), National Cooperative Highway Research Program, June 2001.
4. Victor Elias. *Strategies for Managing Unknown Bridge Foundations*. FHWA-RD-92-030, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C. February 1994.

## CHAPTER 4

### APPLICATION OF MODERN TECHNIQUES TO CONTAIN EROSION

#### Introduction

A scour countermeasure is defined as a measure incorporated at a stream/bridge crossing to monitor, control, inhibit, change, delay, or minimize stream and bridge stability problems because of scour. Table 4.1 shows commonly used scour countermeasures. Table 4.2 shows a description of other types of commonly used countermeasures.

Table 4.1: Commonly used Scour countermeasures.

Countermeasure	Description
Monitoring	Under water inspection by divers or using remote sensors
Rock riprap	Dumped or broken rock
Extended footing	Structure to support the slope or protect it from erosion
Gabions/Reno mattress	Wire mesh baskets, mattress filled with loose stones
Guide Banks (spurs/dyke)	Structure to support the slope or protect it from erosion
Pavement/Channel lining	Reinforced Concrete/bituminous concrete pavement to channel bed/banks
Bridge closure	Temporary detour of traffic during construction
Sacked concrete	Fabric bags filled with concrete and stacked to produce a protective layer
Check dams	Installing sills or drop structures
Artificial riprap	Alternatives to riprap such as tetrapods / toskanes
Concrete filled mat	Porous fabric bags filled with high strength mortar
Jetties	Walls to support the slope or protect bank from erosion
Flexible Revetment	Artificial Armoring
Precast concrete blocks	Concrete blocks of a cellular shape placed as revetment
Retard (timber & sheet piles/Trees)	Wall to support the slope or protect it from erosion
Concrete grouted riprap	Standard riprap with concrete grout
Sacrificial piles	Steel, timber or concrete piles driven upstream to reduce velocity
Soil cement	In-place soil stabilized with cement
Flow deflecting plates	Plates connected to piers to deflect flow
Cable-tied blocks	Concrete blocks /slabs interconnected with steel cables
Braced Piles	Piles braced together in transverse direction
Increase Span/ Relief Bridge	Increase the opening by reconstructing abutment / Provide a new opening by adding span at approaches
Vanes	Obstructions placed upstream to redirect or reduce flow
Tetrapods	Artificial concrete blocks

Table 4.2: Description of other types of countermeasures.

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

### Armoring Countermeasures

Armoring is the most commonly used method in New Jersey and throughout USA. Hydraulic Countermeasures are primarily designed either to modify the flow or resist erosive forces caused by the flow. They are organized into two groups: river training structures and armoring. Their performance 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, it is seen that a majority of bridges located on rivers will be subjected to 100-year flood velocity (resulting generally in peak flood velocity of 6 to 8 ft/sec or less). If the peak flood duration is not long enough to cause extensive erosion, armoring countermeasures should be sufficient for velocities of magnitude between 6 to 8 ft/sec or less.

Armoring is distinctive because they resist the erosive forces caused by a hydraulic condition. Armoring countermeasures do not necessarily alter the hydraulics of a waterway, but act as a resistant layer to hydraulic shear stresses providing protection to more erodible materials underneath. They generally do not vary by function, but vary more in material type. Armoring is classified by two functional groups: revetments and bed armoring, and 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 aerial 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 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 erosion by vortices created by obstructions to the flow. Generally, the same material used for revetments and local armoring. Some important aspects of local armoring are:

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 most commonly used 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, pile 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 failure of a riverbank or an embankment, upstream or downstream of bridge, may lead to change in the direction of flow of river and increased scour at the bridge due to change in hydraulic conditions. The primary function of armoring is reduction in velocity and energy dissipation, both upstream and downstream of the bridge.
3. Erosion of banks immediately upstream and downstream is likely to contribute to increased erosion of bridge foundations. The following off bridge and off culvert locations are likely to be eroded after a major flood. Revetment may be required at such locations, e.g.,
  - 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 are
  - a. Rock riprap
  - b. Concrete pavement or concrete apron
  - c. Concrete armor units
5. Armoring around wing wall: The height of a wing wall decreases at locations away from the bridge because of reduced severity of scour. 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, the same type of armoring as for abutments should be used.

6. Armoring Countermeasures Combined With River Training: Experience has shown that mandatory-armoring countermeasures alone may not be adequate and a combination of optional river training measures and armoring may necessary for the higher 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. Since large investments would be involved, economic considerations become important. Hence, cost reductions may 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 more economically.

It is normal practice to protect 100 to 300 feet of riverbanks by revetment at the 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 has been used for revetment. Filters 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 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 considered, as complementary to armoring when:

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

### **Countermeasures Alternative to Temporary Riprap**

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. In certain situations, required riprap sizes may not be readily available. Hence, the following countermeasures may be used over the conventional use of 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. Grade control structures
  - f. 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.

*Flow altering countermeasures should only be used in combination with primary countermeasures to improve their effectiveness.*

### **Exceptions to Providing a Full-Scale Countermeasure**

An exception may be obtained from the Department, if it can be established that providing a scour countermeasure will not be cost effective. A minimum countermeasure or certain modifications to the recommended guidelines for full-scale countermeasures may be proposed. The modifications may take the form of deviations or a waiver from the countermeasure guidelines. While meeting the safety demands of the bridge, the rationale for such modifications would include any unusual constraints, low traffic volume, low functional classification, low risk bridge and economic considerations. The following criteria or bridge type identification may be used to justify an exception approval.

- a. Traffic volume: Deviations may be obtained in the decreasing order of importance according to AADT.

Importance Type (Increasing order)	AADT
Type 1 bridge	< 4,000
Type 2 bridge	> 4,000 < 25,000
Type 3 bridge	> 25, 000

- b. Functional Classification: Deviations may be obtained in the decreasing order of importance according to functional classification.

Importance Type (Increasing order)	Feature Carried
Type 1 bridge	Local
Type 2 bridge	Collector
Type 3 bridge	Arterial
Type 4 bridge	Interstate

c. Low Risk Bridge

- 1.) That has only a few years of remaining service life.
- 2.) That can be closed due to available alternate routes or available detours.
- 3.) That has limited vertical clearance, where it is difficult to install formal countermeasures effectively.

d. Low Scour Bridges

- 1.) That are provided with overflow relief
- 2.) That are not subjected to backwater
- 3.) That are located away from confluence

e. Types of Channel Configuration: Deviations may be obtained in the decreasing order of importance according to channel configuration. Channel configurations in the increasing order of importance are:

- 1.) Straight
- 2.) Braided or multi-channel
- 3.) Meandering

f. Types of Channel Bottom: Deviations may be obtained in the decreasing order of importance according to channel bottom. Channel bottoms in the increasing order of importance are:

- 1.) Stable
- 2.) Aggrading
- 3.) Degrading

The detailed design of countermeasure is a site-specific problem. In proposing deviation for a countermeasure, 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

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

## CHAPTER 5

### COUNTERMEASURES FOR ABUTMENTS, PIERS, ARCHES & CULVERTS

#### Footing Types and Scour at Abutments

Structures located on streams, which are subjected to scour include highway bridges, railway bridges, movable bridges, swing bridges, arch bridges and culverts. In practice, there are a large variety of scour critical bridges with significant variations in the types of footings. The type of countermeasure required would vary for each type of footing. Examples of different types of spread footings for which scour protection is required are shown in Figure 5.1.

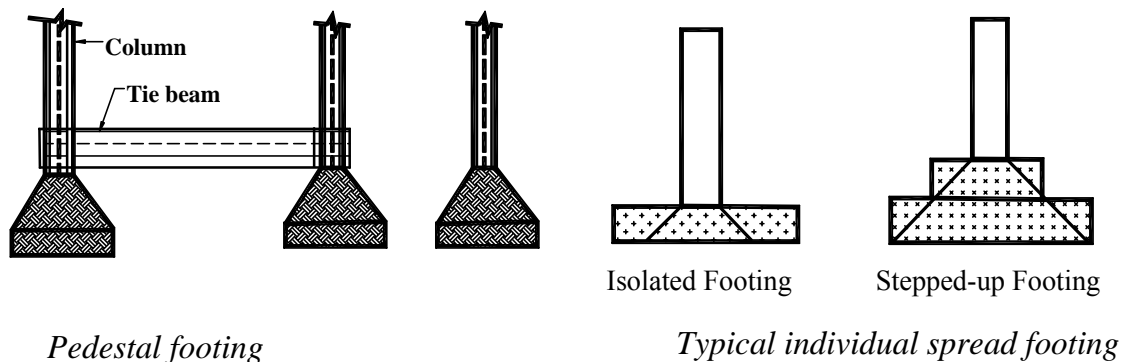


Figure 5.1: Examples of Spread Footings for which scour protection is required.

Bridge abutments and their approach embankments are the most commonly damaged bridge components during floods. The type of footing, whether shallow or deep, has an influence on scour depth and the type of countermeasure required.

Two typical approaches for protecting bridge abutments from scour are:

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

#### Recommended Countermeasures for Abutments

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

##### 1. Local Scour Countermeasures

- a. Riprap

- b. Gabions
- c. Grout / cement Bags
- d. Cable-tied blocks

## 2. RIVER CONTROL COUNTERMEASURES

- a. Guide Banks
- b. Spur Dikes

### Typical Pier Footings

Figures 5.2 to 5.12 show configurations of typical bridge piers, the types of pier footings, typical strengthening measures and types of failures encountered due to soil erosion and settlement. Figures 5.4 to 5.8 show typical foundation arrangements during the planning stage. A combined spread footing will provide greater stability against settlement than, for example, a pile bent. Figures 5.4 to 5.8 are placed in the order of decreasing resistance to long term scour offered by each foundation type. Geotechnical recommendations however need to be followed in selecting the type of footing and the applicable scour countermeasure.

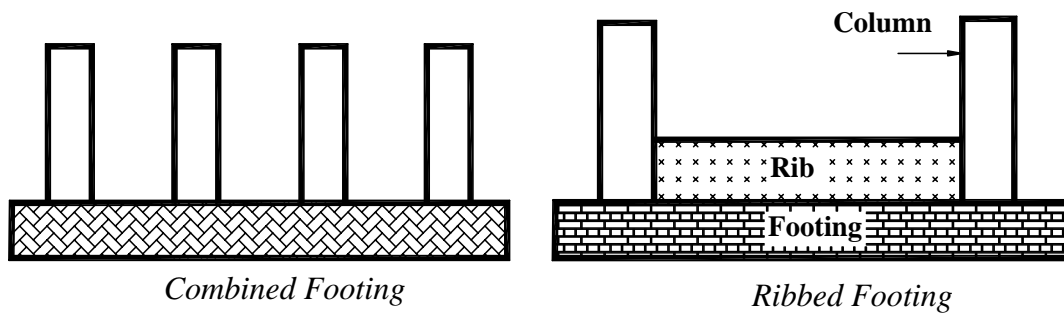
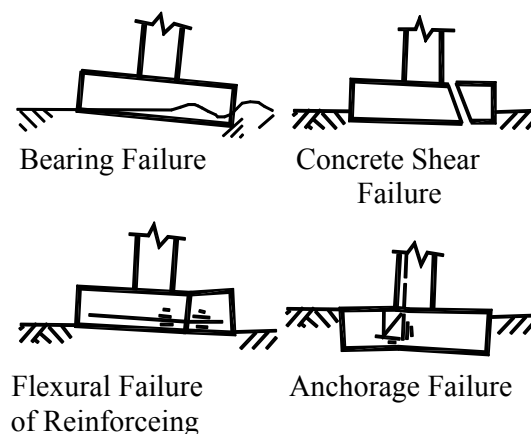
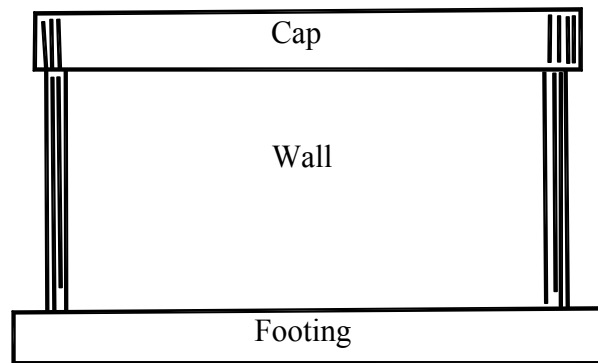


Figure 5.2: Strengthening a combined footing with a concrete rib.



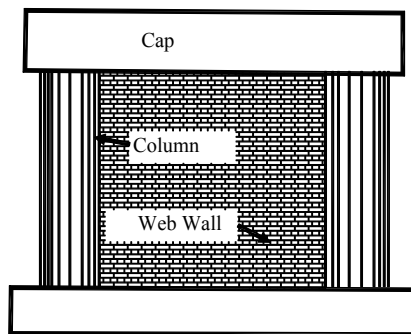
*Typical spread footing failures*

Figure 5.3: Types of footing failures encountered due to soil erosion and settlement.

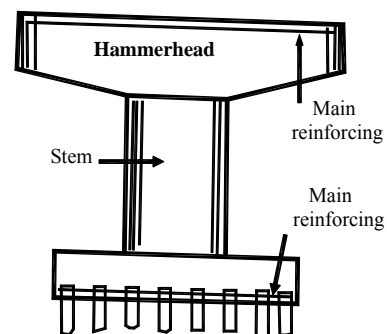


*Solid Wall Pier*

Figure 5.4: Use of Conventional Wall Type Pier – Spread footing is highly stable against settlement or rotation against soil erosion.

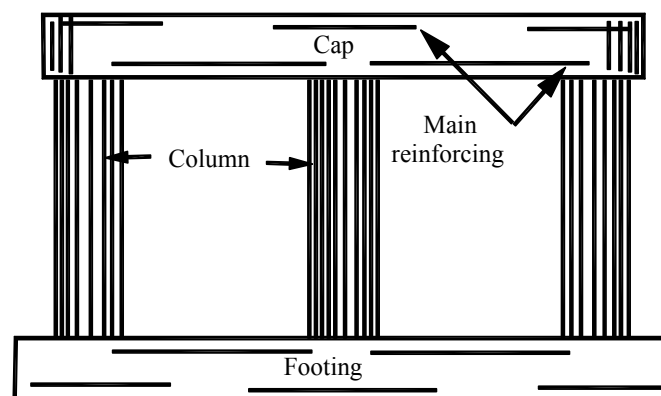


*Dumbbell Pier*



*Hammerhead Pier*

Figure 5.5: (a): Dumbbell Type Pier on spread footing is inferior to solid wall pier in resisting scour; (b): Hammerhead Type Pier on pile rows or pile groups is inferior to dumbbell piers in resisting scour.



*Column Bent Pier*

Figure 5.6: Round columns integral with pier cap on combined spread footing are more susceptible than Hammerhead piers.



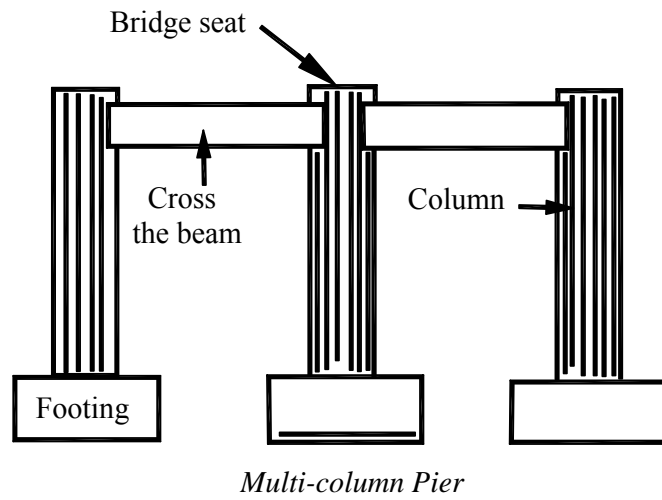


Figure 5.7: Commonly used independent round column piers on separate footings are inferior to round columns integral with pier cap in Figure 5.6.

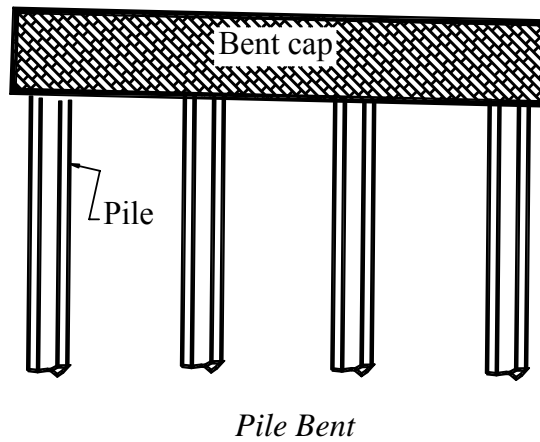


Figure 5.8: Bridge supported on long piles - Erosion at pile bents will increase the long column effect and reduce safe axial capacity of piles.

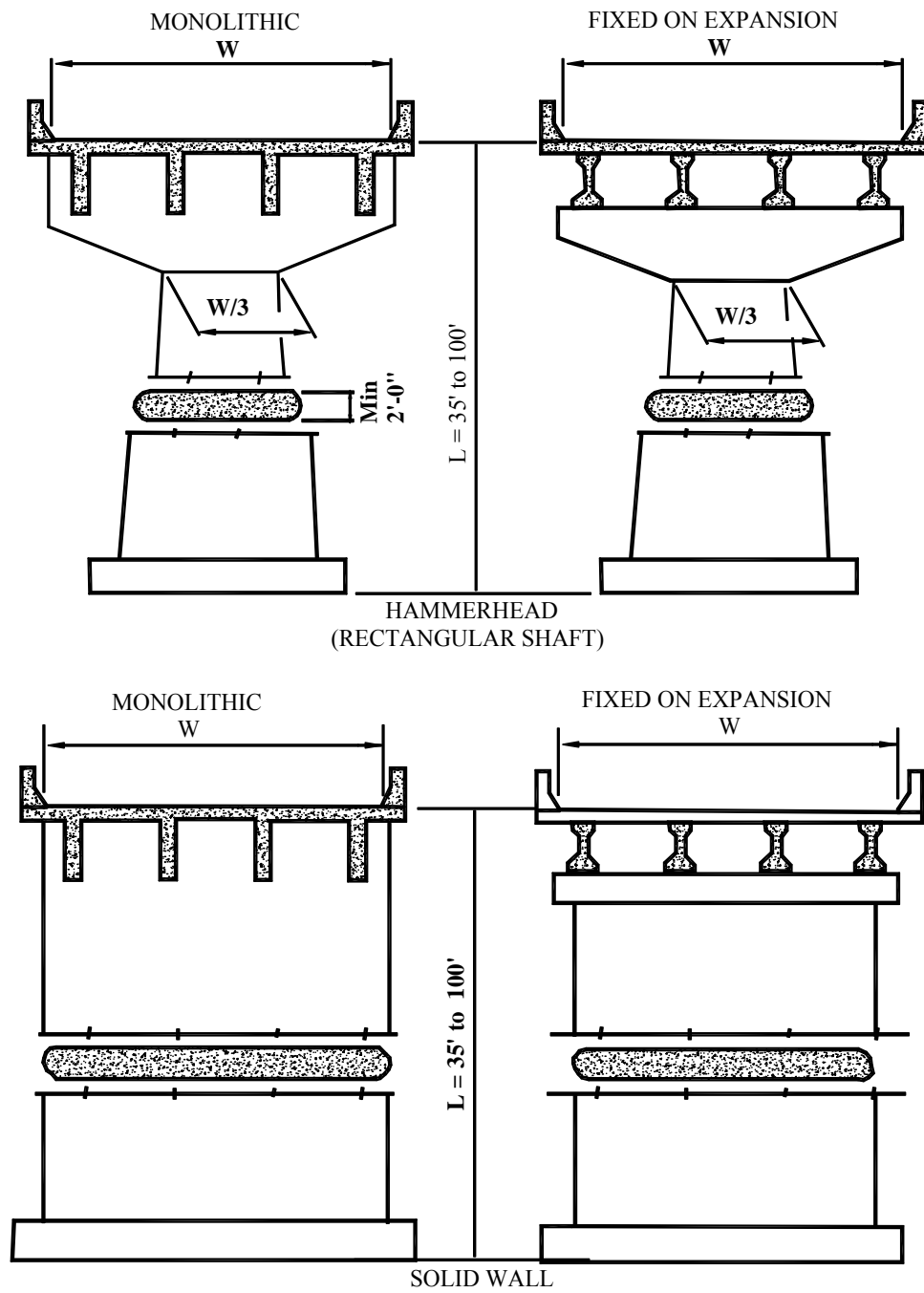


Figure 5.9: Rounded nose of pier helps to reduce local scour.

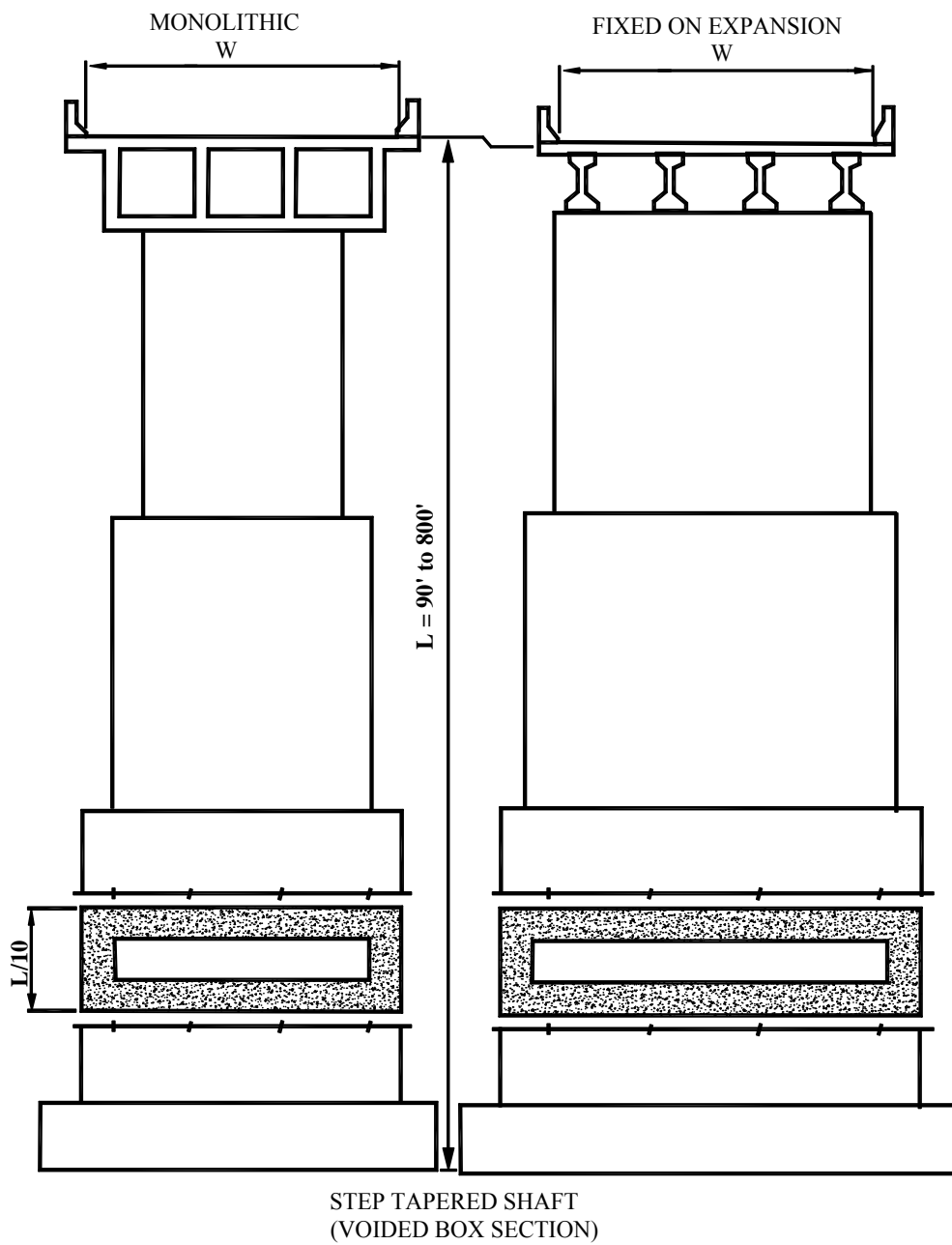


Figure 5.10: Rectangular sharp edges of pier lead to greater scour depth.

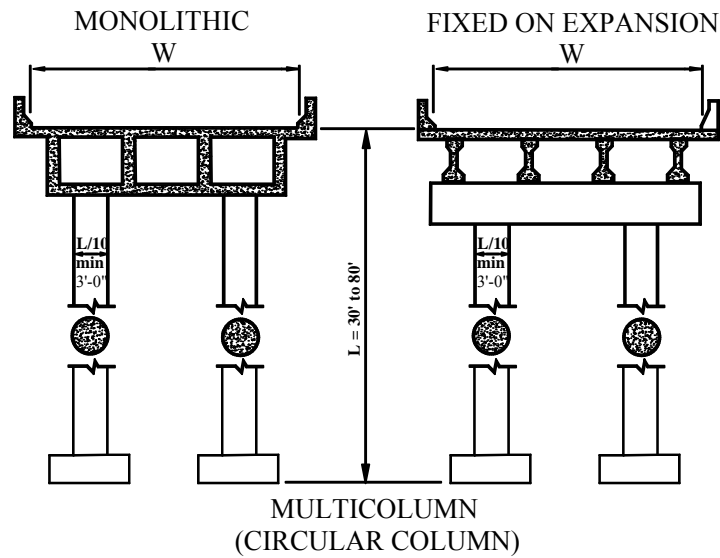


Figure 5.11: Isolated footings of columns are exposed to greater scour.

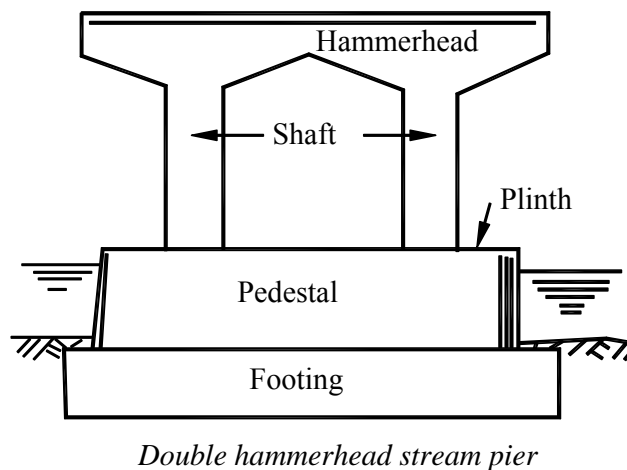


Figure 5.12: Use of a Pedestal braces the footing and prevents unequal foundation settlement.

### Scour at Piers

Isolated footings of piers in Figure 5.11 generally increase the vulnerability of the bridge to failure because of scour. Figures 5.13 to 5.17 detail the causes and locations of scour at pier foundations at typical bridge sites. Figure 5.13 details a situation when change in channel alignment at bends and skewed flow at piers causes scour at outer abutments and piers. Figure 5.14 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.

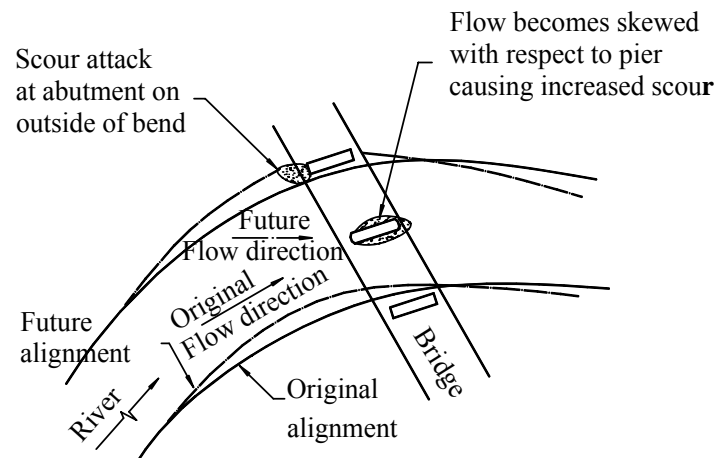


Figure 5.13: Changes in Channel Alignment at Bend Causing Scour at Outer Abutment and Skewed Flow at Pier.

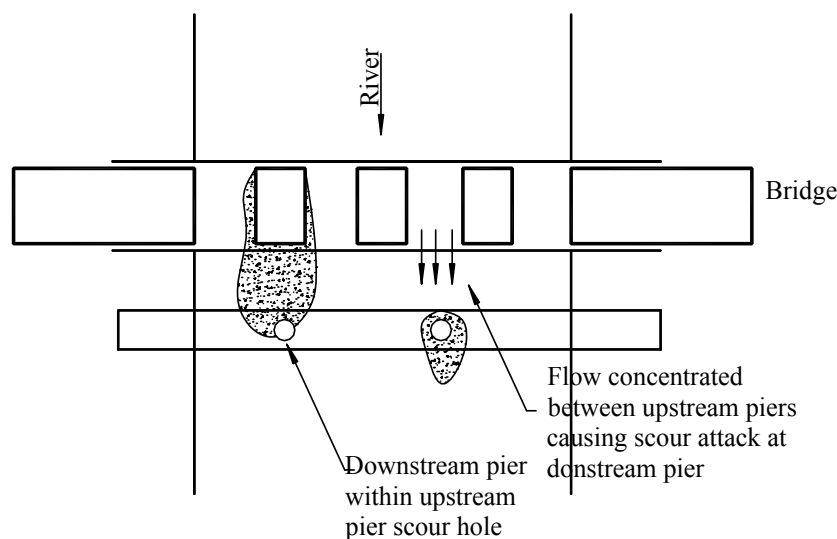


Figure 5.14: Downstream pier within scour hole of the upstream pier and concentration of flow at upstream pier because of construction of adjacent bridges.

Figure 5.15 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 5.16. Removal of downstream flow control devices, such as weirs, may result in gradual degradation of a stream bed. This may expose both pier and abutment foundations over a period of time, as shown in Figure 5.17.

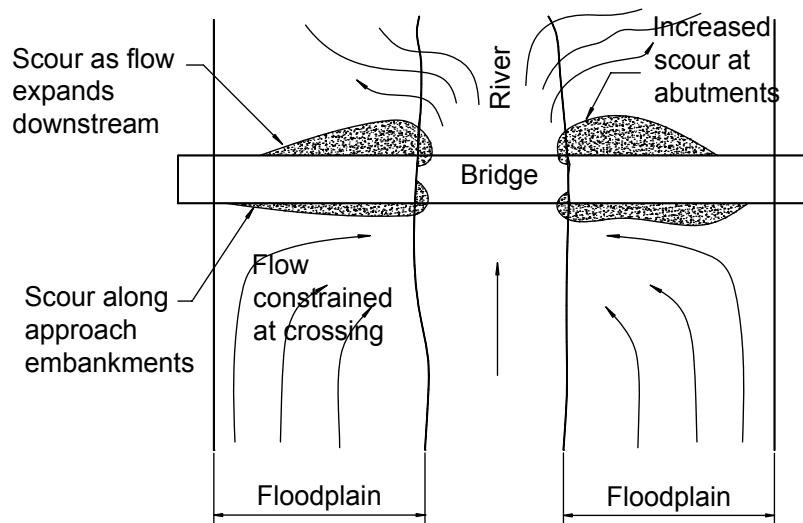


Figure 5.15: Lack of relief openings on floodplain

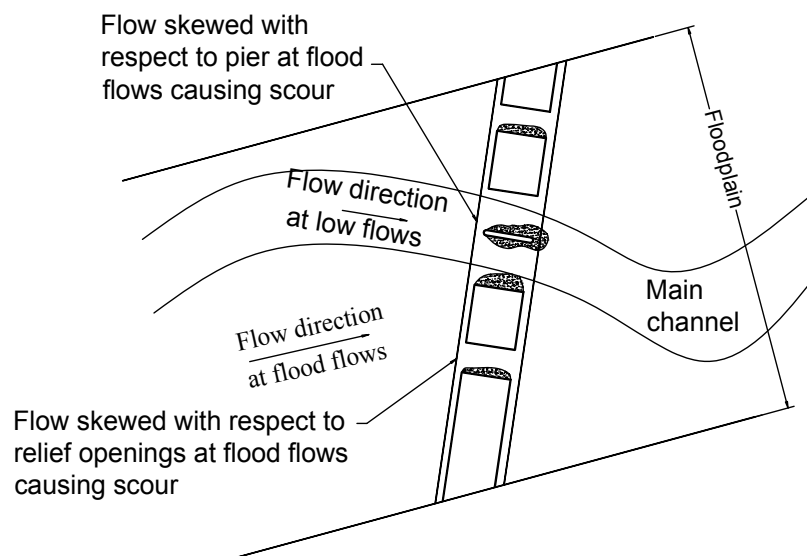


Figure 5.16: Change in flow direction between normal and flood flows resulting in skewed flows at piers, abutments and relief openings.

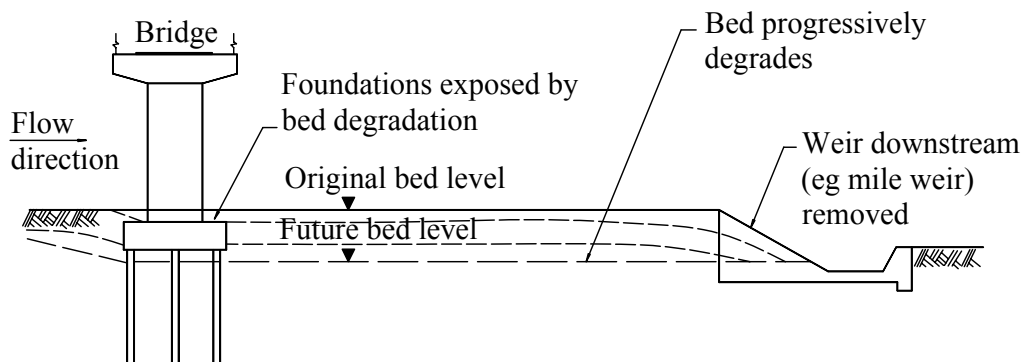


Figure 5.17: Removal of downstream bed control

### Recommended Pier Scour Countermeasures

The following pier countermeasures are recommended for applications to New Jersey bridges:

1. Armoring Countermeasures
  - a. Riprap
  - b. Gabions and Reno Mattresses
  - c. Articulated Concrete Blocks/Cable Tied Blocks
  - d. Concrete Armor Units
2. Flow Altering Countermeasures
  - a. Upstream Sheet piles
  - b. Flow Deflecting Vanes or Plates

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

3. Structural Countermeasures
  - a. Structural Repairs using Tremie concrete
  - b. Grout bags
  - c. Casting concrete aprons
  - d. Shielding by sacrificial piles
  - e. Sheet piles

Guidelines on applications of each of these are detailed in this Handbook.

## Scour Protection at Arches & Culverts

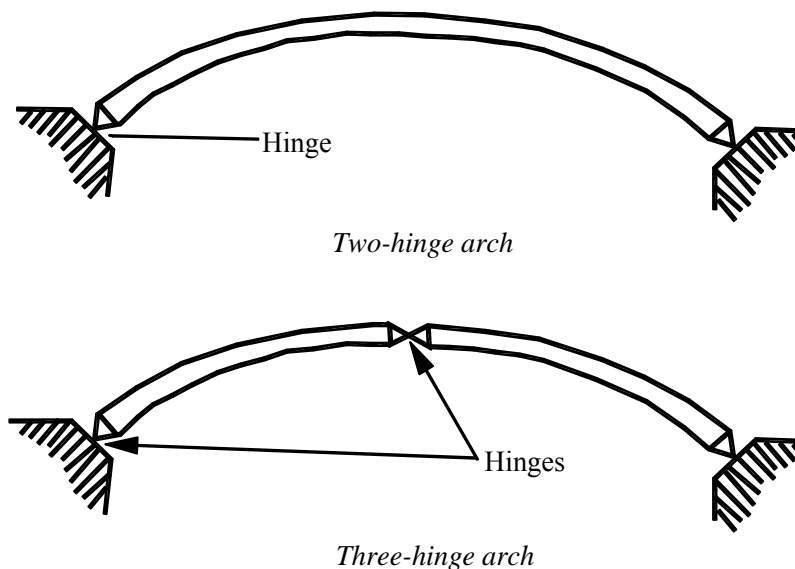
### Hydraulic Design

Chapter 9 of 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.

### Types of Arches

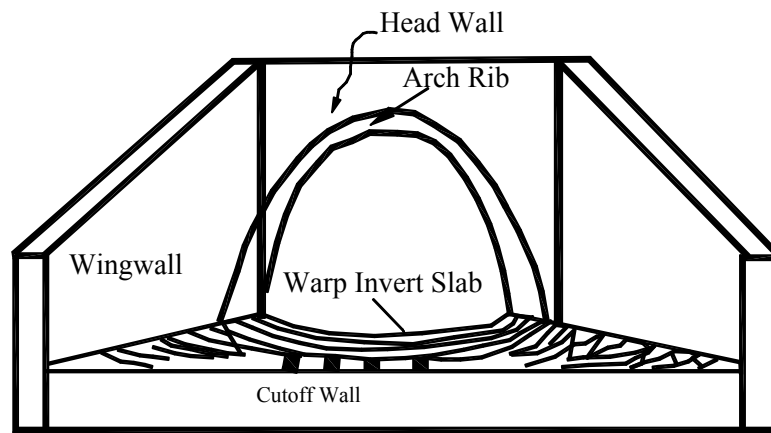
Arches were frequently used in the past. Arch Culverts are usually made of stone and brick. Figure 5.18 shows two and three hinged arches used as culverts. These arches are either:

- Open arch shaped structures, which are anchored into concrete footings, are of semi-circular, horseshoe or elliptical shapes. Compaction of embankment has an influence on the shape of arch. Figure 5.19 illustrates the use of arch shaped culvert. Figure 5.20 shows open spandrel arch type culvert in which the use of special anchored footings are required to resist erosion.
- Close pipe shape structures, which are of smaller radius than open arches, are likely to change their alignment during floods. Reinforced Concrete Pipe Culvert and Metal Pipe Culvert are commonly used.



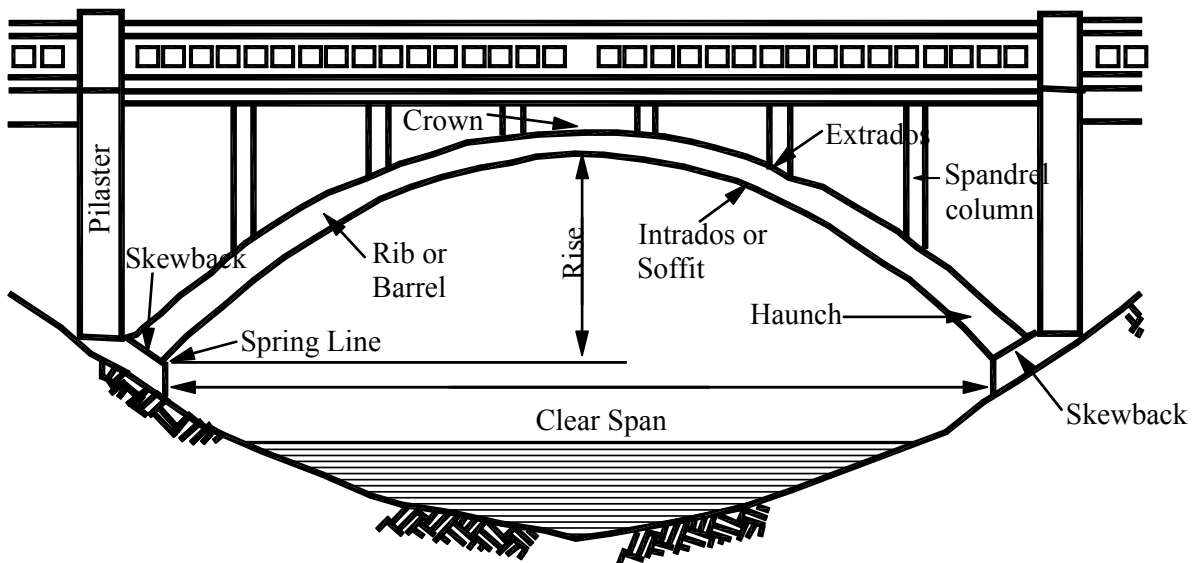
Figures 5.18: Use of two and three hinged arches as culverts





*Arch Culvert*

Figure 5.19: Use of arch shaped culvert.



*Open spandrel arch*

Figure 5.20: Open Spandrel Arch Type Culvert.

- Both types of arches may have wingwalls. Failure of arch foundations due to scour may lead to the collapse of the arch bridge. This may happen if:
  - Footings are cast into un-compacted or disturbed soils. Probing rods should be used to check for voids and scoured areas that may be filled with sediment.

- If the river meanders or overtops and erodes part of embankment backfill.
- Differential settlement is also a common cause of foundation movement from scour. Arches with no invert slab need to be inspected for erosion, undermining of footing and any indication of footing rotation.
- An inadequate drainage system may saturate the earth fill thereby increasing the dead weight on arch and opening mortar joints. Weep holes or lateral drainage pipes may be used.

### **Types of Culverts**

Based on material and shape, culverts are of the following types

1. Reinforced Concrete Box Culvert
2. Precast Bent Frames with footings

Figure 5.21 shows a typical commonly used concrete box culvert.

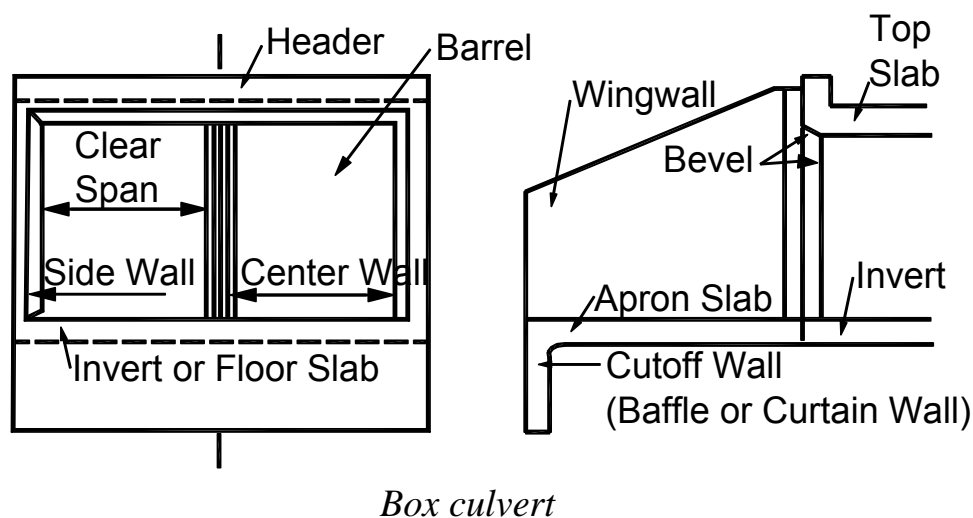


Figure 5.21: Commonly Used Box Culvert.

Causes of scour at culverts:

1. If a culvert is blocked with debris or the stream changes course, the culvert will be inadequate to handle design flows.
2. Poor culvert location
3. Changes in upstream land use such as real estate development, deforestation, clearing or demography.
4. Inadequate design or construction of culvert

5. Changes of slope, flow velocity, width and depth of channel and invert elevation

These may further result in excessive ponding, washing out of roadway embankment and flooding of nearby properties.

### **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

Figure 5.22 shows concrete apron and other end treatments to minimize scour of culverts.

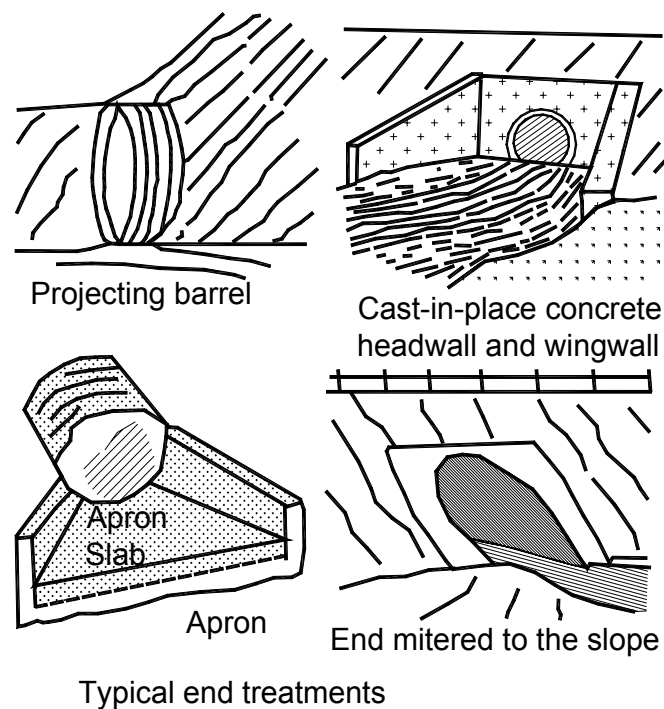


Figure 5.22: Use of concrete apron and other end treatment to minimize scour.

### **Scour at Inlet**

Under inlet control, the capacity of a culvert is controlled at the entrance. Most culverts are designed to operate under inlet control.

Blocking of a culvert entrance is a common cause of culvert failure. Scour takes place due to turbulence when more water is collected at the inlet than can be rapidly discharged. The causes are that waterway opening is too small and debris accumulation at inlet.

Factors affecting inlet control are

- Configuration of inlet edge
- Barrel shape
- Cross sectional area
- Headwater depth

If a waterway opening is too small, poorly located due to skew or a culvert barrel is choked with sediment (Figure 5.23), debris or brushes aggradation will result at the inlet. Turbulence of water is likely to occur, sometimes leading to culvert failure.

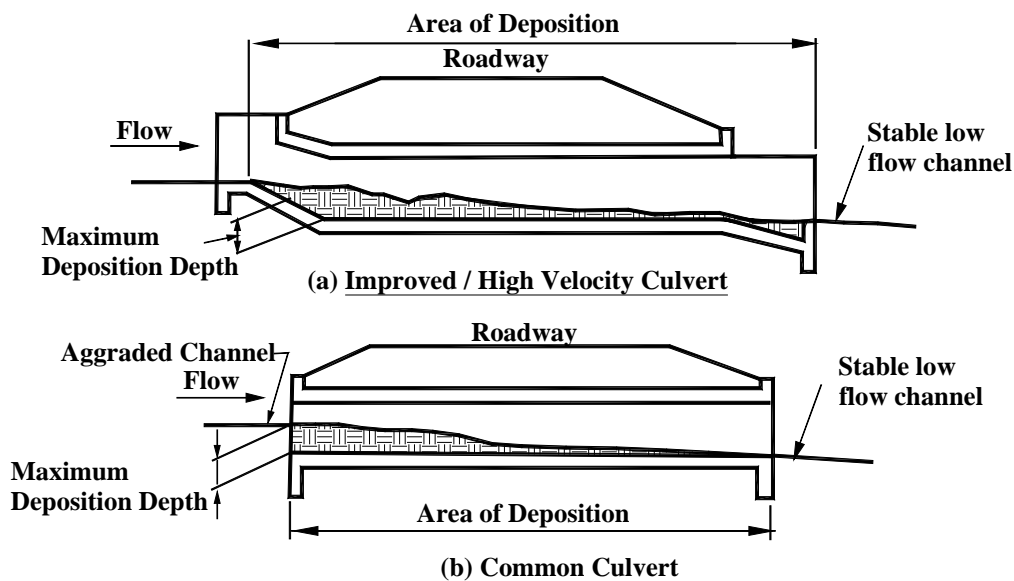


Figure 5.23: Culvert Sediment Deposition.

### **Scour at Outlet**

Culverts operating with outlet control usually have high tail water and usually lie on flat slopes. Water velocity is lower at the outlet. Changes in tail water depth or barrel characteristics may affect capacity.

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. Due to steep slope, velocity is increased and discharge at the outlet increases.

### **Countermeasures for Culverts**

The following countermeasures are recommended as scour countermeasures for culverts:

1. Debris control devices
2. Channel protection at upstream and downstream, such as riprap or gabions
3. Energy dissipaters transition slab
4. Improved inlet and outlet design with headwall
5. Use of stilling basin or apron slab
6. Footings for any flared wing walls at the entry and the exit of culverts should be protected by riprap or alternate armoring countermeasures.
7. 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.
8. Regular visual monitoring should be carried out if riprap has been installed at the entry and exit of culverts.
9. When a culvert is located at the bend of a stream, clogging may occur. Avoid placing culverts at bend locations.

### **Optimization of Countermeasures**

There is a need for optimization of countermeasures from durability and cost considerations. For New Jersey, recent trends have been to focus on increased monitoring by inspections and instrumentation, use of armoring such as gabion mattresses and baskets, using sheeting left in place as permanent sheet piles and channel improvements. However, greater use of above and other types of armoring and structural countermeasures need to be considered.

## **CHAPTER 6**

### **SELECTION OF COUNTERMEASURES**

Selection of an appropriate countermeasure for a bridge footing depends on numerous factors, including river characteristics, local soil profile, type of scour, costs, environmental constraints and constructibility limitations at a site.

#### **Pre-Requisites to Selection of Countermeasures**

Selection and design of countermeasures should be based on hydraulic and scour analyses, 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, 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. Redundancy of the bridges (continuous versus multi-span simply supported).

#### **Factors Affecting Detailed Design of a Countermeasure**

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 the bridge
4. Availability of resources for monitoring frequency and underwater inspection
5. Priority of funding for repairs due to flood damage.
6. Constructability

#### **Studies Required Before Countermeasure Design**

1. Hydrologic Analysis – Refer to Flow Diagram in Appendix II.
2. Hydraulic Analysis - Refer to Flow Diagram in Appendix II.
3. Scour Analysis - The general design procedure for a scour analysis and countermeasure design requires information on bridge type, size, and location (TS&L) of substructure units. The scour analysis should classify the types of scour into the following categories:
  - a. Long term
  - b. Short term
    - 1). Contraction
    - 2). Local
  - c. Abutment Scour
  - d. Pier Scour

e. Wing wall Scour

A scour analysis for a bridge site should be based on procedure recommended by NJDOT for Stage 2 In-depth Scour Evaluation Study and HEC-18.

### **Countermeasure Installation Requirements**

A 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.

1. Permitting identification
2. Right of Way identification
3. Relocation of Utilities
4. Construction Coordination

### **Minimum Countermeasures Requirements**

If the projected (computed) 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 on bed rock or when a spread footing is located or placed below the total scour depth.
2. When an additional pile length equal to the projected scour depth is provided.
3. When pile stiffness exceeds the minimum required and the exposed length of pile due to erosion can safely act as a long column.
4. 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.

An adequate soil cover needs to be maintained for:

1. Frost resistance (minimum frost depth requirement)
2. As-built cosmetic appearance
3. Unforeseen error in the scour analysis data or computations

There are several countermeasures used for protection of bridge abutments by different state Department of Transportation 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.

### **Countermeasures Based on Types of Scour**

The effectiveness of a 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. Table 6.1 shows countermeasures based on types of scour.

Table 6.1 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

### Countermeasures in Ascending Order of Cost

An initial cost consideration for repair of scour critical bridges can be based on following scour countermeasures in ascending order of cost:

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.

### **Matrix of Countermeasures for NJ**

A countermeasure matrix facilitates preliminary selection of feasible alternatives, prior to a more detailed investigation. The matrix lists the countermeasure types in rows, against countermeasure characteristics in columns. Tables 6.2 shows modified countermeasure matrix that can be readily applied to NJ conditions. The table is based on the following information:

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 a scour analysis for each specific site. Not all of the listed matrix countermeasures are applicable for use in 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.

Table 6.2: Modified Bridge Scour & Stream Instability Countermeasures Matrix for New Jersey.

Countermeasures Group	Countermeasures Characteristics										Environmental Considerations			Bridge Rating Status	
	Functional Applications			Functional Applications		Suitable River Environment				Maintenance					
	Local	Scour	Contraction Scour	Stream Instability		River Type	River Size	Velocity	Bed Material	Estimated Allocation of Resources	Meets NJDEP Requirements			Structural Health of Bridge/ Bridge to be replaced/widened	
	Abutment	Pier	Flood-plain & Channel <sup>3</sup>	Vertical / Aggradation Degradation	Lateral Erosion / Meander	Braided <b>B</b> Meandering <b>M</b> Straight <b>S</b>  Suitable for All Cases <b>B M S</b>	Wide <b>W</b> Moderate <b>M</b> Small <b>S</b>  Suitable for All Cases <b>W M S</b>	Moderate Slow	Coarse Sand Fine  Suitable for All Cases <b>C S F</b>	High Moderate Low	Stream Encroachment	Effect On Marine Life / Habitat	Effect On Vegetation	Superstructure Condition	Remaining Life
GROUP 1. HYDRAULIC COUNTERMEASURES											Y Yes    Y Yes    Y Yes    G Good    L Long N No    N No    N No    P Poor    S Short				
GROUP 1A. RIVER TRAINING STRUCTURES															
TRANSVERSE STRUCTURES	o Unsuitable			O Well Suited		D Secondary Use									
Impermeable Spurs (Groins, Wing Dams)	D	D	O	O	O	B M	W M	M S	C S F	M L	Y o N O	Y o N O	Y o N O	P o G O	S o    L    O
Drop Structures (Check Dams, Grade Control)	D	D	D	O	O	B M S	W M S	M S	C S F	M	Y o N O	Y o N O	Y o N O	Y o N O	P o G O
LONGITUDINAL STRUCTURES															
Retards	D	O	O	O	O	B M S	W M S	M S	S F	H M	Y o N O	Y o N O	Y o N O	Y o N O	P o G O
Plantation	O	O	O	O	O	M S	W M S	M S	S F	L	Y o N O	Y o N O	Y o N O	Y o N O	P o G O
Bulkheads	O	O	O	O	O	B M S	W M S	M S	C S F	M	Y o N O	Y o N O	Y o N O	Y o N O	P o G O
Guide Banks	O	D	D	O	D	B M S	W M	M S	C S F	M L	Y o N O	Y o N O	Y o N O	Y o N O	P o G O
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	Y o N O	Y o N O	Y o N O	P o G O	S o    L    O
Rigid Grout Filled Mattress / Concrete Fabric Mat	O	D	D	D	O	B M S	W M S	M S	C S F	M	Y o N O	Y o N O	Y o N O	P o G O	S o    L    O
Flexible / articulating															
Riprap on Textile	O	D	D	D	O	B M S	W M S	M S	C S F	M	Y o N O	Y o N O	Y o N O	P o G O	S o    L    O
Riprap Fill Trench	D	O	O	O	O	B M S	W M S	M S	C S F	M	Y o N O	Y o N O	Y o N O	P o G O	S o    L    O
Gabion/Gabion Mattress	O	D	D	D	O	B M S	W M S	M S	S F	M	Y o N O	Y o N O	Y o N O	P o G O	S o    L    O
Articulated Concrete Blocks (Interlocking/ Cable Tied)	O	D	D	D	O	B M S	W M S	M S	C S F	M L	Y o N O	Y o N O	Y o N O	P o G O	S o    L    O
	LOCAL SCOUR ARMORING (X Not Applicable)														
Riprap (Fill/Apron)	O	D	X	X	X	B M S	W M S	M S	C S F	H M	Y o	Y o	Y o	P o	S o    L    O

											N o	N o	N o	G o	
Concrete Armor Units (Toskanes, Tetrapods)	D	D	X	X	X	BMS	WMS	MS	CSF	M L	Y o N o	Y o N o	Y o N o	P o G o	S o L o
Grout Filled Bags / Sand Cement Bags	O	D	X	X	X	BMS	WMS	MS	CSF	HM	Y o N o	Y o N o	Y o N o	P o G o	S o L o
Gabions	O	D	X	X	X	BMS	WMS	MS	CSF	M	Y o N o	Y o N o	Y o N o	P o G o	S o L o
Articulated/ Hollow Concrete Blocks	O	D	X	X	X	BMS	WMS	MS	SF	M L	Y o N o	Y o N o	Y o N o	P o G o	S o L o
Sheet Pile	D	D	X	X	X	BMS	WMS	MS	CSF	M L	Y o N o	Y o N o	Y o N o	P o G o	S o L o

GROUP 2 STRUCTURAL COUNTERMEASURES

	FOUNDATION STRENGTHENING														
Crutch Bents / Underpinning	O	O	O	O	D	BMS	WMS	MS	CSF	L	Y o N o	Y o N o	Y o N o	P o G o	S o L o
Pumped Concrete/ Grout Under Footing	O	O	D	D	D	BMS	WMS	MS	CSF	M	Y o N o	Y o N o	Y o N o	P o G o	S o L o
Lower Foundation/ Curtain Wall	O	O	O	O	O	BMS	WMS	MS	CSF	L	Y o N o	Y o N o	Y o N o	P o G o	S o L o
PIER GEOMETRY MODIFICATION															
Extended Footings	X	O	X	X	X	BMS	WMS	MS	CSF	L	Y o N o	Y o N o	Y o N o	P o G o	S o L o
Sacrificial Piles	X	O	X	X	X	BMS	WMS	MS	CSF	HM	Y o N o	Y o N o	Y o N o	P o G o	S o L o

GROUP 3 MONITORING

	FIXED INSTRUMENTATION														
Sonar Scour Monitor	D	O	O	O	D	BMS	WMS	MS	CSF	M	Y o N o	Y o N o	Y o N o	P o G o	S o L o
Magnetic Sliding Collar	O	O	O	O	D	BMS	WMS	MS	CSF	M	Y o N o	Y o N o	Y o N o	P o G o	S o L o
	PORTABLE INSTRUMENTATION														
Physical Probes	O	O	O	O	O	BMS	WMS	MS	CSF	L	Y o N o	Y o N o	Y o N o	P o G o	S o L o
Sonar Probes	O	O	O	O	O	BMS	WMS	MS	CSF	L	Y o N o	Y o N o	Y o N o	P o G o	S o L o
	VISUAL MONITORING														
Periodic Inspection	O	O	O	O	O	BMS	WMS	MS	CSF	H	Y o N o	Y o N o	Y o N o	P o G o	S o L o
Flood Watch	O	O	O	O	O	BMS	WMS	MS	CSF	H	Y o N o	Y o N o	Y o N o	P o G o	S o L o

NOTE: The Master Countermeasures Matrix given in HEC-23 (Table 1) Has Been Modified Based on Suitability to New Jersey River Scour Conditions.

## **Selection of a Countermeasure**

A selected countermeasure needs to satisfy cost, environmental and constructibility requirements in addition to the functional requirement at a site. The selection of a countermeasure may depend on several site dependent considerations:

1. For new bridges, effective measures such as optimum location of bridge, planning of deep foundations or adopting adequate sizes of openings should serve a primary role. Countermeasures (such as river training or structural countermeasures and armoring) may serve a secondary role.
2. For existing bridges, if scour depth and magnitude of erosion warrant the use of special countermeasures (such as river training or structural countermeasures), the accompanying use of armoring may serve a secondary role. If scour depth and magnitude of erosion are not high, armoring alone may be sufficient as a primary countermeasure.
3. If pre-cast concrete segments (ACB or cable tied blocks) are selected, they should be tested in a laboratory to determine if the use of any admixtures in the concrete mix may be harmful to marine life. The vendor may provide a certificate to this effect.
4. Since armoring countermeasures function as gravity forces on soil, use of heavy density aggregates should be specified. This will increase dead weight of precast blocks and decrease the volume of armoring by 15% to 25%.
5. Success rate of a similar countermeasure used in other States in similar types of flow conditions should be reviewed during selection of a countermeasure. Riprap may be used as a temporary measure at bridges. Average rankings of different scour countermeasures by 35 State DOTs are presented in NCHRP 24-7 [Parker et al (1998)].

In order to optimize the selection of countermeasure, steps in Figure 6.1 may be followed.

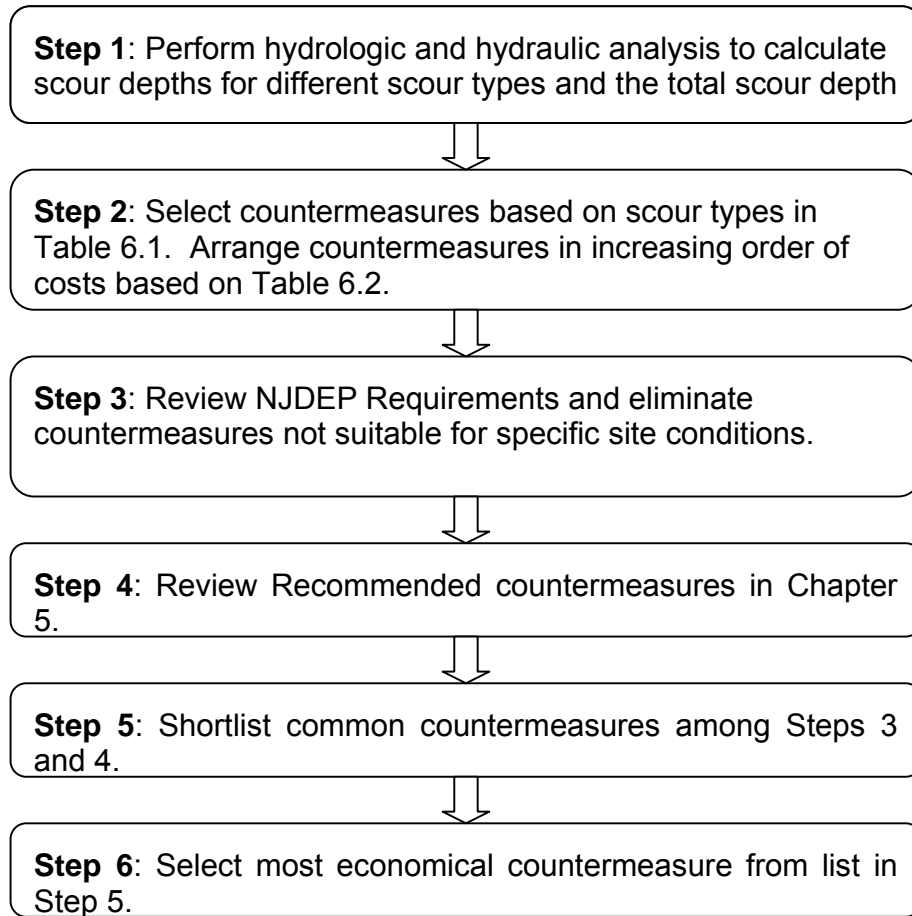


Figure 6.1: Selection of a Countermeasures for an Existing Bridge.

## **CHAPTER 7**

### **MONITORING AS PREVENTIVE COUNTERMEASURE**

#### **Introduction**

Monitoring of bridges for scour is one of the most cost-effective countermeasures. When the number of scour critical bridges becomes high, the cost of repairing each bridge would exceed the maintenance budgets available. Systematic monitoring helps in understanding scientifically, the erosion process, proper diagnosis of erosion problem and in selecting the type of countermeasure, such as harnessing the river or selecting the armoring of riverbed instead.

#### **Description of Monitoring Countermeasures**

For scour-critical bridges, monitoring can be used to identify potential bridge scour problems before they develop. Regular monitoring of all bridges is now mandatory by NBIS. The inspection cycle is every two years for bridges and four years for culverts. However, bridges classified as “structurally deficient” need to be inspected every year or more frequently, depending on the condition of the bridge. It is considered as a more economic countermeasure, since it prevents the use of expensive countermeasure by implementing regular maintenance and timely repairs.

Scour critical bridges typically require maintenance. Changes or deterioration in scour problem over time and through flood events can only be evaluated through an adaptive management program based on monitoring. A well-designed scour monitoring program will provide the following advantages:

1. It can be used as an effective tool for 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 continued long-term function of the countermeasure. Such maintenance is called “adaptive management” because it helps in identifying, over time, functionally effective countermeasures while minimizing impacts to the natural habitat.
4. Monitoring of 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.

#### **Visual Inspection**

Visual Scour 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 in addition to biennial inspections.

A normal 2-year inspection cycle with soundings (where required) for all scour-critical bridges should be performed. 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. The guideline on “Underwater Inspection and Evaluation of New Jersey Bridges” should be followed for underwater inspections.

Figure 7.1 represents the FHWA coding guide for scour monitoring of bridge foundations and actions required when scour depth is above, within or below the footing.

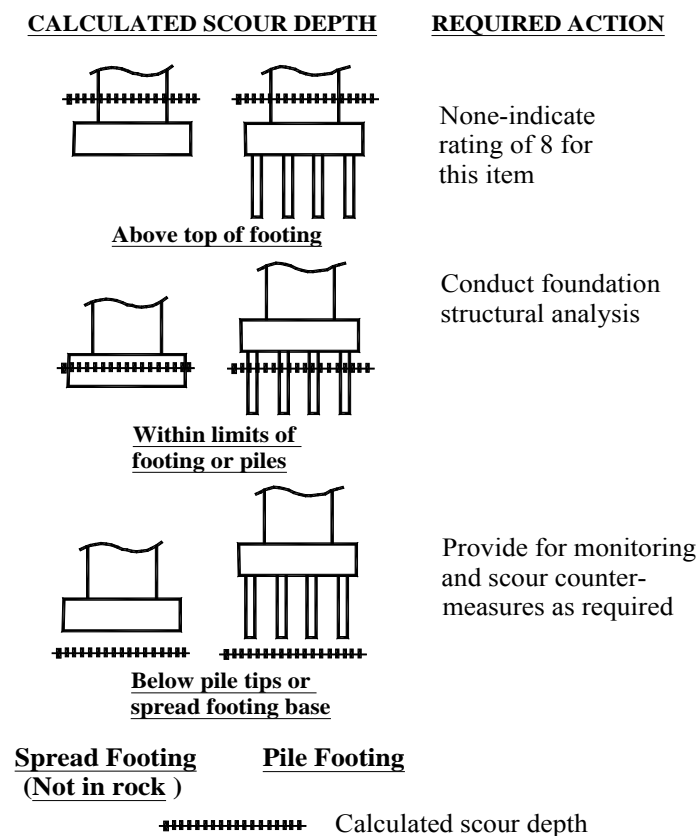


Figure 7.1: FHWA Coding Guide Item 113 used for Scour Rating.

### Underwater inspection

Underwater inspection may be necessary for bridges whose foundations cannot be visually inspected. It should be planned once every four years as a part of the visual inspection after a flood. The channel bed elevations measured through underwater inspection should be compared with historical data to identify changes due to scour. Figure 7.2 below shows the flow chart for visual and underwater inspection of bridges.



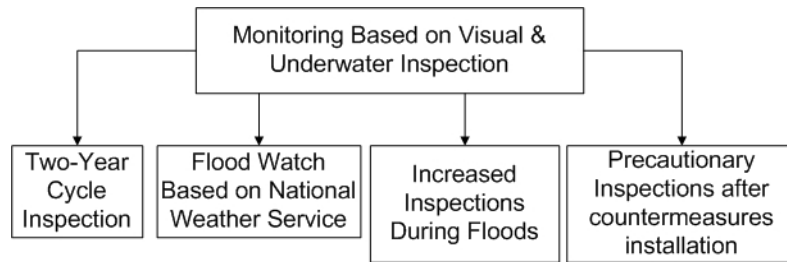


Figure 7.2: Flow Diagrams for Visual and Underwater Monitoring Scour.

### **Flood Watch List**

The flow diagram in Figure 7.3 shows various steps leading to preparing a flood watch list for monitoring high-risk (scour critical) bridges prior to floods.

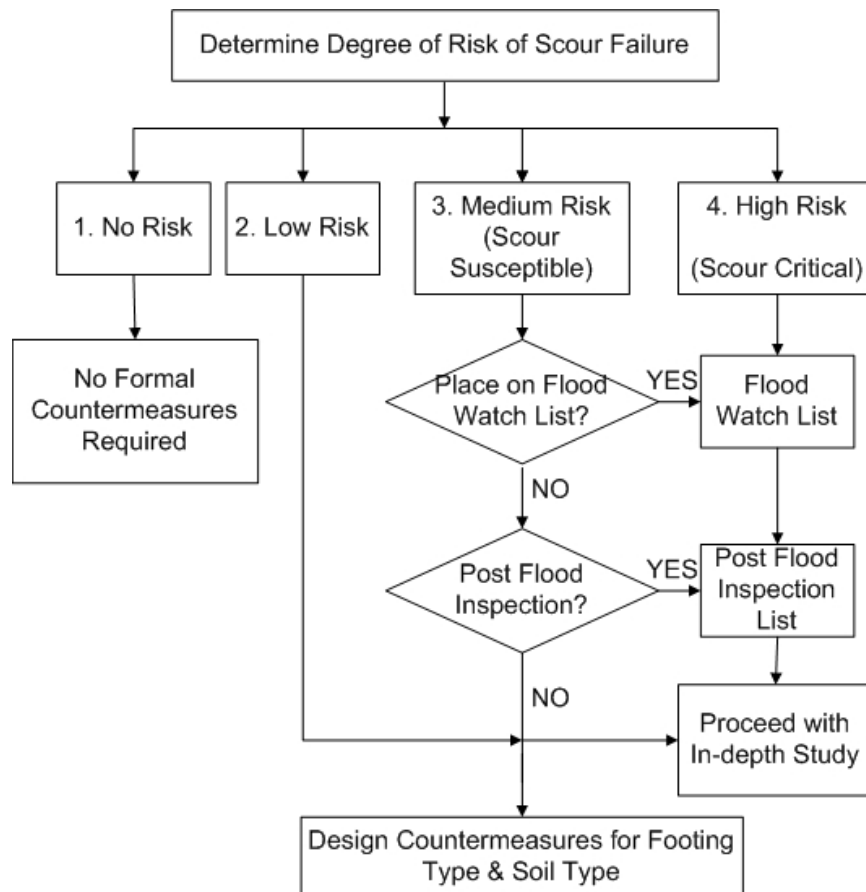


Figure 7.3: Flow Diagrams for Preparing Flood Watch List.

### **Simultaneous Monitoring of Twin Bridges**

When two twin bridges, such as Northbound and Southbound, are located close to each other on the same stream or river, both bridges are likely to suffer from similar scour problems. Hence, monitoring of the scour problem at one bridge should be

accompanied by a similar monitoring and rating of the other, irrespective of the bridge prioritized for inspection.

### **Frequency of Monitoring**

It is desirable to inspect scour critical bridges after every flood and after installation of a countermeasure. Frequency of monitoring is a site-specific issue and it may depend on following issues:

1. Erosion to bridge footings
2. Performance assessment of installed 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

### **Scour Monitoring Program**

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 include 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 should be re-established since bridge protection measures may rely 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, to identify any “obvious or apparent” channel response impact and to address any potential risk.
3. Upstream development should be monitored to ensure that bridge protection measures do not fail. Bridge scour problems should be managed through planned

and integrated approach. This will avoid or minimize the need for “reactive” bridge repair projects.

### **Types of Monitoring**

Monitoring can be accomplished using instrumentation or visual inspection. A well-designed instrumentation program for scour monitoring can be very cost-effective. Two types of instrumentation can be used to monitor bridge scour: fixed instrumentation and portable instrumentation.

#### **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 maximum scour depth. Data from sensors can be stored in a data logger on site and can be retrieved manually at fixed intervals or they can be transmitted to a remote computer system through a wireless network.

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 continuous monitoring over long intervals.

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 beds during peak floods. Any other newly developed portable instrumentation can also be used for scour monitoring provided that the reliability of the instrumentation is verified through visual monitoring or laboratory tests.

## **CHAPTER 8**

### **PLANNING OF NEW BRIDGES TO RESIST SCOUR**

#### **General Requirements**

A bridge designer has greater control over design parameters. The principles of scour reduction-planning need to be applied such that future investments in unnecessary countermeasures can be avoided. Planning to avoid overtopping should be considered.

Scour 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. To ensure the long-term safety of bridge foundations, supplementary countermeasures, such as riprap, gabions, etc., should be incorporated at the design stage as an integral part of a new structure and may be used as a second line of defense against scour. The designer has more options, for minimizing scour, in the planning stage than for an existing bridge. These options include:

1. Selecting suitable locations on the stream away from confluence, bends and dams.
2. Making the opening wide enough, to minimize contraction scour.
3. Providing deep foundations.
4. Providing river training measures or structural countermeasures at the preferred time of bridge construction, thereby minimizing costs.

For example, it may be difficult to drive sheeting around a pier or abutment for an existing bridge. The sheeting may initially serve as a cofferdam during construction in place of stream diversion. It may then be left in place as permanent sheeting. Planning of a new bridge should include the following five steps:

#### **Economic Planning**

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 to select the most appropriate foundation type.
2. Deciding, if justifiable, on a design return period other than 100 years (such as 50 years) and check return period of 500 years (super flood). In case of embankments with levees, a 50-year flood may govern over a 100-year flood for peak flood velocity and water surface elevation.
3. Scour risk assessment based on hydraulic and hydrologic analysis.
4. Right of way and construction issues related to proposed scour countermeasures.

### **Hydraulic planning**

The goal of hydraulic 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 a 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 countermeasure design for new bridges. The following factors should be considered in the structural planning of new bridges.

1. Locating bridges to avoid adverse flood flow patterns
2. Streamlining bridge elements to minimize obstructions to flow.
3. Designing foundations to resist scour without relying on the use of riprap or other countermeasures.
4. Designing deep foundations or foundations on rock, where practicable;
5. 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. If countermeasures are applied to structures after they are constructed, their use 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 very cost-effective. Although new bridges should be designed to be safe against scour, a well planned monitoring program incorporated during planning/construction phase may be valuable in continued safety of the bridge and in gaining advanced knowledge on scour countermeasures. Important factors to be considered in monitoring 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. 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.

### **Scour Reduction Planning**

Several important factors should be considered during the hydraulic design phase of the bridge to minimize the risk of scour.

#### **Location related**

1. Avoid locating structures at a confluence of two or more channels.
2. Avoid locating structures at or near sharp bends; locations on straight reaches or gentle bends are preferred.
3. Locate bridge crossings at the head or apex of the alluvial fan.
4. Check channel stability using aerial and satellite photography, historic maps, changes in flow direction can increase scour significantly.
5. Consider the stability of the channel vertically and laterally.
6. Physical modeling should be considered for major structures on alluvial rivers or channels, major tidal crossings, barrages and complex flow conditions that cannot be modeled using simple 1-D models.
7. Consider river morphology: channel widening, realignment, and changes in agricultural practices that may reduce scour
8. Analyze the effect of existing nearby structures or effects of removal of old structures on flow conditions around the bridge

#### **Hydraulic design related**

1. Size of waterway opening: consider construction cost versus scour risk
2. Size and number of relief openings: This measure is used for rivers with large waterway widths. 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.
3. Consider scour due to changes in flow direction at low flows and flood flows.

4. Consider effect of operational requirements causing concentration of flow on one side of structure (e.g., opening of weir gate on one side)
5. Overtopping of approach embankments: Lower level of approach embankment than bridge deck will avoid washout of the bridge.
6. Consider effects of removal of downstream or upstream bed control
7. Channel improvements

### **Streamlining structural elements**

1. Abutments: Sloping (“spill-through”) abutments cause significantly less scour than a vertical wall abutment.
2. Integral abutments with a single row of piles and semi-integral abutments, which are similar to stub abutments on piles, have a lower scour depth as calculated from standard scour analysis equations.
3. 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. Wing walls at 90° to the longitudinal flow direction are also acceptable if turbulence due to separation of flow is unlikely to be a significant problem.
4. Pier groups: For bridges on scour critical rivers, the number of piers selected shall be as small as possible. This can be achieved by selecting longer spans. Minimum obstruction from the pier group during a peak flood event will prevent water overtopping the deck.
5. Pier shapes: The best hydraulic performance is given by rectangular piers having a wedged-shaped nose (known as “cutwaters”).
6. 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 the high water elevation may be more appropriate.
7. Where debris accumulation is likely to be a problem, debris deflectors should be used.
8. Overall structure alignment and alignment of elements: Where the bridge deck could become submerged by an extreme flood, it may be appropriate to streamline the underside of the bridge deck by rounding the upstream and downstream faces to facilitate passage of debris.

### **Foundations for New Bridges**

For the design of new bridges, bridge foundations 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 foundations should be designed to withstand scour during floods equal to or less than the 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 superflood conditions.

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 cases of pile foundations, 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 (See Table 3.4).

#### **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.

### **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 the Tables 8.1 and 8.2 in this chapter. 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. Elevation of the lower chord of the bridge should be increased a minimum of 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 requires designing pressure flow through the bridge waterway.

## **Substructure**

### **Piers (Substructures)**

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 may range 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 HEC-18 may tend to over-estimate the scour depth because of lack of verification of field conditions. Recognizing this, the

abutment scour equations may be used to develop insight as to the scour potential at an abutment. Engineering judgment must then 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 with the use of this alternative.
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 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. When warranted, riprap, 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 supplementary countermeasure for the protection of new foundations (piers/abutment). Riprap should be used with prior excavation and with filter layer and only as a supplemental countermeasure.

Table 8.1: 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	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 bridge cost	Bed armoring or sheet piles. Monitoring
Abutment Type	Use stub or Integral in place of full height.	Local scour is minimum. $K_2 = 0.5$	None	Use sheet piles, Monitoring
Vertical 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 flow direction	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. $K_1$ is lower.	Increased Formwork cost.	Monitoring
Pier Foundation	Use deep piles in place of short piles	Additional pile lengths compensate loss of soil due to scour. Minimum degradation	Cost of piles increases	Use sheet piles with riprap to protect scour of top of piles, Monitoring
Spread Footing on Soil	Place bottom of footing below total scour line.	Footing settlement is avoided	None	Bed armoring such as riprap or gabion required, Monitoring
Spread Footing on Weathered Rock	Determine RQD to estimate rock erodibility	Allowance is made for scour of rock	None	Bed armoring such as riprap or gabion required, Monitoring
Spread Footing on Rock	Bottom of footing on rock	Footing settlement is avoided	None	Monitoring
Bearings	Place bearings above M.W.L.	Reduced rust or damage to bearings.	Increased bridge height	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 8.2: 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	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	Increase of span length	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
Open Parapets and Railing	Avoid fence or use of solid parapet	Increases flow during peak flood. Contraction scour is minimum	None	None
Deck Profile	Use of 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

### **Specific Guidelines to Minimize Different Types of Scour of New Bridges**

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 abutments
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. Provide continual Maintenance Planning

#### **Degradation**

1. Use check dams or drop structures on small to medium streams
2. Use channel lining

3. Use deeper foundations
4. Provide adequate setback of abutments
5. Use rock and wire mattress for small channels

River meander

1. Locate bridges on straight reaches of streams between bends.

Braided channels

1. Build one long bridge, more than one bridge or a relief bridge

### **Geotechnical Considerations**

In addition to structural planning, if warranted, supplementary 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 for existing bridges to a particular soil type.

### **Requirements of Foundation Design**

#### **Spread Footings on Soil**

Place bottom of footings 3 feet below the total scour line. Although 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 supplementary 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%.

For anchorage of concrete footing inside rock, use minimum 3" depth into the underlying bedrock. Any recommendations given in geotechnical report on rock quality shall be followed regarding limitations in cutting into bedrock layers.

### **Spread Footings on Non-erodible Rock**

This condition is less common in New Jersey since non-erodible rock is not generally found within 10 feet of river beds.

### **Overtopping of Flood Water during Floods**

Occasionally, flood water may overtop a bridge deck. This may occur due to unforeseen change in demography or unusual snow melt or storms resulting in peak floods, the quantum of which was not allocated in the original hydrology analysis. This factor is generally beyond the control of a structural engineer since a large span design or river training measures, in anticipation of unknown factors, is not economical. However, the following factors can be incorporated in the planning stage:

1. Effective drainage provision: Longitudinal slope of profile of base line geometry should be 2% or more for rapid disposal of water to the approaches and beyond. In the transverse direction approximately 4% cross slope needs to be provided. Drainage inlets and pipes need to be provided and connected directly to public sewerage system, rather than dispose off the water back in the river.
2. A sag curve on the deck should be avoided to prevent water accumulation.
3. Solid parapets, which trap water, should be avoided. Instead parapets with open rails should be used.
4. Bridge deck profile should be raised by a few feet, if possible. This may not be always possible since access to adjoining properties will be affected.
5. Continuity of deck: Twin bridges, each catering for traffic flow in opposite directions, should be planned instead of a single bridge. The adverse scour or overtopping effects of one bridge will be less for the other bridge, due to cushioning of water overflow by the first bridge.



## CHAPTER 9

### CONSTRUCTIBILITY ISSUES

#### General Constructability Issues

Before starting a design, it is important to know soil conditions and construction factors, which are likely to affect the selection of countermeasures. The following constructability issues should be evaluated:

**Duration of construction:** The available flow width may be reduced due to construction of cofferdams and embankment. This may lead to increased scour in the channel and around the structure because of increased flow velocity. Hence, construction of the above items shall be done during off-flood season.

1. **Maintenance and Protection of Traffic:** During installation, 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.
2. **Underwater work:** Health and safety of construction personnel may be of concern if depth of water is high. Trained divers may be required.
3. **Access to site:** Temporary road for transportation of materials and equipment adjacent to the channel bank may be difficult to construct. Wooden mats should be used when lane width is restricted.
4. **Temporary Works:** Temporary construction works may be required. More economical alternatives implementing quick construction and safety needs to be carefully evaluated.
5. **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.
6. **Environmental risks:** Pollution of river from construction material may occur. Channel needs to be cleaned. Approvals for stream encroachment permits would be necessary.
7. **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 companies would be required.
8. **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.
9. **Specialized work:** Modern countermeasures require new construction techniques. The contractor performing such tasks needs to train his construction crew for such techniques.

10. **Availability of labor and plant:** Some types 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 distances may be expensive.
11. **Limited vertical clearance under the bridge:** It may be difficult to construct a cofferdam or drive sheeting under a bridge if restricted vertical clearance exists. Placement of countermeasures will also be difficult.

### **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. An emergency installation is typically much more costly.
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, trees and vegetations may be providing protection or may eventually protect a 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.

### **Safety Considerations**

1. Working adjacent to fast, unpredictable currents, rapidly rising water levels can be extremely dangerous.
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.

### **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')

### **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".
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.

### **Cofferdams**

If the water depth is not high, temporary cofferdams may be required for construction in dry conditions. Without dry conditions the quality of placement of countermeasures will be difficult to monitor or maintain. Figure 9.1 shows an elevation view of a typical Cofferdam.

#### **Cofferdam Construction Notes**

1. Sheet piles may be warranted for cofferdams. The guidance in the NJDOT Bridge and Structures Design Manual should be followed.
2. Sheet piles may be braced with perimeter walls and knee braces as required for improving their stability.
3. Excavate to the design depth of the riprap layer or gabion mat.
4. Install riprap/alternate countermeasures within the cofferdam.
5. Backfill to original bank/river bed.
6. Remove bracing system, complete backfill and if warranted remove sheet piles.
7. If water depth is less than 3 feet, sandbags may be used in place of sheet piles.
8. The guidance provided in the NJDOT Bridge and Structures Design Manual should be followed for provision of cofferdam.

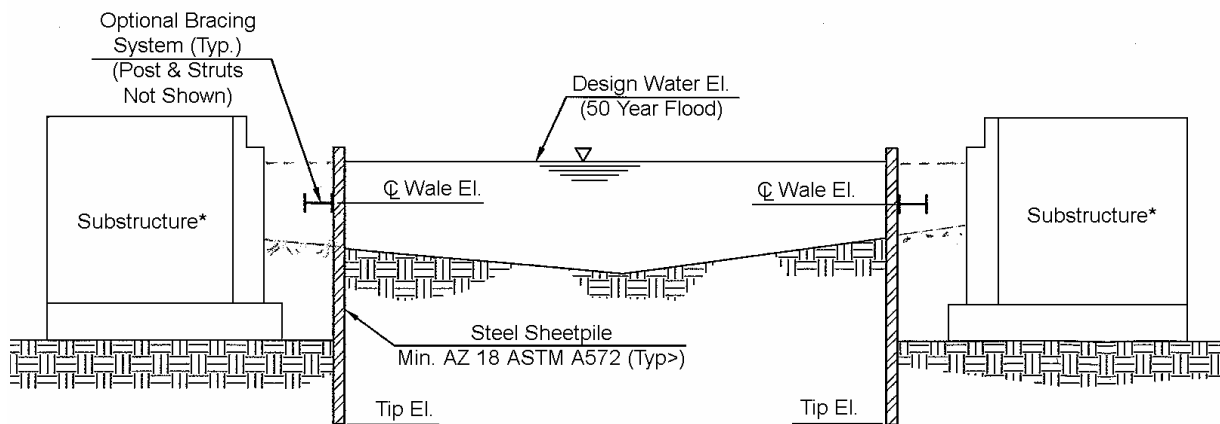


Figure 9.1: Cofferdam Elevation.

### Sheet Piling Left in Position

For underwater construction of the following abutments and pier types, temporary sheeting on the stream side is required for installing countermeasures at the sides of spread footings/pile caps:

#### 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 the completion of countermeasures installation.

### Traffic and Utilities Issues

1. Site access: Adequate access to the site shall be provided for trucks to deliver countermeasures material.
2. Right of Way: Construction easements and right of way may have to be purchased for the duration of construction.

3. Possible detours: Detour, lane closure or night time work may be necessary. Coordination with Traffic Control would be required.
4. Emergency vehicles and school bus services shall not be affected by lane closures.
5. Utilities: Relocation of any utilities at the sides of abutments or piers may be necessary for the duration of construction.
6. Coordination with utility companies would be required.
7. Four weeks before the start of construction traffic police needs to be informed of shut down or detour of one or more lanes.
8. Warning signs showing dates and times of shutdown are required to be posted well in advance for the information of users.
9. Construction time be kept to a minimum or performed at night time.

### **Case Study of Countermeasures Construction**

Appendix III shows construction drawings for a number of projects in NJ, where gabions were used as countermeasures. The gabions are over laid by a soil layer, where vegetation is grown for resisting erosion during floods. The format of the drawings may be used for other types of countermeasures.

Details show precautionary measures (see construction notes) to keep the river water clean, such as the use of turbidity dam, silt fence and traffic control during construction to comply with NJDEP underwater construction permit requirements with minimum impact on plant and marine life.

## **CHAPTER 10**

### **GEOTEXTILE AND GRANULAR FILTERS**

#### **General Use of Filters as Armoring**

Filters are a secondary countermeasure but when combined with a primary countermeasure such as rock riprap, gabions or artificial riprap, they act as an additional armoring by making the armoring system more effective.

The use of a filter is to ensure that underlying fine sediment particles do not leach through the voids of the individual stones in riprap and other countermeasures. Filter prevents migration of subsoil particles through the countermeasure. 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 beneficial.

In the case of interconnected gabions on a sand bed river, a geotextile filter should be placed underneath the baskets 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.

Commonly used filters are granular, geotextile and composite types. The three main references for filters are

- 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.
- NJDOT Soil Erosion and Sediment Control Standards.
- CIRIA Manual 2002 and CUR/RWS Report 169 (Center for Civil Engineering Research & Codes, 1995, UK).

In addition, manufacturer's technical specifications for geotextiles need to be referred. The design parameters are empirical and based on test results for specific conditions. New products for geotextiles, for example, may recommend alternate solutions.

#### **Testing Program For Geotextile Quality**

Geotextile materials that permit innovative approaches of filter placement for riprap and other countermeasures have been developed through an appropriate testing programs. These testing programs consists of:

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.

### **Limitations of Filters**

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  $D_{50} > 1/16^{\text{th}}$  inch but suitable for sand bed stream for which  $1/400^{\text{th}}$  inch  $< D_{50} < 1/16^{\text{th}}$  inch.

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 required.

### **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
    - i. Relatively low cost
    - ii. Large areas can be laid quickly
    - iii. Small construction thickness. Volume of riprap is therefore reduced considerably compared to no filter armoring.
    - iv. Generally provides a broad band of failure threshold.
    - v. Non-woven types can cope with soil variations and allow for settlement.
  - b. Disadvantages
    - i. Long-term behavior is less certain than granular filters
    - ii. Difficult to lay in deep water
    - iii. Difficult to place in high currents
    - iv. Must be pressed evenly against the subgrade by the armor layer
    - v. Damage can be difficult to repair
    - vi. Difficult to identify exact location of geotextile failure.
    - vii. 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
    - viii. Openings can become blocked
    - ix. It cannot easily be 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.

c. Woven type of geotextile

- i. They are formed using regularly placed fibers orientated at right angles to give uniform hole sizes.
- ii. Woven geotextiles are generally stronger than non-woven and can be used as filters for soils of a particular size.
- iii. They can also be appropriate where very high porosities are required.

d. Non-woven type of geotextile

- i. Non-woven geotextiles are formed using randomly placed fibers, giving a range of hole sizes.
- ii. 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.
- iii. 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: Granular filter is placed in layers and requires excavation.

a. Advantages

- i. Deforms, so good contact is maintained between subsoil and armor layer
- ii. Repairs are relatively easy and damage is sometimes self-healing
- iii. Durable

b. Disadvantages

- i. Excavation may be required to lay it
- ii. Accurate placing difficult in deep water
- iii. Difficult to place in high currents
- iv. Careful placing needed to achieve required thickness
- v. Grading needs to be carefully controlled
- vi. Multiple layers may be needed to meet filter requirements
- vii. Required grading may be difficult to obtain locally

3. Composite Filter: Composite filter is composed of sacks or mattresses and is useful for deep water or high current conditions.

a. Advantages:

- i. 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 geotextiles.



- ii. 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.

### **Cost**

1. The cost of material and installation for geotextile is two to three times for providing granular filter.
2. An average cost for providing a geotextile layer needs to be assessed. The cost may vary for each county and water depth.

### **Vendor Details**

Details from manufacturers must be obtained. An approved equivalent product may also be used.

### **Monitoring and Inspection**

Geotextile requires monitoring more than the granular filter. 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.

### **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. Detailed design guidelines for geotextiles are available in the following publication:

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.

1. **Functions:** The following functions should be considered in the design of Geotextiles:
  - a. Soil retention: It is related to the size of pores or holes (characteristic opening size) in geotextile.
  - b. Permeability
  - c. Strength
2. **Design Considerations**
  - a. Anchorage: The area of the filter should be sufficient to allow for the anchorage 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 protective granular separation layer is required between the geotextile and the 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 following specifications for Geotextiles as suggested by The Center for Civil Engineering Research & Codes 1995, CUR/RWS Report No. 169 are presented as guidelines. Reference should be made to manufacturer's literature for the relevant ratios based on geotextile testing:
  - i. For Geo-textiles laid against non-cohesive, uniform soils
$$O_{95}/D_{85}(\text{base}) < 1$$
, where  $O_{95}$  is opening size where 95% of pores are smaller and  $D_{85}$  is the diameter of rip rap and filter material of which 85% are finer by weight.
  - ii. For Geotextiles laid against cohesive soils
$$O_{90}/D_{10}(\text{base}) < 1.5 D_{60}(\text{base})/D_{10}(\text{base})$$
$$O_{90}/D_{50}(\text{base}) < 1$$
$$O_{90} < 0.5 \text{ mm}$$
Where  $O_{90}$  is opening size where 90% of pores are smaller;  $D_{10}$ ,  $D_{50}$ , and  $D_{60}$  are the diameters of filter material of which 10, 50 and 60% are finer by weight. The base material may be used as the filter if it meets the above criteria.
  - iii. 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.
  - iv. 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.

## **Design Procedures for Granular Filters**

### **1. Design considerations:**

- a. Granular filters are normally made of sand, gravel or stones. 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 4 and 6 inches or  $D_{100}$  or  $1.5 D_{50}$ , where one layer is used.
- c. Normally a thickness  $> 8$  inches is required.

- d. The following criteria are recommended, based on CIRIA (2002) and CUR (1991) and CUR/RWS Report 169 (Center for Civil Engineering Research & Codes, 1995), Refer CIRIA C55 Manual, Section 5.6.2.

- i. For uniformly graded material:

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

- ii. For well-graded material:

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

Also, as per NJDOT Soil Erosion and Sediment Control Standards,

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

- iii. 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.  
 iv. 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.  
 v. For gap-graded base material: As per CIRIA C55 Manual, Section 5.6.2

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

As per NJDOT Soil Erosion and Sediment Control Standards, for all types of material to ensure adequate permeability:

$$D_{15} \text{ (Coarser Layer)} / D_{15} \text{ (finer Layer)} < 40.$$

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:  $D_{15} \text{ (Coarser Layer)} / D_{15} \text{ (finer Layer)} < 40.$

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

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

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.

- e. The thickness of each filter layer should be greater than 4 inches and should be at least 6 inches where only one layer is required. Normally, a thickness of at least 8 inches to 10 inches 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. When placed underwater or in high currents, the layer thickness should be increased by about 50 per cent.

## **CHAPTER 11**

### **DESIGN GUIDELINES FOR ARMORING COUNTERMEASURES**

In this chapter, design guidelines for the following armoring countermeasures are presented:

- Gabions
- Articulated Concrete Blocks
- Concrete Armor Units
- Grout Bags

#### **Gabions**

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

A thinner version of gabions 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 11.1 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 may be unstable
  - c. On upstream or downstream of bridges and culverts, as river training energy dissipaters.

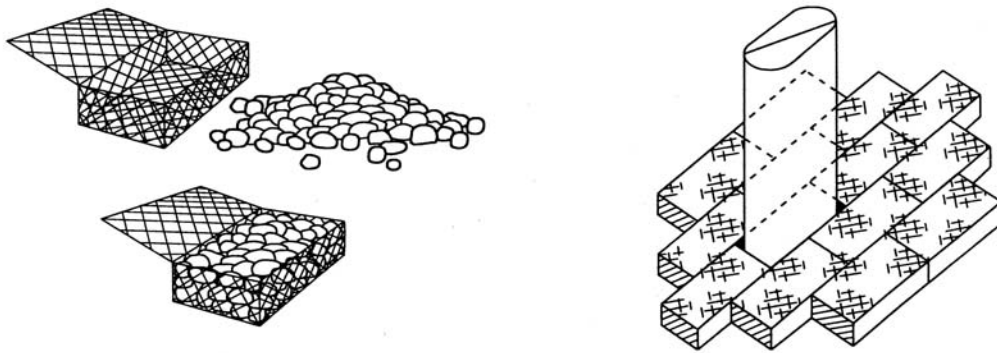


Figure 11.1: Gabions and Reno Mattresses for Bridge Scour Countermeasure.

### **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 lesser 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 is used for slope protection at riverbanks and as guide banks. Riprap stone 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 that are placed at the toe of a 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 11.2 shows an example of wire-enclosed riprap installed in embankments of the Burlington County Bridge.



(a)



(b)

Figure 11.2: Use of Wire enclosed Riprap at Burlington County Bridge No. 128a.

### **Possible Failures 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 factors, 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 above factor.

### **General Guidelines for Gabions**

Following general guidelines should be considered during design and installation of gabions.

1. **Use of Filter Layers with Gabions:** On sand river beds, 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 10.

2. **Design Tools:** Following tools may be used for the design of gabions.
  - a. Approved commercial software may be used.
  - b. Construction drawings must be based on detailed designs.
  - c. A design spreadsheet may be used if approved by NJDOT.
3. **Cost:** An estimated current cost of \$ 85 to \$ 95 per SF of pier area may be used. Actual costs may vary, depending on local conditions.
4. **Construction Permits:** Stream encroachment and other applicable permits will be required. Refer to Guidelines given for permit application in NJDEP Stream Encroachment Technical Manual.
5. **Durability and Maintenance:** The following types of failures may occur and may be avoided by good construction practice:
  - a. Failure of meshes and stones fallout due to corrosion, abrasion and damage during construction.
  - b. Winnowing failure due to erosion of underlying bed material through the gabions due to failure of filter layers and inadequate gabion thickness during floods.
  - c. Excessive movement of stone within the baskets may occur at high currents due to poor packing.

### **Design Guidelines for Gabions**

A detailed procedure for sizing and layout of gabions, and its application to an example bridge in New Jersey is presented in this section.

Design guidelines for gabions are based on laboratory experiments conducted by Parker et al (1998) under the NCHRP 24-07 project, CIRIA (2002) and HEC-23. Other recommended sources of information on design of gabions are U.S. Army Corps of Engineers (1991), Maynard (1995) and Racine (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. **Manning's "n" value:** The "n" value used for gabions shall be 0.025.
3. **Sizing of Gabions:** By enclosing the stones within the wire mesh, smaller size stones than those used for conventional riprap can be used. 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. The following procedures that are based on CIRIA (2002) and Parker et al (1998) should be used in the absence of any design manual provided by a manufacturer.

The thickness of gabions should be determined on the basis of critical velocity of flow as indicated in Table 11.1 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 11.1 by considering the design velocity as the critical velocity so that there is an extra factor of safety up to the limiting velocity when gabions fail.

Table 11.1 provides lower and upper bounds on critical (design) velocities, stone size and limiting velocity for a gabion thickness. There are two values of stones sizes, critical velocities and limiting velocities for each range of gabions thickness. Thickness and stone size for gabions should be selected for the lower value of the critical (design) velocity for a conservative design. For example, for a critical velocity of 13 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.

Table 11.1: 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 (2002)

In sizing gabions using Table 11.1, the 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.

Gabions can also be sized according to NJDOT Soil Erosion & Sediments Control Standards using Table 11.2 below for maximum velocity up to 14 ft/sec,

Table 11.2: NJDOT Soil Erosion & Sediments Control Standards for Gabions

Gabion Thickness (Ft.)	Maximum Velocity (ft/sec.)
½	6
¾	11
1	14

Stone size for gabions can also be calculated by the Pilarczyk Equation [CIRIA (2002)]

$$d_{n50} = \frac{\mu}{(1-p)(s-1)} \frac{0.035 K_T K_Y U^2}{\Psi_{CR} K_s 2g} \quad (11-1)$$

where,

$\Psi_{CR}$  = Stability Factor = 0.07 for gabions

$p$  = Porosity of stone filling the gabions < 0.4

$\mu$  (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) =  $(k_d k_l)$  where

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

$$k_l = \sin (\varphi - \chi) / \sin \varphi$$

$\varepsilon$  = Angle of bank to the horizontal

$\varphi$  = Internal angle of friction of the revetment

$\chi$  = Angle of the channel invert to the horizontal

Equation (11.1) needs to be calculated iteratively using the following procedure:

Step 1: Using stone size ( $d_{n50}$ ) based on Table 11.1, 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 Table 11.1 and Equation (11-1).

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

$$V = \frac{0.069U^6K^6}{\rho_r^3(\rho - 1)^3g^3} \quad (11-2)$$

where,

$V$  = Allowable volume of individual unconnected basket in  $\text{ft}^3$   
 $U$  = Approach design flow velocity in  $\text{ft/sec}$   
 $K$  = 1.5 for round nosed piers and 1.7 for square nosed piers  
 $\rho_r$  = Density of rock in the basket or box  
 $\rho$  = Density of water  
 $g$  =  $32.2 \text{ ft/s}^2$

Selected volume of gabion basket should be greater than the volume obtained from Equation (11-2).

#### Special Sizing Notes:

- When sizing gabions, both lower and upper bounds on stone size should be established using Table 11.1, Equation (11-1) and NJDOT Soil Erosion and Sediment Control Standards in Table 11.2.
- All gabions or Reno mattresses placed against bottom of a channel should be underlain by filter fabric or gravel filter designed according to guidelines in Chapter 10.
- The smallest stone size to be placed in a wire net should not be less than 4 inches or greater than 12 inches. Actual size may be based on manufacturer's recommendations.
- 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 11.3 and 11.4). The extent of filter shall be smaller than the extent of gabions as shown in Figure 11.4.

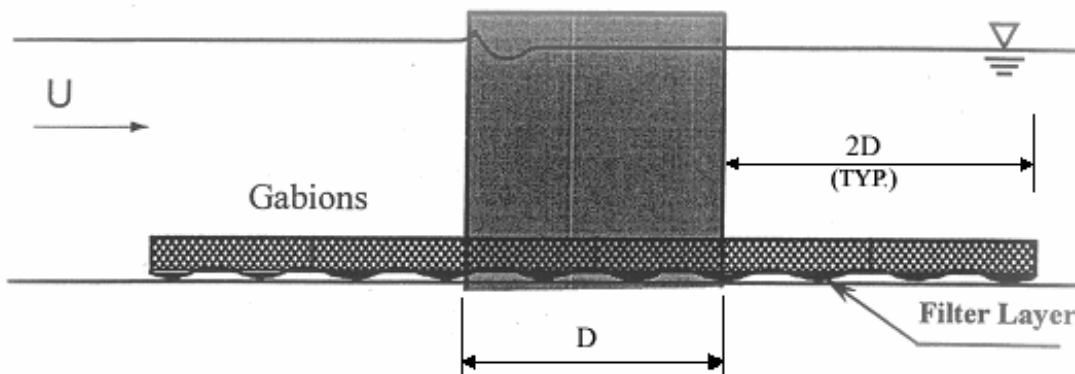


Figure 11.3: Gabion Installation of Bridge Piers.

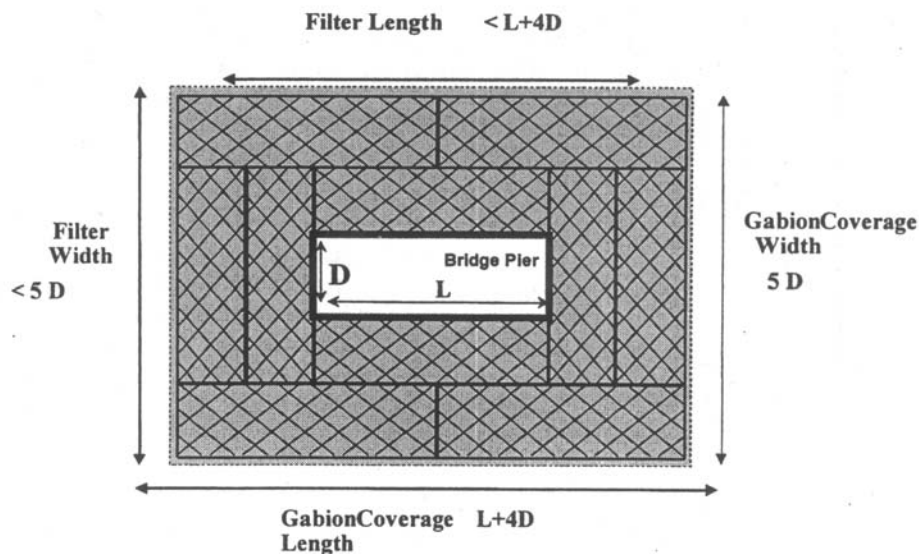


Figure 11.4: Extent of Gabions and Geotextile Filter around Bridge Piers.

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 "Design Guidelines for Riprap at Bridge Abutments" Section of Chapter 14. For spill-through abutments, gabions should be installed in the same configuration shown in Figure 14.7 in Chapter 14 for riprap, i.e., the width of riprap from the toe of the abutment should at least be two times the flow depth.

Figures 11.5 and 11.6 show typical layouts of gabions for embankments and spill-through bridge abutments. For small openings, the gabion nets should extend the full width between the two abutments.

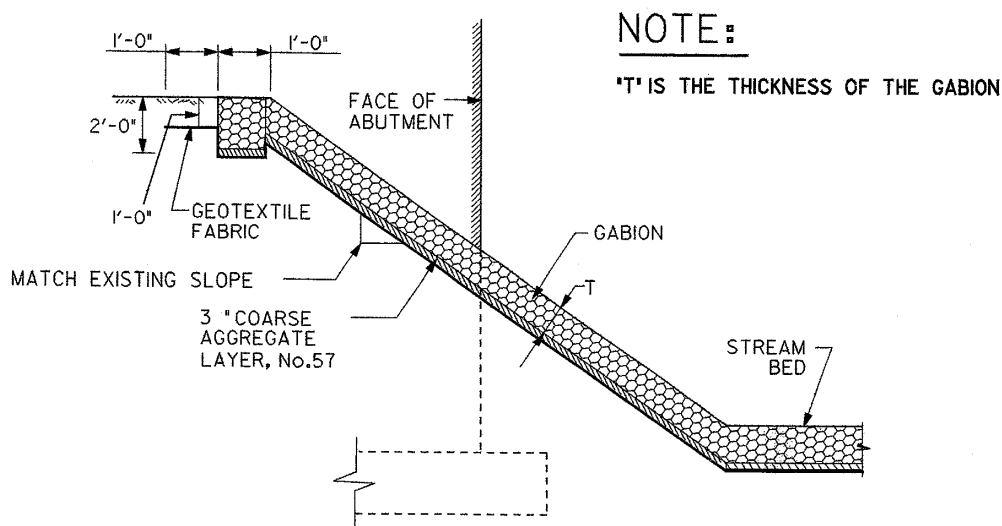


Figure 11.5: Gabion Detail at Embankment

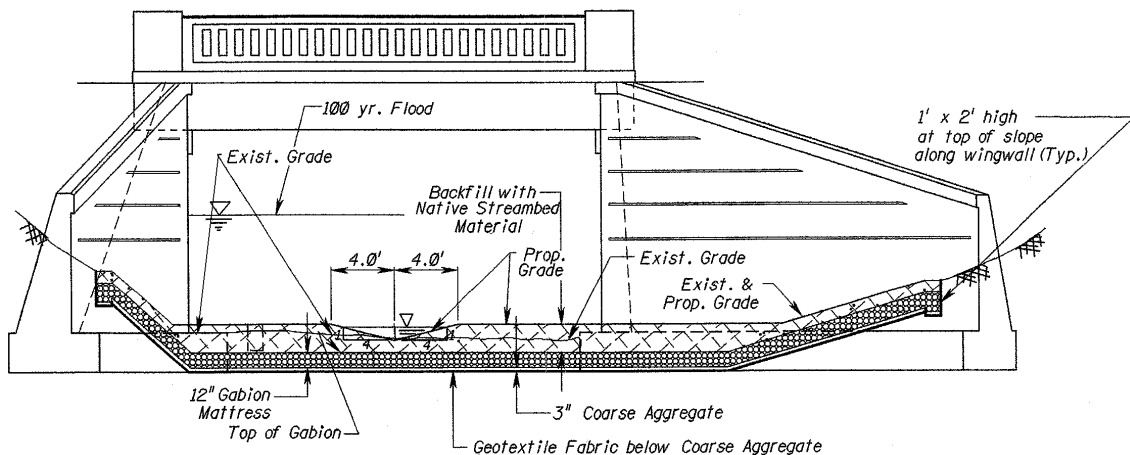


Figure 11.6: Gabion Detail at Embankment: Upstream Elevation, Looking Downstream.

6. **Scale Factors:** Scale factors for depth 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:

a. Material

1. 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.
2. Polymer mesh – woven type.

b. Coating

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

c. Mesh Details

1. The Mesh Pattern can be hexagonal, rectangular or V-shaped (See Figure 11.7)
2. 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 1/8<sup>th</sup> inch.

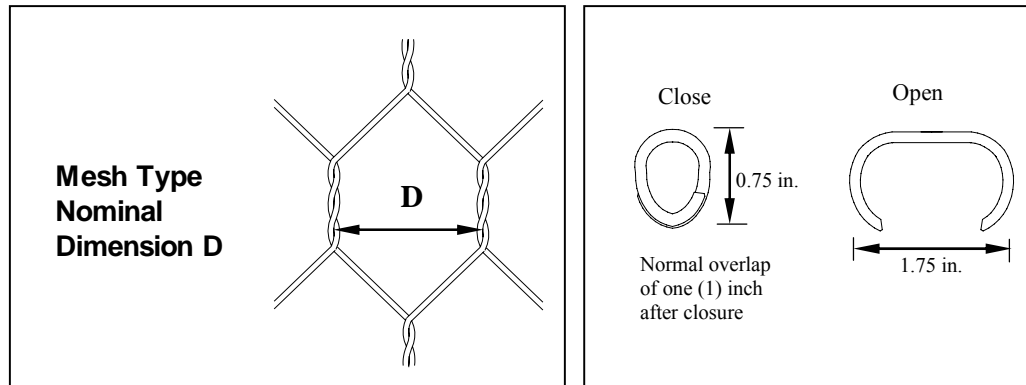


Figure 11.7: Mesh patterns of gabions.

### Design Example for Gabions

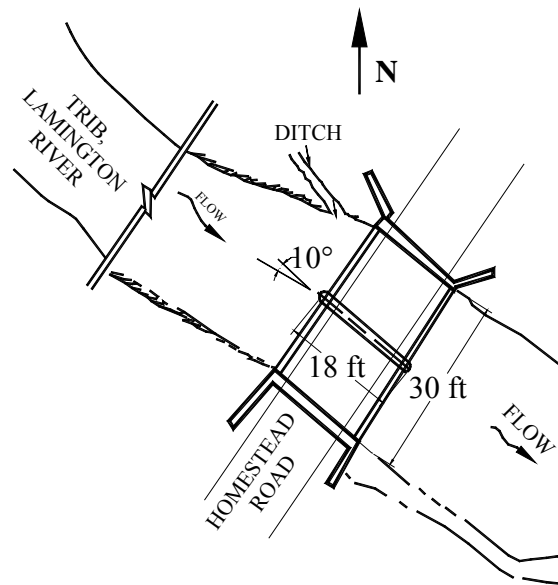
A typical New Jersey bridge has a single round nosed pier and vertical wall abutments. Layout of the bridge site is shown in Figure 11.8. Elevations of pier and abutment are shown in Figure 11.9. Hydraulic, foundation and scour data for the bridge are as follows:

#### Pier

Design scour depth at pier	= 5 ft
Width of pier footing	= 7'-6"
Length of Pier (Width of Bridge)	= 18 ft
Width of pier	= 3'-6"
V100	= 5.08 ft/sec
V500	= 7.03 ft/sec
Specific Gravity of stone, $S_s$	= 2.24

#### Abutment

Flow Depth, $y_0$	= 6.12 ft
V100 at abutment	= 5.54 ft/sec
V500 at abutment	= 7.67 ft/sec
Specific Gravity of Stone, $S_s$	= 2.24
Width of abutment footing	= 10 ft
Width of abutment	= 8 ft
Design scour depth at abutment	= 5'-6"



PLAN (NOT TO SCALE)

Figure 11.8: Site Bridge Layout of Bridge

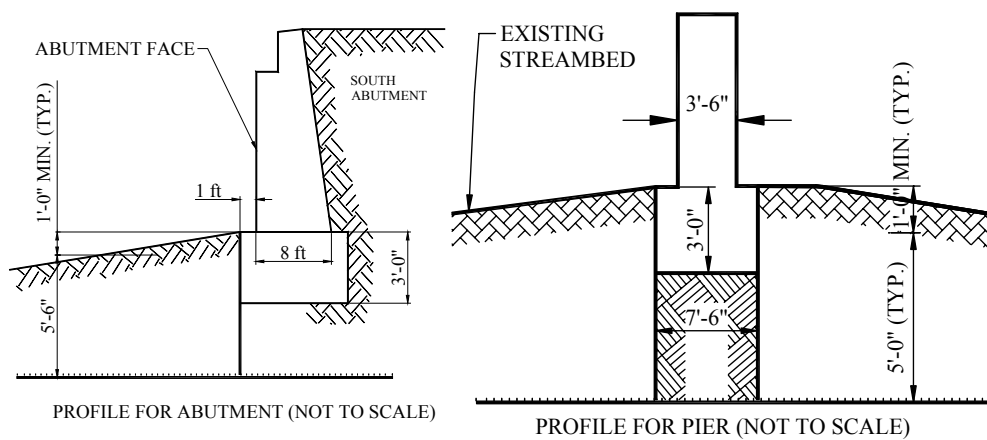


Figure 11.9: Elevations of Pier and Abutments.

### Design of Gabions for Piers

Critical Velocity =  $V_{100} = 5.08$  ft/sec. As per Table 11.1, for critical velocity < 11 ft/sec,

Thickness of Gabion = 0.5 ft

(Note: Thickness is the same as per Table 11.2)

Maximum Stone Size = 4.3 inch

Minimum Stone Size = 3.3 inch

Stone size as per Equation (11-1) = 4.0 inch

Obtained using the following parameters in Equation (11-1)

$$\Psi_{CR} = 0.07$$

$$p = 0.2 < 0.4$$

$$\mu = 0.75 \text{ for continuous protection}$$

$$s = 2.24$$

$$K_T = 1.0 \text{ for normal river turbulence}$$

$$K_Y = (d_{n50} / y_0)^{0.2}$$

$$y_0 = 6.12 \text{ ft}$$

$$\varepsilon = \text{Angle of bank to the horizontal} = 1V: 2.5H = 21.8^\circ$$

$$\phi = \text{Internal angle of friction of the revetment} = 46.5^\circ$$

$$\chi = \text{Angle of the channel invert to the horizontal} \\ = 2.23^\circ$$

and assuming an initial value of  $d_{n50} = 3.3$  inch,  $d_{n50} = 4.04$  inch is obtained after several iterations in Equation (11-1).

Hence, recommended stone sizes = 4.0 to 4.3 inch

Minimum Volume of Gabions for Piers (Equation 11-2)

$$= 0.20 \text{ ft}^3$$

Calculated by assuming following values in Equation (11-2) as:

$$U = \text{Approach design flow velocity in ft/sec} = 5.08 \text{ ft/s}$$

$$K = 1.5 \text{ for round nosed piers}$$

$$\rho_r = \text{Density of rock in the basket or box} = 140 \text{ lb/ft}^3$$

$$\rho = \text{Density of water} = 62.4 \text{ lb/cft}$$

$$g = 32.2 \text{ ft/s}^2$$

Minimum height of basket = 6 inch

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

$$> 1.5 \times 4.3 \text{ inch} = 5.7 \text{ inch}$$

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

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. Stone filling of filter fabric to be as per manufacturer's requirements.

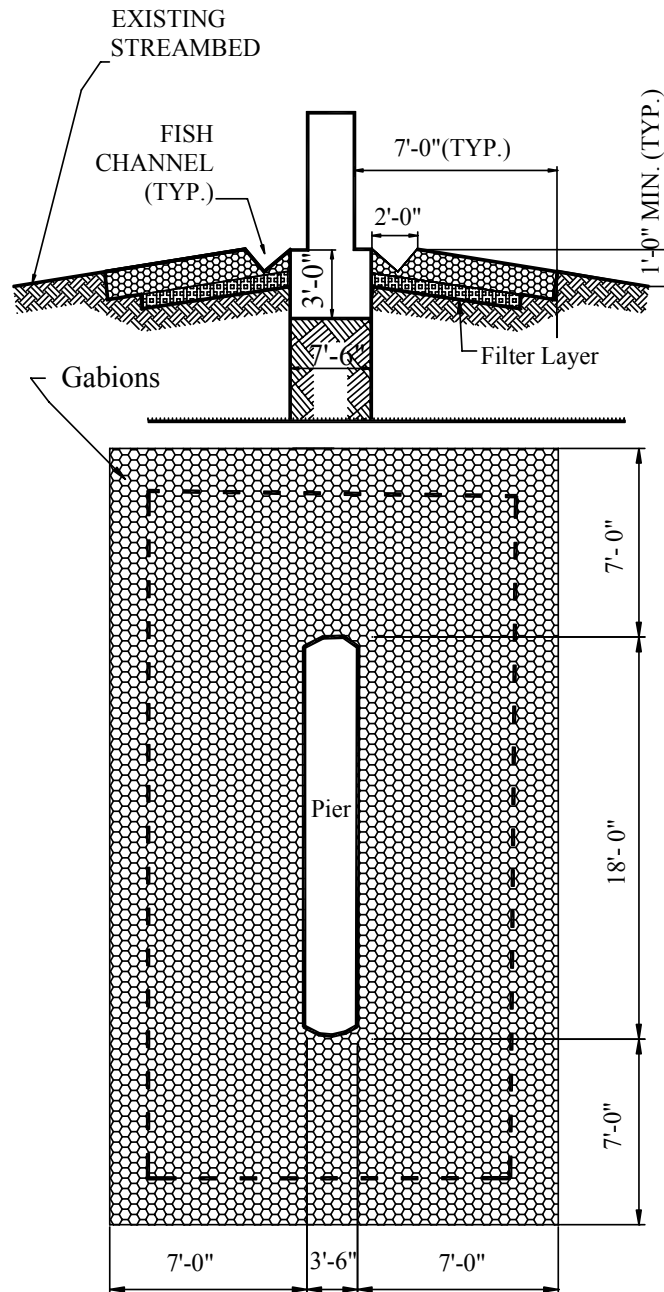


Figure 11.10: Extent of Gabions around Example Bridge Pier.

### Design of Gabions for Abutments:

Critical Velocity = Average Velocity for 100 years,  $V_{100} = 5.54$  ft/sec. As per Table 11.1, 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 11-1 (Follow the procedure for piers)	= 5.0 inch
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

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

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 Chapter 14, i.e., the extent of gabions for vertical wall abutments should be the greater of

$$2W = 2 \times 8 = 16 \text{ ft}$$

$$X + 18'' + y \cot \phi = 1 + 1.5 + 5.5 \cot (46.5^\circ) = 7.72 \text{ ft}$$

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 width of the bridge opening is 30 feet.

## **Articulated Concrete Blocks**

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 are 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 11.11 shows examples of interlocking and cable-tied block systems.

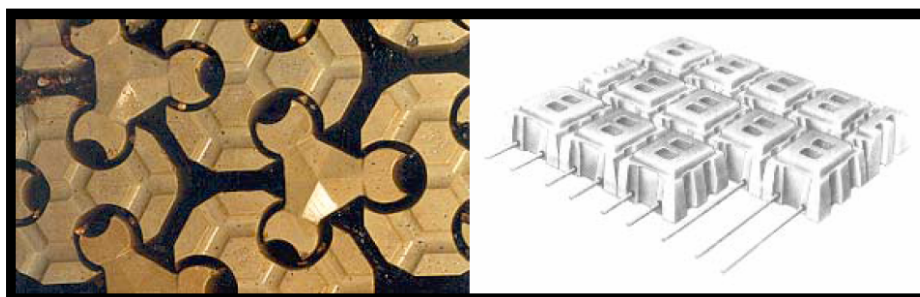
Cable Tied Blocks or Cable Tied Mattresses are the most popular forms of ACBs. They are concrete blocks tied together by steel cables to form a mat, as shown in Figure 11.12. The cables are either galvanized steel or polyester. The network of manageable blocks tied together has the capability of resisting 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 are capable of moving large stones. 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. They 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.

### **Advantages of Articulated Concrete Blocks**

1. Flexibility and ability to withstand strong currents,
2. A Pre-attached geotextile,
3. Resistance to ice
4. Cost-competitiveness.

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



(a)

(b)

Figure 11.11: Examples of Interlocking and Cable -Tied Articulated Concrete Blocks.

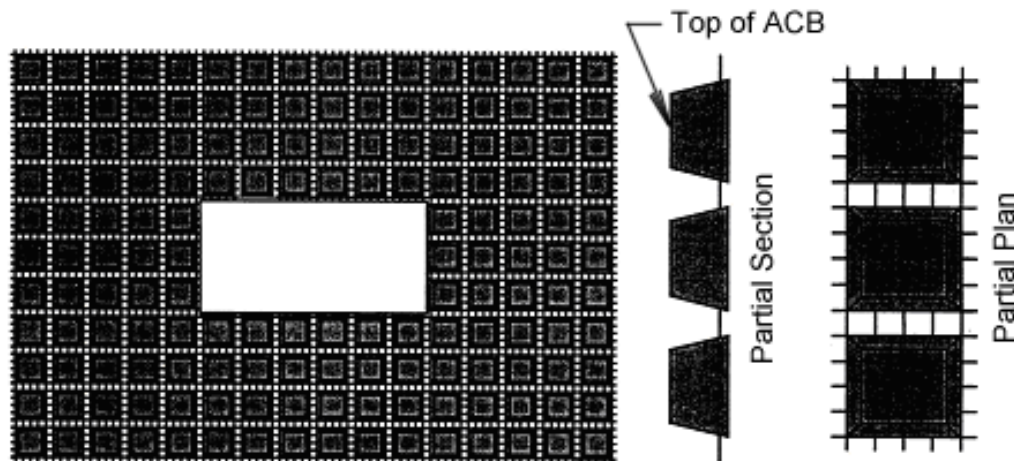


Figure 11.12: Cable-tied blocks for bridge scour countermeasure.

### **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, this 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.

### **General Guidelines for Articulated Concrete Blocks**

The following general guidelines should be followed during the design of articulated concrete blocks.

1. **Limitations:** ACBs may suffer from the following limitation.
  - Steel cables are likely to become corroded.
  - The blocks need to be anchored to the substructure, which may transfer tensile stresses to 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.
2. **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.

3. **Constructability Issues:** Following constructability issues should be considered during the design of ACBs.
- Construction observation/inspection to ensure that blocks are installed within the design tolerance is essential for a 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 the 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 may be 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.
4. **Costs:** Typically laid-in-place wet placement cost is approximately \$15-16/ft<sup>2</sup>. Cost to place cable-tied blocks around a 4 ft x 20 ft rectangular pier should be estimated based on local conditions.

### **Design Guidelines for Articulated Concrete 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 ACBs 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 ACBs for bridge abutments.

The failure of ACBs 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. Figures 11.13 and 11.14 show sample design charts in Design Guideline 4 of HEC-23. The design chart in Figure 11.13 represents the stability of ACB's placed flat on the channel bed neglecting the influence of abutment slope. Chart in Figure 11.14 accounts for the

effect of abutment slope in the factor of safety. The factor of safety can then be computed by

$$SF = \frac{\tau_a}{\tau_0}(SF_a)K_1 \text{ or } SF = \frac{V_a}{V_0}(SF_a)K_1 \quad (11-3)$$

where

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

$\tau_0$  and  $V_0$  = Design shear stress and velocity

$K_1$  = side slope correction factor.

$SF_a$  = Factor of Safety in developing charts.

**Notes:**

*Charts in Figures 11.13 and 11.14 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 are 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.*

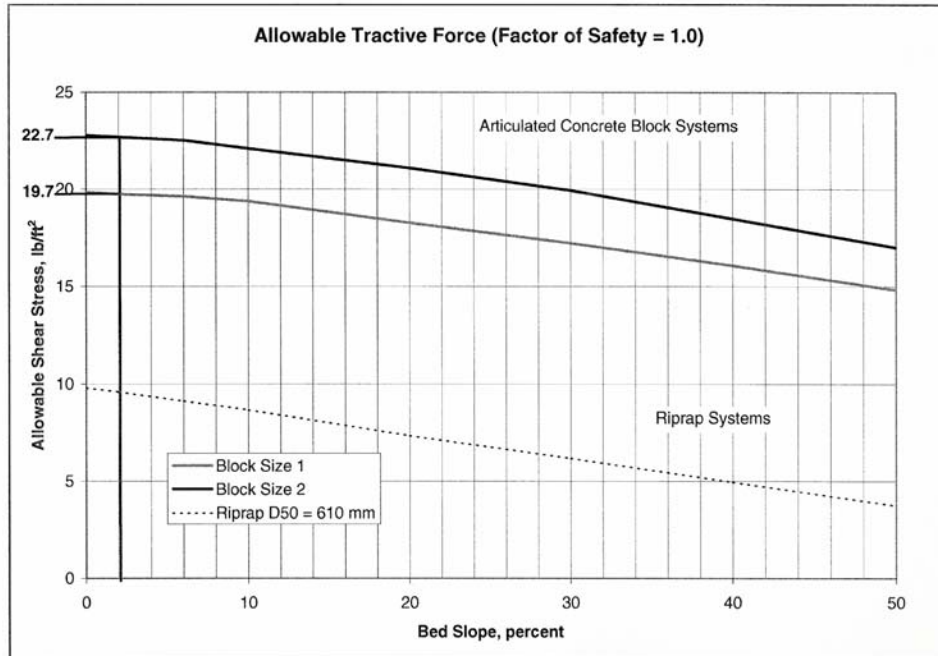


Figure 11.13: Chart for Allowable Shear Stress vs. Bed Slope (From HEC-23, **chart for Illustration only**, use similar chart provided by the manufacturer should be used for the design).

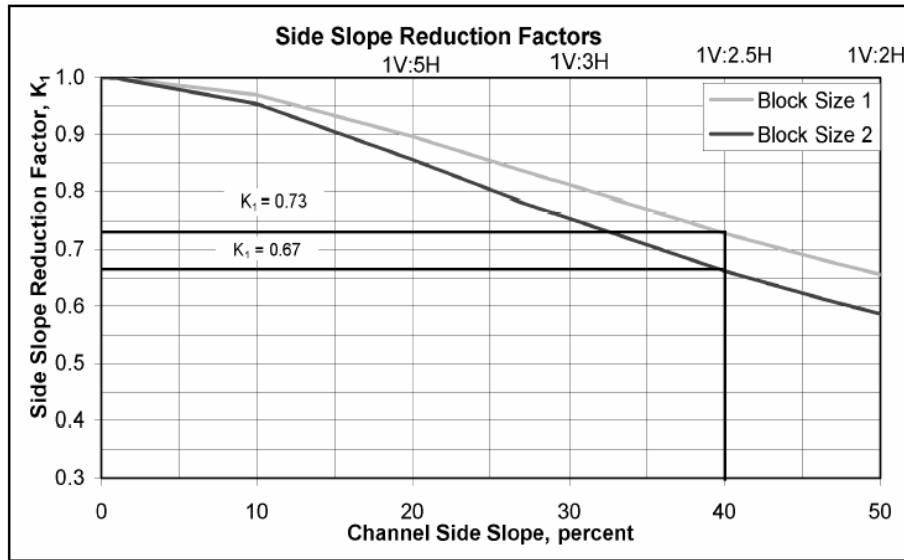


Figure 11.14: Chart for Side Slope Correction Factor (From HEC-23, **chart for illustration only**, use similar chart provided by the manufacturer should be used for the design).

For cases when design charts provided by the manufacturer are not available, Equation (11-4) based on a factor of safety approach in HEC-23 can be used to calculate the factor of safety as

$$SF = \frac{\cos\left(\frac{l_2}{l_1}\right)}{\eta'\left(\frac{l_2}{l_1}\right) + \sin\theta\cos\beta + \frac{l_3F'_D \cos\delta + l_4F'_L}{l_1W_A}} \quad (11-4)$$

where

$l_1$  and  $l_2$  = Moment arms of the weight of the block for side slope and longitudinal slope as shown in Figure 11.15

$l_3$  and  $l_4$  = Moment arms of the lift and drag forces on the block, as shown in Figure 11.15.

$W_A$  = 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

$\theta$  = Side slope angle with respect to the horizontal plane

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

$\lambda$  = Angle between horizontal plane and velocity vector = Bed slope when flow is parallel to the bed

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

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

$$\eta' = \left\{ \frac{\frac{M}{N} + \sin(\lambda + \beta)}{\frac{M}{N} + 1} \right\} \eta; \quad \frac{M}{N} = \frac{l_4 F_L}{l_2 F_D}$$

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

$\tau_0$  = Shear stress on the channel boundaries

$\tau_c$  = Critical shear stress when failure occurs

$F_D$  = Drag Force on the block

$F_L$  = Lift Force on the block

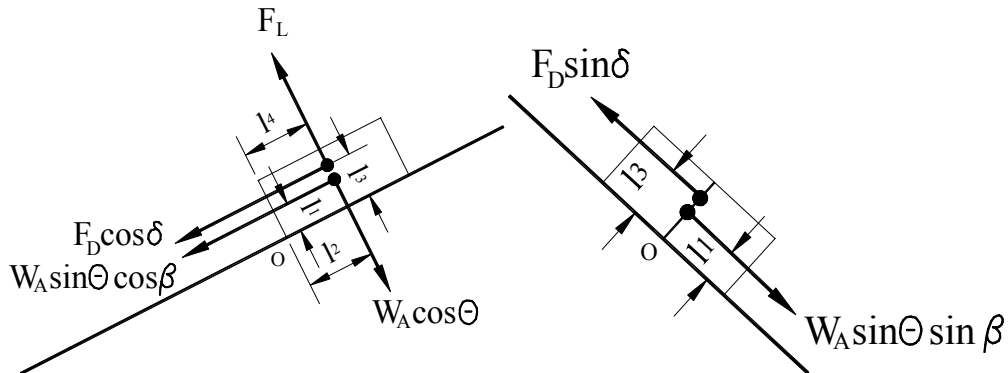


Figure 11.15: Definitions of Moment-Arms in Equation (11-4).

Cable-tied blocks should be designed by Equation (11-4) with much higher factor of safety than that using the Chart in Figures 11.13 and 11.14 to account for uncertainty in hydraulic characteristics of blocks.

## 2. Layout at Bridge Abutments

Layout dimensions of ACBs around bridge abutments should be the same as that for



riprap in Chapter 14.

### 3. Sizing at Bridge Piers

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

$$\zeta = a_{cb} \frac{\rho_{cb}}{\rho_{cb} - \rho} \rho V^2 \quad (11-5)$$

where:

$$a_{cb} = 0.20$$

$\rho$  = density of water = 62.166 lb/ft<sup>3</sup>

$\rho_{cb}$  = 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  $\zeta$  by the relation

$$\zeta = \rho_{cb} g H_{cb} (1 - p) \quad (11-6)$$

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 11.16].

In the event that the angle of attack  $\beta$  exceeds  $15^\circ$ , the cover is taken at least  $4D/\cos(\beta)$  [See Figure 11.16].

### 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.

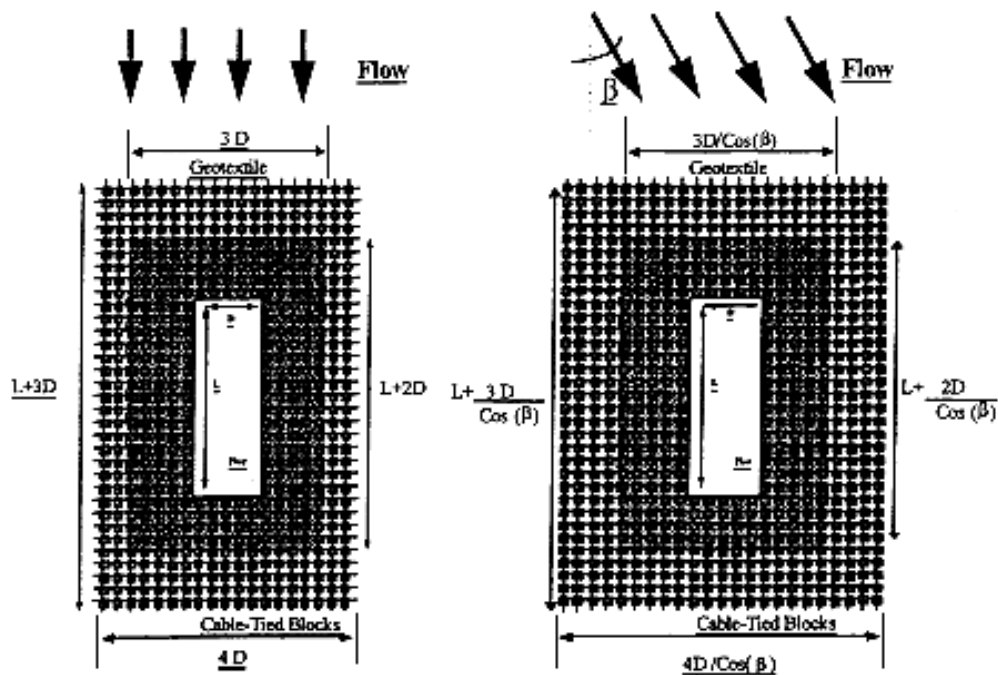


Figure 11.16: Installation and Layout of Cable-Tied Blocks around Bridge Piers.

## 6. Geotextile Filter at Piers and Abutments

Geotextile filter should not be used for gravel bed rivers. For sand bed rivers fasten the geotextile filter firmly under the base of block mattresses. The geotextile material should extend outwards  $1D$  from every face of piers and should never extend as far out as the mattress, as shown in Figure 11.16.

## 7. Pier Sealing Requirement

Performance of cable-tied blocks depends on sealing between the 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:

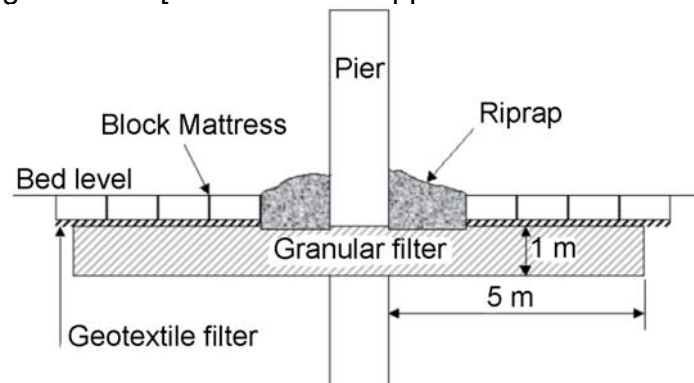
- Anchors: Use of tension anchors in addition to grout around the pier seal can provide additional support for the mat. Grout at the pier seal will reduce scouring underneath the mat. Following types of anchors, based on field applications by MnDOT for a pier at TH 32 over Clearwater River at Red Lake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

Duckbill anchors, 3 - 4 ft deep. Use Duckbill anchors at corners and about every 8 ft around pier footings.

- 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].

- 
- Pier No. 2
- 34'-0"
- Distribution Slab
- Seal
- Place Grout Bags to form Bulkhead tight to underside of seal (Typ)
- #4 Bar (Typ.)  
? 4'-0" O.C.  
(Min. 2 per Bag)
- Pier No. 2
- Pier No. 2
- 2'-0" Overlap of Geotextile Between adjacent mats (Typ)
- Grout Fill
- Note: Pump grout into each long tube until discharge at monitor tube. Close valve and repeat operation at next long tube until grout completely fills voided area below seal

d. Pier seal can be provided by placing granular filter material to a depth of about 3 ft below the stream bed for about 16 ft around the pier. The geotextile filter and block mat placed on the stream bed 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 11.18 [Based on field applications in Netherlands].



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### Design Example for Cable Tied Blocks

A typical New Jersey bridge 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, $\tau_0 = 4.9 \text{ lb/ft}^2$	Bed Shear, $\tau_0 = 4.4 \text{ lb/ft}^2$

*Charts in Figures 11.13 and 11.14 are considered to be supplied by the manufacturers for the design of blocks.*

### Design of Cable-Tied Blocks for Abutments using Charts

From Figure 11.13, 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:

$$\tau_a = 19.7 \text{ lb/ft}^2 \quad (\text{allowable shear stress for Block Size 1})$$

$$\tau_a = 22.7 \text{ lb/ft}^2 \quad (\text{allowable shear stress for Block Size 2})$$

From Figure 11.14, reduction factor for a 1V: 2.5H (40%) side slope of the abutment is:

$$K_1 = 0.73 \quad (\text{for Block Size 1})$$

$$K_1 = 0.67 \quad (\text{for Block Size 2})$$

The factor of safety for the two blocks can be calculated from Equation (11-3) as:

$$SF = \frac{\tau_a}{\tau_0} (SF_a) K_1 = \frac{19.7}{4.9} (1) 0.73 = 2.9 \quad (\text{for Block Size 1})$$

$$SF = \frac{\tau_a}{\tau_0} (SF_a) K_1 = \frac{22.7}{4.4} (1) 0.67 = 3.5 \quad (\text{for Block Size 2})$$

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 Using Equation 11-4

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.)	$\Delta Z$ (in.)	$\omega$ (in.)	$\tau_c^*$ (lb/ft <sup>2</sup> )
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

$\tau_c^*$  has been determined from testing.

Factor of safety parameters:

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

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

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

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

Conservatively assuming that  $F_L = F_D$ ,

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

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

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

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

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

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

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

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

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:

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

$$\text{Design Velocity for Block Size 1} = 5.10 \text{ ft/s}$$

$$\text{Design Velocity for Block Size 2} = 5.57 \text{ ft/s}$$

$$\text{Drag Force, } F_D' \text{ for Block Size 1} = 1.14$$

$$\text{Drag Force, } F_D' \text{ for Block Size 2} = 1.37$$

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:

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

Factor of Safety for Block Size 2:

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

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

The layout and extent of cable-tied blocks for bridge abutments should be the same as that discussed for riprap in Chapter 14.

### ***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 given in this chapter can be used. For example, for concrete cable-tied blocks with  $\rho_{cb} = 115 \text{ lb/ft}^3$ , weight per unit area of the mattress for design velocity,  $U = 5.10 \text{ ft/sec}$  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 (11-6) 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 11.16 as:

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

The layout and extent of cable-tied block is shown in Figure 11.19 below. Cable-tied block mat should be sealed to piers as per "Pier Sealing Requirement" described above.

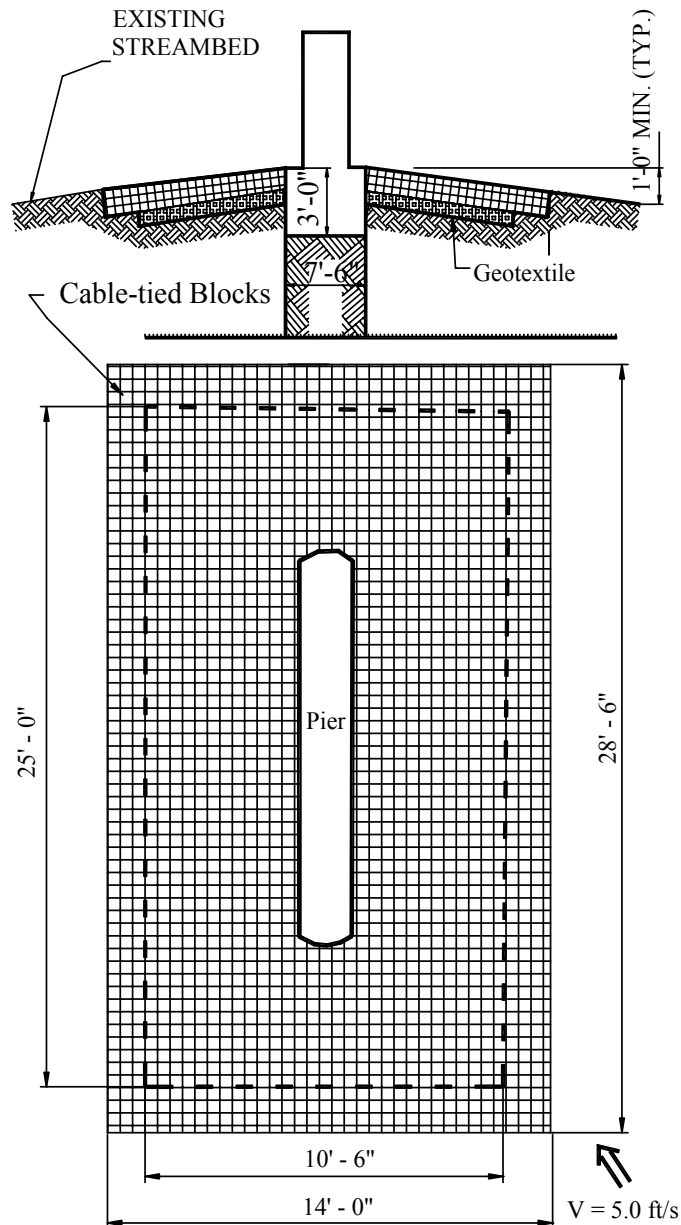


Figure 11.19: Layout of Cable-Tied Blocks around Bridge Pier.

## Concrete Armor Units

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 is 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 11.20 shows examples of concrete armor units.

### 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 11.20) and A-jacks (shown in Figure 11.21) developed by Armortec Company. Detailed guidelines and examples of scour countermeasure using Toskanes are presented in Design Guideline 6 of HEC-23.

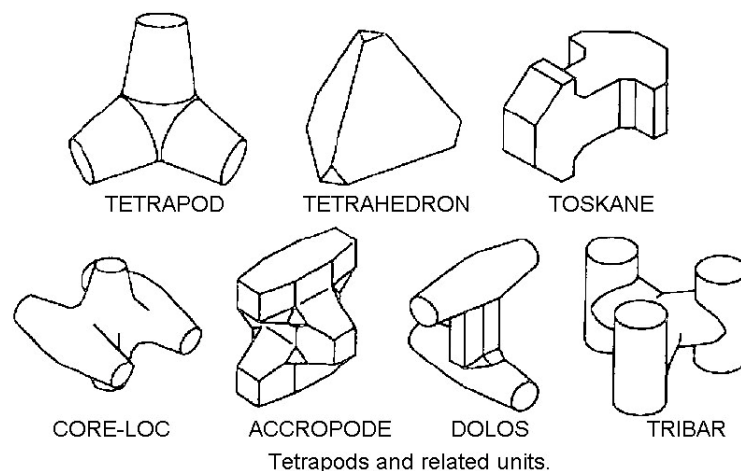


Figure 11.20: Precast concrete armor units

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 11.21: A-Jacks Module For Pier Scour Countermeasures.

#### General Guidelines for Pre-Cast Concrete Armor Units

1. **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 and being undermined during high turbulence.
2. **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.
3. **Construction Permits:** Stream encroachment and other applicable permits will be required. Refer to Guidelines given for permit application in NJDEP Stream Encroachment Technical Manual.
4. **Durability and Maintenance:** The following types of failures may occur and may be avoided by good construction practice:
  - a. Undermining of blocks may occur because of bed movement.
  - b. Development of a gap between edge of concrete blocks and the structure, leading to loss of underlying material.
  - c. Edge failure due to formation of a scour hole in the natural bed adjacent to ACB protection in which blocks can fall in.
  - d. Movement and progressive collapse at slopes when edges are not tied in.

## Design Guidelines for Concrete Armor Units: Toskanes

Toskanes are the most popular form of concrete armor units for scour protection of bridge piers and abutments. Figure 11.22 shows CSU Toskane design parameters and dimensions. 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 and should be used only when design guidelines provided by manufacturers are not available.

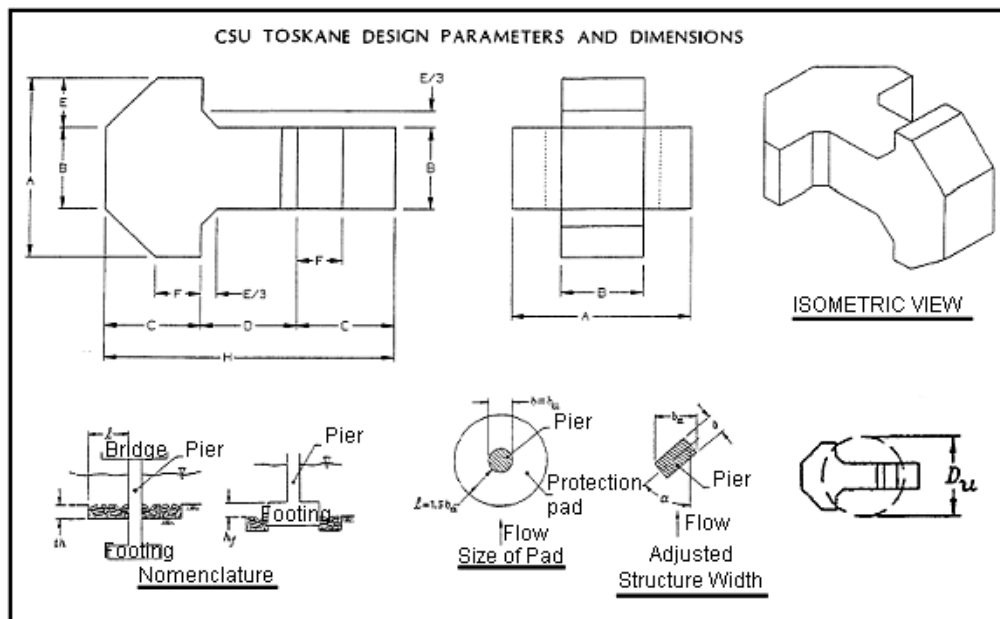


Figure 11.22: Toskane Shapes.

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

$$V_v = 1.5V_0C_lC_sC_hC_i \quad (11-7)$$

where factor "1.5" is the factor of safety. Other parameters in Equation (11-7) 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,  $\alpha$ , is greater than  $5^\circ$ , 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.*

$C_i$  = Random or pattern installation factor. A random installation refers to the units being dumped into position. In a pattern installation, every Toskane is uniformly placed to create a geometric pattern around the pier. 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 ( $\alpha$ ) for high flow conditions  $< 5^\circ$

$b_a = L \sin \alpha + b \cos \alpha$  if the angle attack ( $\alpha$ ) for high flow conditions is  $> 5^\circ$ ,  $L$  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  $5 \text{ ft} \leq b \leq 20$  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:

$$D_u = \frac{0.255V_v \sqrt{\frac{b_a}{g}}}{S_g - 1} \quad (11-8)$$

where  $S_g$  is specific gravity of Toskanes. Table 11.3 shows Toskanes design parameters and dimensions. Table 11.4 shows Toskanes design dimensions in terms of Toskanes height  $H$ . Table 11.5 shows recommended standard sizes of Toskanes.

Sizing of Toskanes can be done through the following steps:

- Determine  $D_u$  from Equation (11-8) based on velocity value,  $V_v$ , and the adjusted structure width,  $b_a$ .
- Select standard Toskane size from Table 11.5 such that  $D_u$  of selected Toskane is greater than that calculated from Equation (11-8).
- Check the  $b_a / D_u$  ratio using the diameter,  $D_u$ , of a standard Toskane size in Table 11.5. If the  $b_a / D_u > 21$ , select the next larger size of Toskane. Repeat until  $b_a / D_u < 21$ .

Table 11.3: Toskane Design Parameters and Dimensions.

Design Parameter Dimensions	
Toskane Length (H)	$1.608D_u$
Equivalent Spherical Dimension ( $D_u$ )	$0.622H$
Volume (V)	$0.5236D_u^3 = 0.1263H^3$
Specific Weight ( $\gamma$ )	$23.5 \text{ kN/m}^3 = 150 \text{ lb/ft}^3$
Density ( $\rho$ )	$2400 \text{ kg/m}^3 = 4.66 \text{ Slug/ft}^3$
No. of Toskanes Per Unit area, (N)**	$0.85V^{-2/3} = 1.309D_u^{-2}$
2 Layer Thickness (th)	$2.0D_u = 1.24H$
Filter Requirements	$D_{85(\text{filter})} = 0.22D_u$
Size of Pad (l)	$l_{\min} = 1.5b_a \text{ (Piers)}$ $l_{\min} = 2.0b_a \text{ (Abutments)}$
**Toskanes per unit area assuming a 2-layer thickness of $2D_u$	

Table 11.4: Toskanes Design Dimensions.

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

Table 11.5: Standard Sizes of Toskanes.

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

4. **Pad radius:**

Use  $1.5b_a$  pad radius 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 11.3 to determine number of Toskanes per unit area and the pad thickness. Pads with randomly placed units should 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. The filter should be as recommended by the manufacturer.

If the filter is granular,  $d_{85}$  of the filter material directly below the Toskane layer is  $0.22D_u$  as per Table 11.3. Additional layers of filter, that may be needed based on the gradation of the bed material.

7. **Placement**

Toskanes can be installed around bridge piers and abutments in the configuration as shown in Figures 11.23 and 11.24.

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.

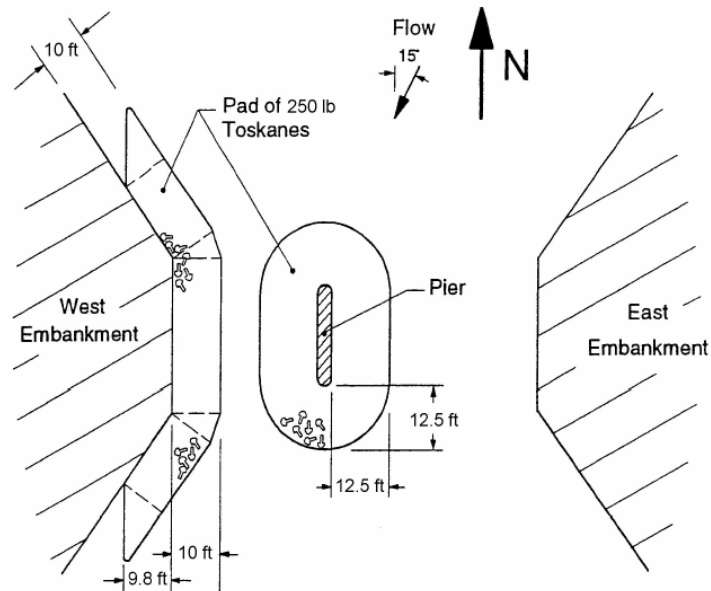


Figure 11.23: Typical Placement of Toskanes around Bridge Piers and Abutments.

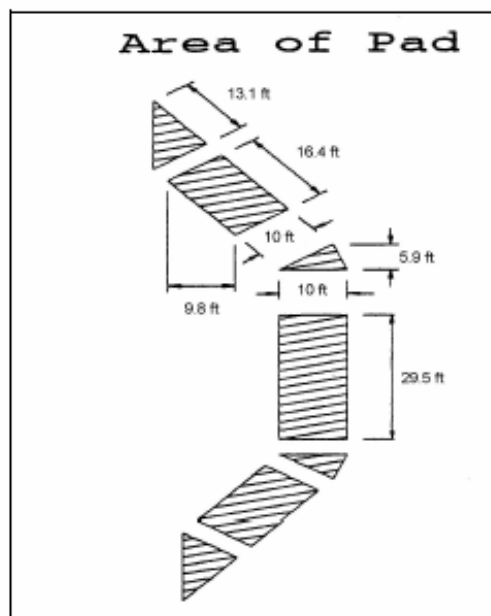


Figure 11.24: Layout of Toskanes for Bridge Abutment.

### Design Example of Toskanes for Piers

A typical bridge 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 6.10 ft/s for 100 years flood and has an angle of attack of  $10^\circ$ . 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,  $\alpha = 10^\circ > 5^\circ$ )

$C_h = 1.0$  (Top of the pad is level with the bed)

$C_i = 1.0$  (Randomly installed pad of Toskanes)

$$V_v = 1.5 V_0 C_l C_s C_h C_i = (1.5)(6.10)(1.1)(1.0)(1.0)(1.0) = 10.06 \text{ ft/s}$$

2. Adjusted structure width,  $b_a$  (ft)

$$b_a = L \sin \alpha + b \cos \alpha = 6.57 \text{ ft}$$

Angle of attack,  $\alpha = 10^\circ$ .

Length of pier,  $L = 18 \text{ ft}$ .

Pier width,  $b = 3.3 \text{ ft}$ .

3. Using Equation (11-8), the equivalent spherical diameter,  $D_u$ , for  $V_v = 10.06 \text{ ft/s}$ ,  $b_a = 6.57 \text{ ft}$  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 11.5, a standard sized 250 lb Toskane unit with  $D_u = 1.47 \text{ ft}$  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.

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

The Toskanes will be installed around the pier, a horizontal distance of 10 ft from the wall, in the layout shown in Figure 11.18.

5. From Table 11.3, the number of Toskanes per unit area for the 250 lb Toskane size with a pad thickness of  $2D_u$  is  $1.309/1.47^2 = 0.61 \text{ Toskanes/ft}^2$ . Hence, the total area of the pad in Figure is:

$$\text{Area} = 2(14.5(10)) + (\pi(11.75^2 - 1.75^2)) = 1139 \text{ ft}^2$$

$$\text{No. of Toskanes} = 0.61 (1139) = 695 \text{ Toskanes}$$

The pad thickness is  $2Du = 3$  ft

6. Since the bed material consists of cobbles and gravel, a granular filter with  $d_{85} = 95\text{mm} = 3.75$  inches is added beneath the pad of Toskanes.
7. Cobbles and gravel are sufficiently large so no additional filter layers are required.

Toskanes should be installed around the pier in a typical configuration shown in Figure 11.25 below.

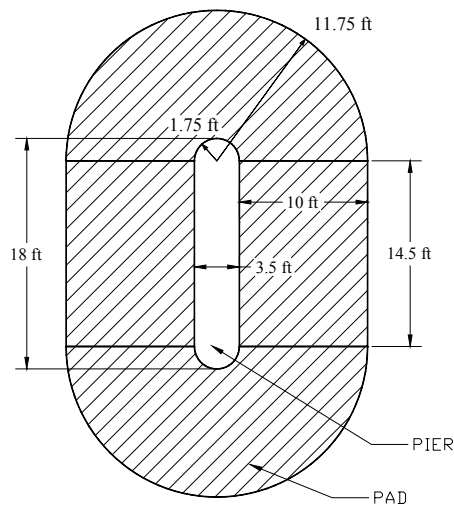


Figure 11.25: Layout of Toskanes around the Bridge Pier.

### Design Example of Toskanes for Abutments

The bridge Tributary to Lamington River 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.  $V_{100} = 5.54$  ft/s.

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

$$V_v = 1.5 V_0 C_l C_s C_h C_i = 6.36 \text{ ft/s}$$

$C_l = 0.9$  (The abutment is located near the bank, outside of the thalweg)

$C_s = 0.85$  (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)

2. Adjusted structure width,  $b_a$  (ft)



Since the west riverbank 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. Therefore an average value of  $b_a$  is

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

Hence,  $b_a = 5 \text{ ft}$

3. Using Equation 11-8, the equivalent spherical diameter,  $D_u$ , for  $V_v = 6.36 \text{ ft/s}$ ,  $b_a = 5 \text{ ft}$  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 11.3, 250 lb Toskane with  $D_u = 1.47 \text{ ft}$  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.

$$\text{Pad Radius, } l = 2.0(5.0) = 10 \text{ ft}$$

The Toskanes will be installed along the abutment and wingwalls a horizontal distance of 10 ft from the wall in a pattern similar to that in Figure 11.24. Other dimensions in Figure 11.24 depend on specific abutment and wingwall dimensions.

5. The pad thickness is  $2D_u$ , which will result in 0.61 Toskanes/ft<sup>2</sup>. The total area of the pad in Figure 11.18 is,

$$\text{Area} = (10)(29.5) + 2(10)(16.4) + 2(0.5)(10)(5.9) + 2(0.5)(10)(13.1) = 813 \text{ ft}^2$$

$$\text{Number of Toskanes} = (813)(0.61) = 496 \text{ Toskanes.}$$

6. Granular filter with  $d_{85} = 3.75 \text{ inch}$  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.

## Design Guidelines for Concrete Armor Units: 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 estimated by equating the overturning moment due to total drag force to the resisting moment due to submerged weight,

$$F_d H_d = W_s L_w; F_d = 0.5 C_D \rho A V^2 \quad (11-9)$$

where:

- $F_d$ : Drag Force
- $C_d$ : Drag Coefficient = 1.05 (a value of 1.2 can be assumed for conservative design)
- $\rho$ : 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 (11-9) are obtained from physical characteristics of A-Jacks. Table 11.6 shows hydraulic characteristics of prototype size 5x4x5 A-Jacks module based on laboratory testing.

Table 11.6: Hydraulic characteristics of 5x4x5 A-Jacks modules

A-Jacks System	Tip-Tip Dimensions 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	16x52x40	1,030(467)	540(245)	10.7(3.3)
AJ-48	48	32x104x80	8,270(375)	4,300(1,950)	15.1(4.6)
AJ-72	72	48x156x120	27,900 (12,655)	14,500(6,577)	18.5(5.6)
AJ-96	96	64x208x160	66,200(30,028)	34,400(15,604)	21.4(6.5)
Notes:					
1. Volume of concrete in ft <sup>3</sup> for a 14-unit module is $14 \times 0.071 \times L^3$ , where $L$ is the tip-to-tip dimension of the armor unit.					
2. Values in table assume a unit weight (or mass) of 130 lbs/ft <sup>3</sup> (2,083 kg/m <sup>3</sup> ) for concrete.					

Table Reproduced from HEC-23

## 2. Geometry of A-jacks modules

Figure 11.26 shows recommended layout of A-Jacks modules around a pier of width “D” and unprotected depth of scour,  $y_s$  (as calculated using 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.

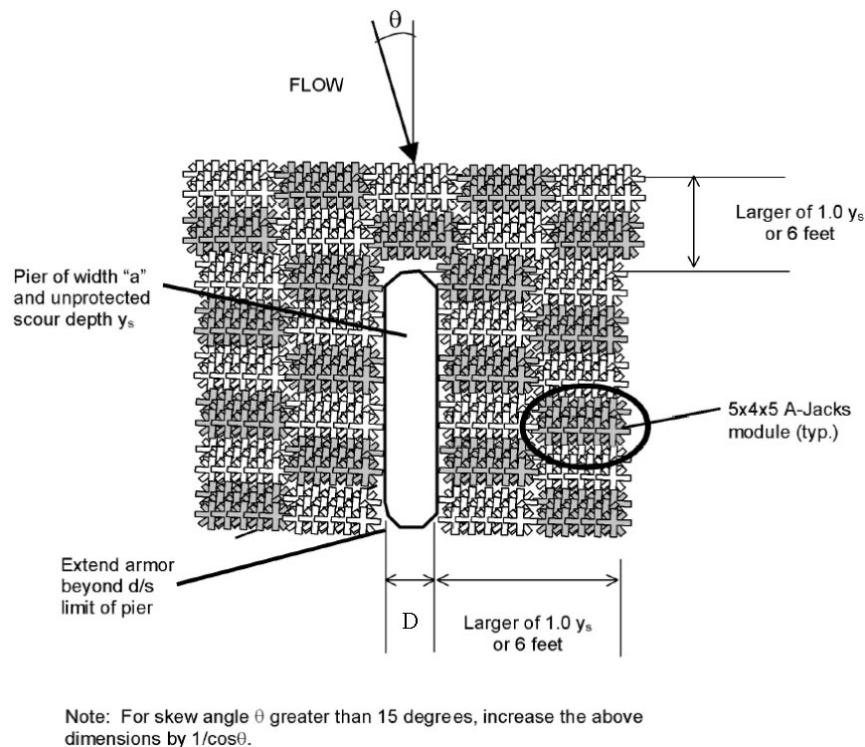


Figure 11.26: Typical Layout of A-Jacks Modules for Bridge Piers.

## 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

A 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 (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:

$$\begin{aligned}\text{Retention:} \quad & D_{85(\text{Lower})} > 0.25D_{15(\text{Upper})} \\ & D_{50(\text{Lower})} > 0.14D_{50(\text{Upper})}\end{aligned}$$

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

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

where  $D_{xx}$  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 the upper layer should be based on Table 11.7 criteria.

When using geotextile, a layer of blast stone must be placed on top before installing A-Jacks modules. Geotextile material should have 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 11.7 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)

## Grout Filled Bags and Mats

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.

It was found that properly installed grout mats and grout bags reduce scour depth to a degree generally comparable with riprap. In cases of small bridges, bags can be installed where it is difficult to bring in equipment for the placement of riprap.

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.

Figure 11.27 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 11.27 (b) and (c)) provided better protection.

### 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. For 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.

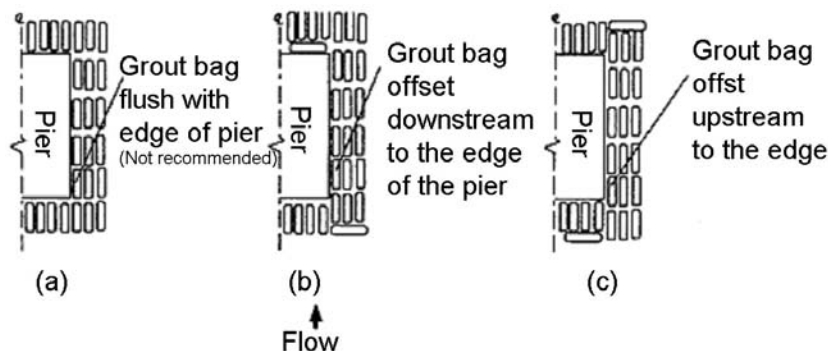


Figure 11.27: Grout Filled Bags and Mattresses for Pier Scour Countermeasures.

### **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 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.

### **General Guidelines for Grout Bags**

1. **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. The bags are filled up, generally on the site with dry sand, wet sand, dry mixture of 15 % cement and 85% dry sand, by weight or wet grout pumped into bags. The dry mixture of cement and sand hydrates and hardens on contact with water. Figures 11.28 to 11.33 show applications using grout bags at abutments and piers.
2. **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.
3. **Construction Permits:** Stream encroachment and other applicable permits will be required. Refer to Guidelines given for permit application in NJDEP Stream Encroachment Technical Manual.
4. **Durability and Maintenance:** The following types of failures may occur and should be avoided:
  - a. Undermining
  - b. Gradual collapse
  - c. Settlement of ground
  - d. 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.
5. **Cost:** Grout bags are more expensive than riprap. Costs in 2007 are estimated to be \$25 to \$30 per sq. ft. for each layer of grout bags.

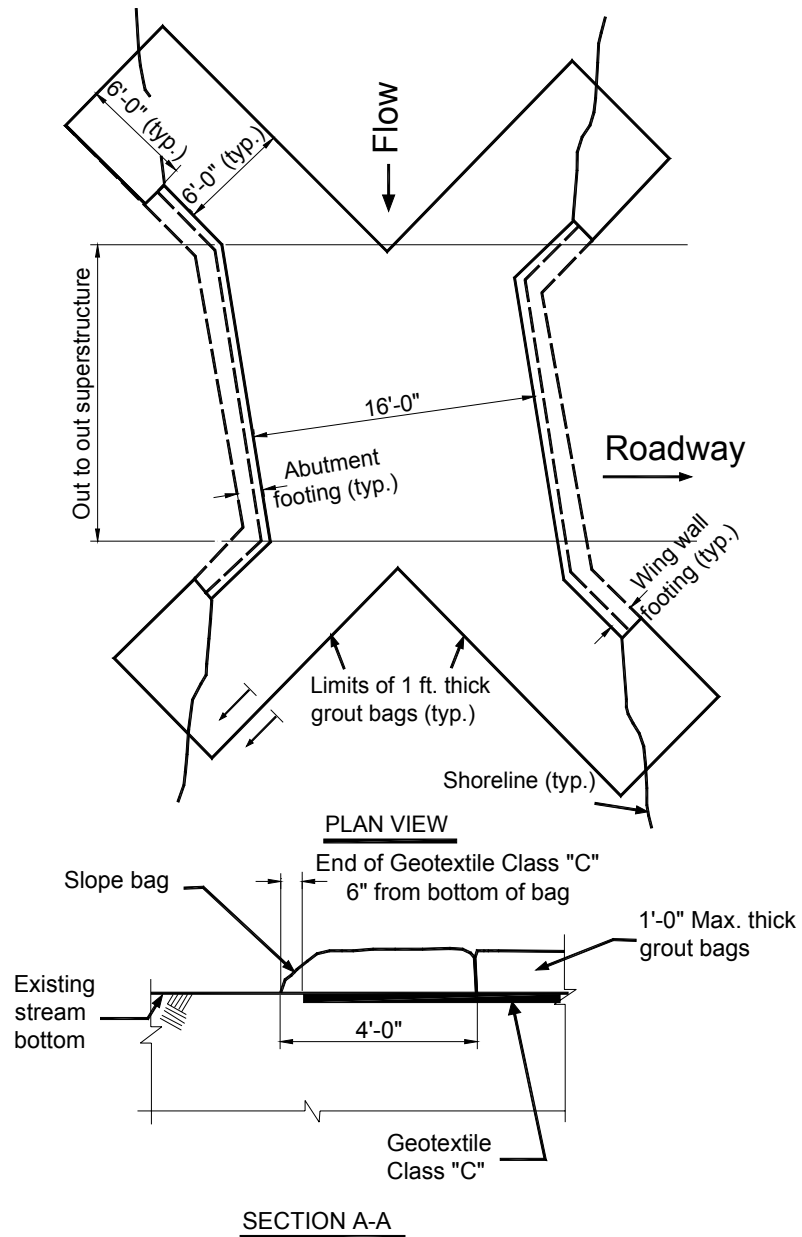


Figure 11.28: Plan View of Grout Bags (Case Where Scour Potential Exists for Full Channel Width) N.T.S

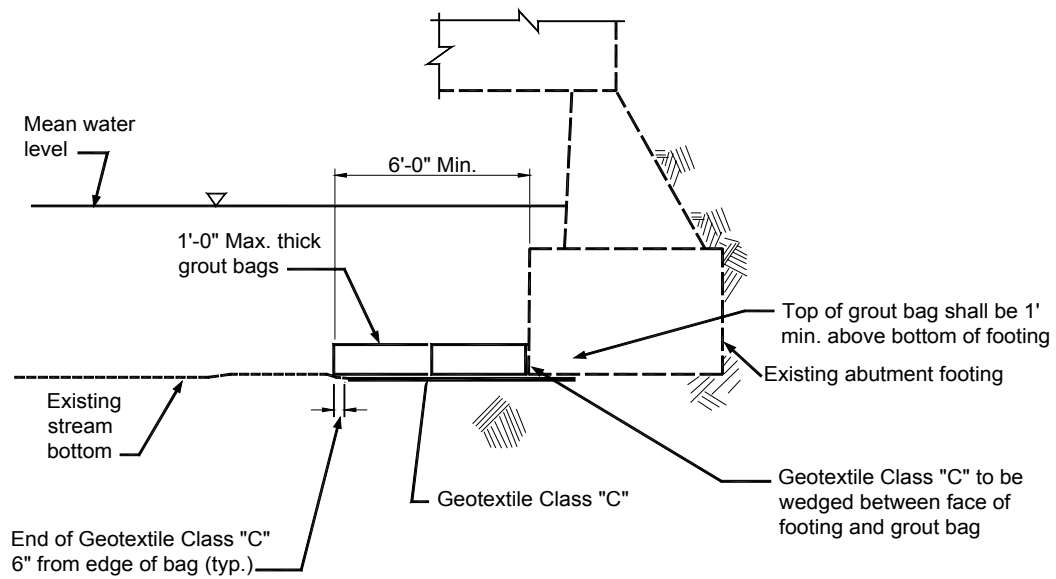


Figure 11.29: Grout Bag Section (Section through Abutment) Case Where Scour Potential Exists at Abutment N.T.S

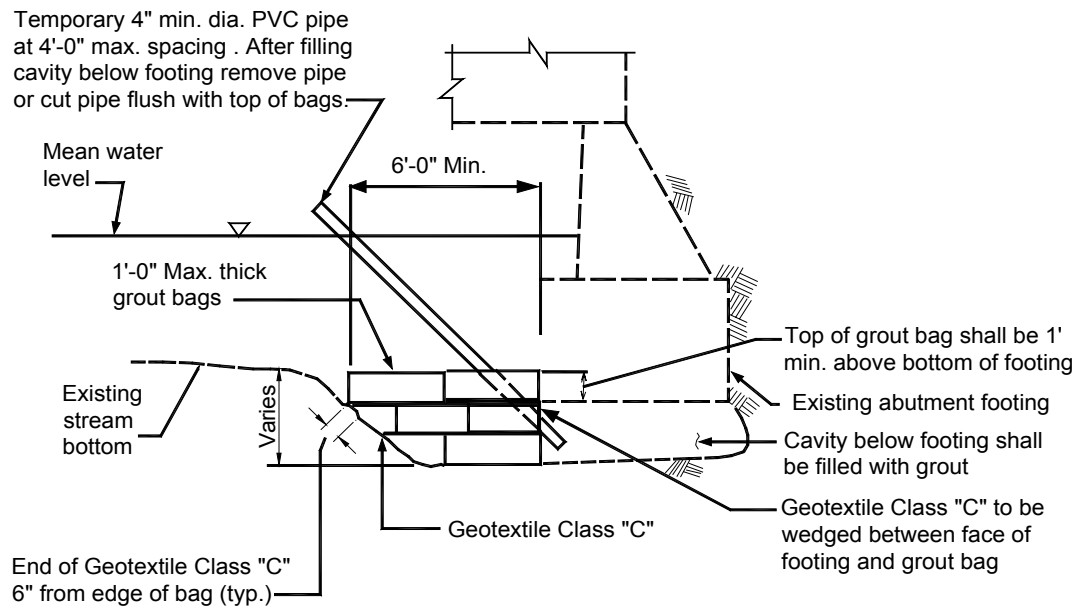


Figure 11.30: Grout Bag Section (Section Through Abutment) Case Where Scour and Undermining Has Occurred at Abutment N.T.S



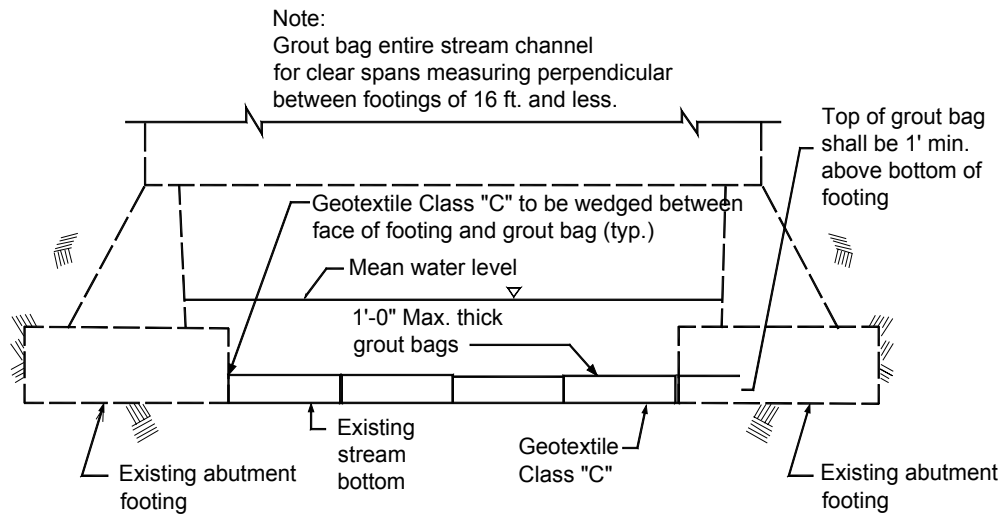
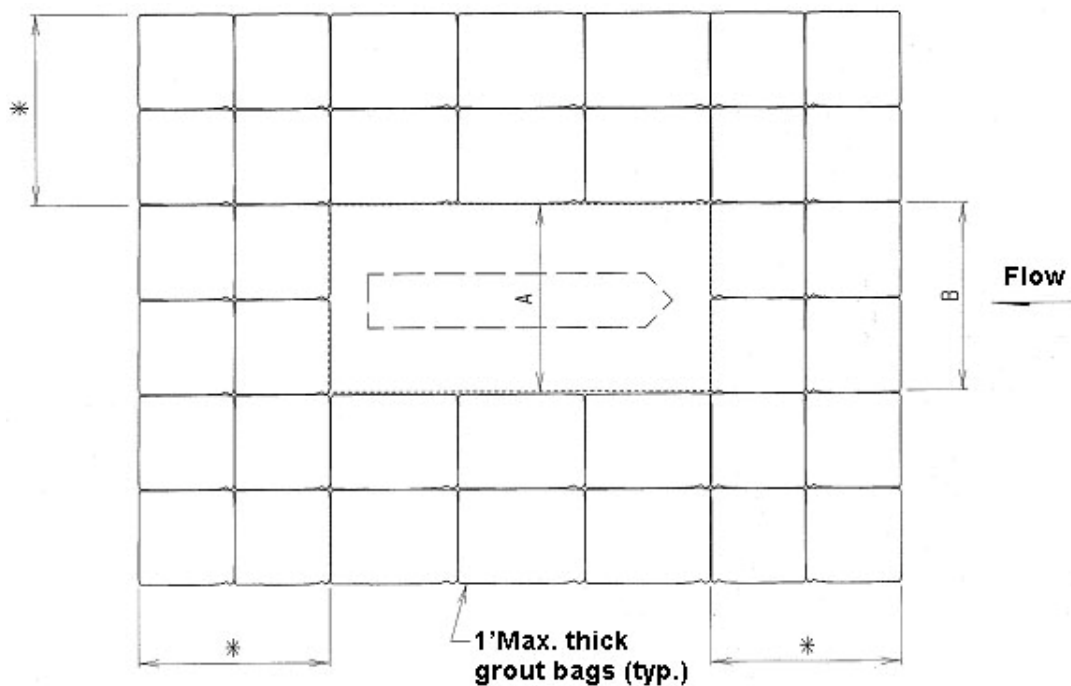


Figure 11.31: Section View of Grout Bag (Thru Abutments and Channel) Case where Scour Potential Exists for Full Channel Width N.T.S.



A = Width of Footing

B = Length of grout bags in front and behind the pier to match pier footing width

\* = 2A or 6 ft, whichever greater with a maximum of 12 ft.

Figure 11.32: Plan View of Grout Bag Installation at Pier N.T.S.

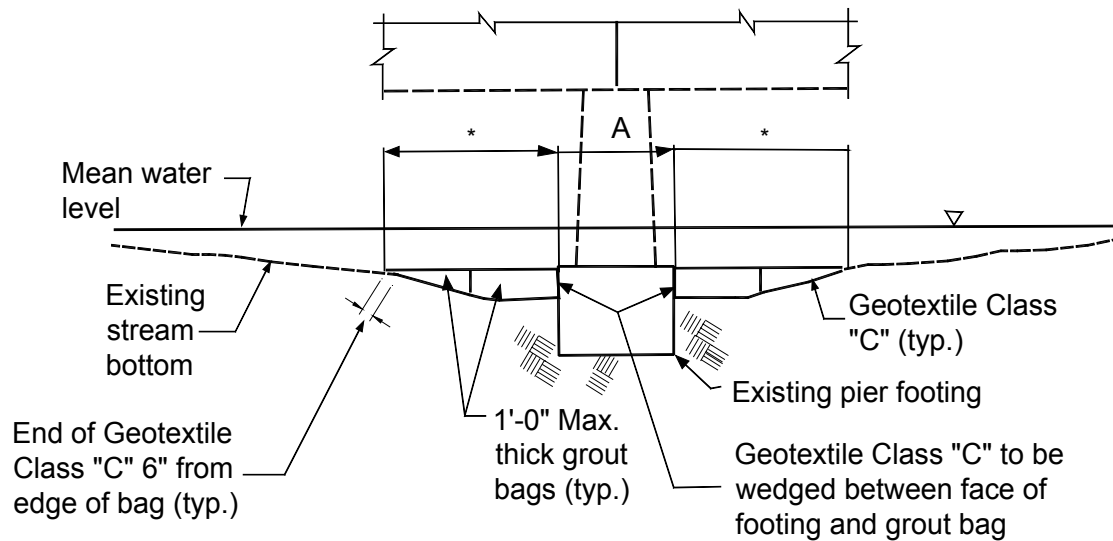


Figure 11.33: Grout Bag Section (Section Thru Pier) Case Where Scour Potential Exists at Pier (N.T.S.)

### Design Guidelines for Grout Bags

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<sup>3</sup>, while mattresses are larger in size up to 15 ft<sup>3</sup> 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.

## **CHAPTER 12**

### **RIVER TRAINING MEASURES**

#### **Introduction**

River training and flow altering countermeasures can be more important and effective than armoring since they harness the river and control the flood velocities. Due to cost considerations and special applications for higher velocity rivers, these measures may generally take a secondary role, when used in combination with the primary armoring. They are not attached to the bridge substructure and are used primarily to control floods.

#### **River Training Countermeasures**

River training structures modify a river's flow. They are distinctive in that they alter hydraulics to mitigate undesirable erosional and/or depositional conditions at a particular location or in a river flood plane. They are more suitable when local scour is a problem. River training structures can be constructed of various material types. Some of the common types of river training methods 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
7. Auxiliary bridge

River training structures are described as transverse, longitudinal or aerial depending on their orientation with respect to the stream flow.

1. Transverse river training structures 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 wide and where bend radii are less than 328 ft.
2. Longitudinal river training structures 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. Aerial river training structures cannot be described as transverse or longitudinal when acting as a system. This group also includes countermeasure "treatments" which have aerial characteristics such as channelization, flow relief, and sediment

detention. Examples of Aerial River Training are vertical (bed elevation control) countermeasures, such as sills or weirs.

### Flow Altering Countermeasures

These countermeasures are recommended for diverting scour away from bridge piers and should be used in combination with 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, 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 triangular configuration upstream of piers is shown in Figure 12.1 & 12.2.

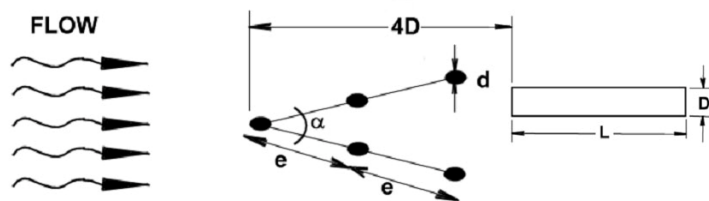


Figure 12.1: Plan of Upstream Sacrificial Piles

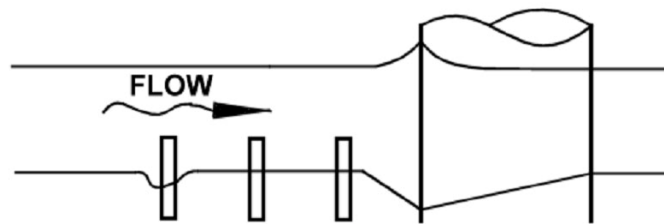
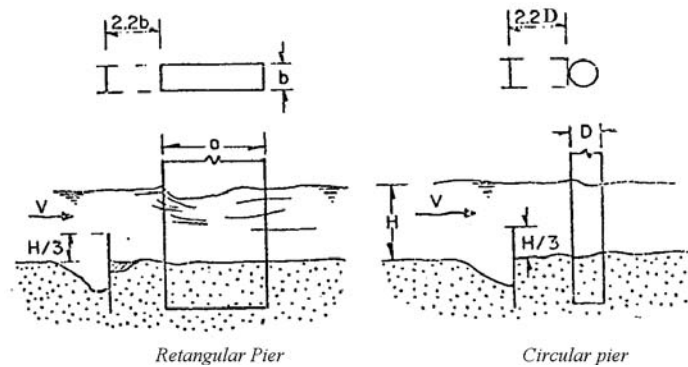


Figure 12.2: Elevation of Upstream Sacrificial Piles

2. **Upstream Sheet Piles:** Upstream sheet piles are placed upstream of bridge piers to arrest scour in the lee of sheet piles. Figure 12.3 shows the recommended configuration of sheet piles. The width of sheet piles should be equal to the width of bridge piers and they should protrude only one third of the depth above the river bed.



**Rectangular Pier**

**Circular Pier**

Figure 12.3: Installation Layout of Sheet piles upstream of rectangular and circular piers.

### Selection of River Training Countermeasures

The selection of river training countermeasures will be based on the following considerations:

1. Flood velocity: medium or high
2. Flow conditions: Overtopping or over bank
3. River: 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

Figure 12.4 shows examples of different types of river training countermeasures.

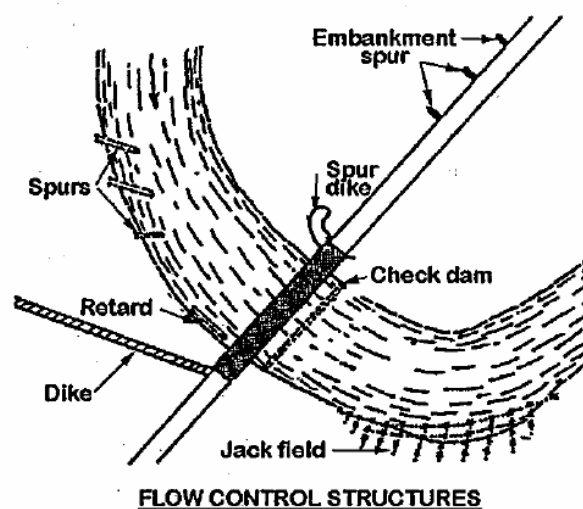


Figure 12.4: Examples of Different Types of River Training Measures.

## **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 should be made on project specific conditions.

## **Comparison of River Training Measures**

Descriptions of river training countermeasures, their use on the basis of scour types; advantages and disadvantages are discussed below. Recommendations for suitability to various conditions are discussed under "Remarks".

1. Countermeasure: Retard (earth, timber and steel sheet piles)
  - Scour Type: Local scour, Meandering stream or shifting of thalweg
  - Description: Permeable or impermeable structure parallel to banks, to reduce flow velocity and induce deposition.
  - Advantages: Suitable for maintaining channel alignment. Induce deposition.
  - Disadvantages: Minimum disadvantages since piles are buried below river bed.  
Expensive
  - Remarks: Recommended for high flood Velocities in NJ.
2. Countermeasure: Channel Improvements (channelization)
  - Scour Type: Contraction and local scour, Aggradation
  - Description: Channel modifications to increase flow capacity and sediment transport, including dredging, channel clearing.
  - Advantages: Suitable for aggradation or if upstream / down-stream of bridge is clogged.
  - Disadvantages: Minimum disadvantages
  - Remarks: Recommended. River encroachment permit requirements apply in New Jersey.
3. Countermeasure: Guide Banks / Guide walls
  - Scour Type: Local scour at abutments / channel braiding
  - Description: Straight or outward curving earth structure /fill to form embankments upstream to align flow through the bridge opening and reduce abutment scour.

- Advantages: Improves flow conditions, Moves point of local scour away from abutment, prevents erosion by eddy action.
  - Disadvantages: Minimum disadvantages. Expensive
  - Remarks: Recommended for wide rivers with high flood velocity, River encroachment and other permit requirements apply in NJ.
4. Countermeasure: Groyne (spur / dike / deflector)
- Scour Type: Local scour, Upstream Lateral erosion and degradation
  - Description: Impermeable or permeable structure, which projects into flow to alter flow direction, reduces velocity and induces deposition.
  - Advantages: Suitable for containment of over bank flow and for braided streams. Proven effective
  - Disadvantages: Does not prevent downstream lateral erosion of banks or degradation of channel. They project above riverbed. Not applicable to streams or narrow channels.
  - Remarks: Recommended for wide rivers with high flood velocities. River encroachment permit requirements apply in NJ.
5. Countermeasure: Grade Control Structure /Check dams
- Scour Type: Contraction and local scour, Degradation Aggradation and lateral erosion of banks
  - Description: 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
  - Advantages: Suitable for high flood velocities.
  - Disadvantages: Expensive to install since riprap is required downstream of grade control structure.
  - Remarks: Difficult to meet environmental requirements in NJ Since fish passage is adversely affected
6. Countermeasure: Collars
- Scour Type: Local scour
  - Description: Thin horizontal plate attached to base of pier to deflect flow away from sediment bed
  - Advantages: Suitable for high velocity rivers & for long span bridges. Low cost and maintenance
  - Disadvantages: Debris accumulation for small spans. Does not eliminate scour, not much experience
  - Remarks: Not easy to construct. River encroachment permit requirements apply.

## 7. Countermeasure: Relief Bridge

- Scour Type: Local scour
- Description: Constructing an additional or auxiliary bridge adjacent to the scour critical bridge to minimize the discharge and flood velocity
- Advantages: Suitable for wide rivers and overcomes the problems associated with defects in the original planning and size of opening.
- Disadvantages: May cost nearly the same as a replacement bridge.  
Environmentally, it may create additional problems. It may be difficult to acquire the right of way in developed areas.
- Remarks: Fewer applications in NJ due to limited widths of the streams.  
Additional river encroachment permit requirements may be applicable.

### Description of Selected Types of River Training Measures

**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 and channel cleaning. 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.

**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. Figure 12.5 shows layout and configurations of guide banks.

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. 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 will be the same as those for abutments.



Alternatively, stone sizes for riprap can be selected on the basis of following equation,

$$d_r = 0.0282V^2 \quad \text{for } 1V : 2H \text{ side slope}$$

$$d_r = 0.0216V^2 \quad \text{for } 1V : 3H \text{ side slope}$$

$$d_r = 0.0418V^2 \quad \text{for launching apron}$$

where  $V$  is the mean velocity of the approach flow.

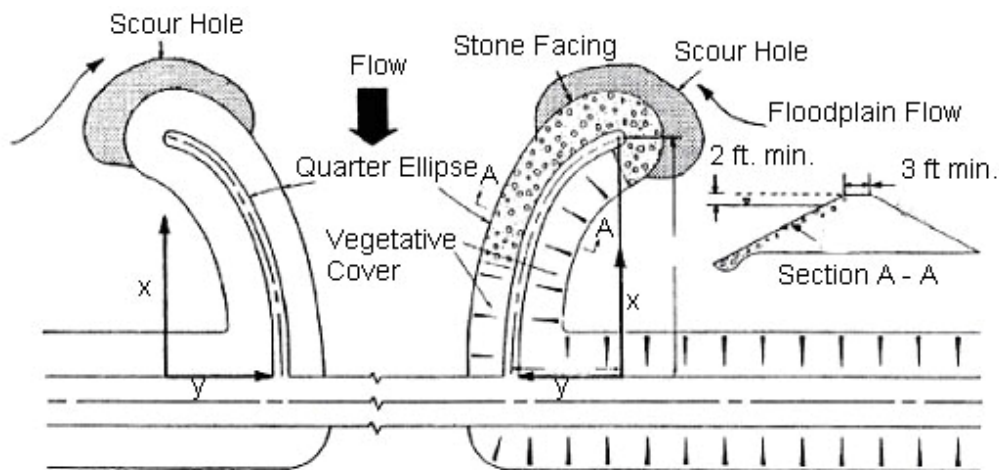


Figure 12.5: Layout and Configuration of Guide Banks.

**Guide Walls:** Guide walls are similar to guide banks. Typical examples of Guide Walls are shown in Figures 12.6.

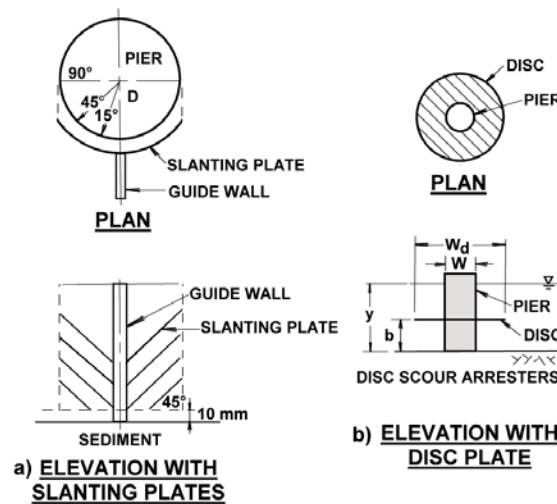


Figure 12.6: Plan & Elevation of (a) Guide Wall with Slanting Plate, (b) Disc Scour Arrestors.

## **CHAPTER 13**

### **FOUNDATION MODIFICATION AND STRUCTURAL COUNTERMEASURES**

#### **Introduction**

Structural countermeasures are site specific and there is no single technique, which would apply to different problems at the same time. They involve minor to major repairs and modifications to 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 can involve foundation-strengthening, conversion from a simple span to continuous span configuration or pier geometry modifications. Special provisions for technical specifications based on recommended guidelines may be required and should be used for preparing construction drawings and contract documents. Structural countermeasures are more effective and have direct application than armoring. With advancements in structural applications, more sophisticated structural countermeasures are becoming available for adaptation by structural engineers.

1. Foundation strengthening includes supplementing the original foundation. This involves use of additional reinforcement and/or increasing the size of the original foundation. 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 the use of armoring or no armoring. This is generally used when scour is of a high magnitude. However, structural repairs such as grouting holes and cracks are generally required. Most commonly used for existing bridges are:

1. Foundation Shielding such as constructing concrete apron/curtain walls
2. Sheet Piling local to foundations to act as shielding (sheeting left in place after construction is completed)
3. Soldier piles and precast panels
4. Mini piles driven through spread footings
5. Underpinning under the footing

6. Extended Footings
7. Use of open parapet or railings to permit deck drainage of flood water.
8. Use of integral abutments

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.

### **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 and soldier piles
3. Extended Footings
4. Underpinning Methods such as driving mini piles
5. Open parapets or railings
6. Use of integral abutments

The final selection must consider environmental considerations and cost. Figure 13.1 shows a flowchart for substructure repairs prior to installation. Illustrations of different structural countermeasures are shown in Figures 13.2 to 13.17.

### **Evaluating (Scoured) Unsupported Pile Lengths**

Scour around piles leads to exposed pile lengths. Picture in Figure 13.18 shows exposed pile bents at Peckman’s River Bridge on Route 46 in New Jersey. This reduces axial capacity. The higher the scour, the lower is the axial capacity. After a peak flood has exposed pile lengths, it is necessary to compute the axial capacity, a reduced factor of safety and the safety of the foundation. A pile program such as L-Pile may be used.

### **Use of Modern Underpinning Methods**

A footing settlement is likely to occur due to erosion of soil during a flood. A detailed subsurface information and an understanding of critical ground behavior are essential. The following methods can be used:

1. **Conventional pit method** consists of enlarging or deepening existing foundations by selectively removing soil from beneath the foundations and replacing it with concrete, soil reinforcement or grout. The superstructure needs to be shored to prevent settlement. The disadvantage is that the superstructure deformation cannot be avoided.

2. **Soil treatment** changes the physical properties of the ground to make it more supportive and stronger. Ground treatment will strengthen the ground, which will also act as a load transfer mechanism.
3. **Minipiles or micropiles** (as defined by FHWA) are 2 to 8 inches diameter steel piles. They transfer the load by relieving existing footings, which would function only as pile cap. This technology requires powerful drills applied from above the foundation.
4. **Jet grouting** serves as an excavation support and ground water control system. High velocity injection fluids erodes the soil and replaces the soil with an engineered grout, forming a cementitious product known as soilcrete, which has compression strength exceeding 1000- psi. The technique has been used to reconnect pile caps to decayed piles, for enlarging existing spread footings, and for creating deep footings in difficult access locations. Specialized equipment and experienced contractors are required.
5. **Compaction grouting** densifies and strengthens the loose soil by injecting a very viscous sand-cement grout. Very stiff, continuous grout columns can be formed directly beneath the structure. Specialized equipment and experienced contractors are required.
6. **Chemical grouting** is a soil treatment system, which virtually glues the soil together to increase its strength. An injection system is used to give soil strengths over 300 psi.

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	Minipiles driven through footings	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

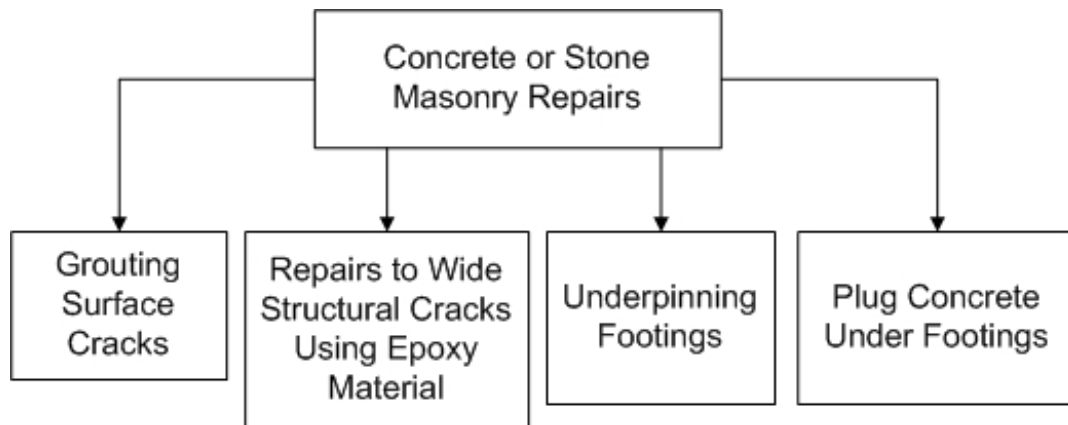


Figure 13.1: Substructure Repairs Prior to Installing Structural countermeasures.

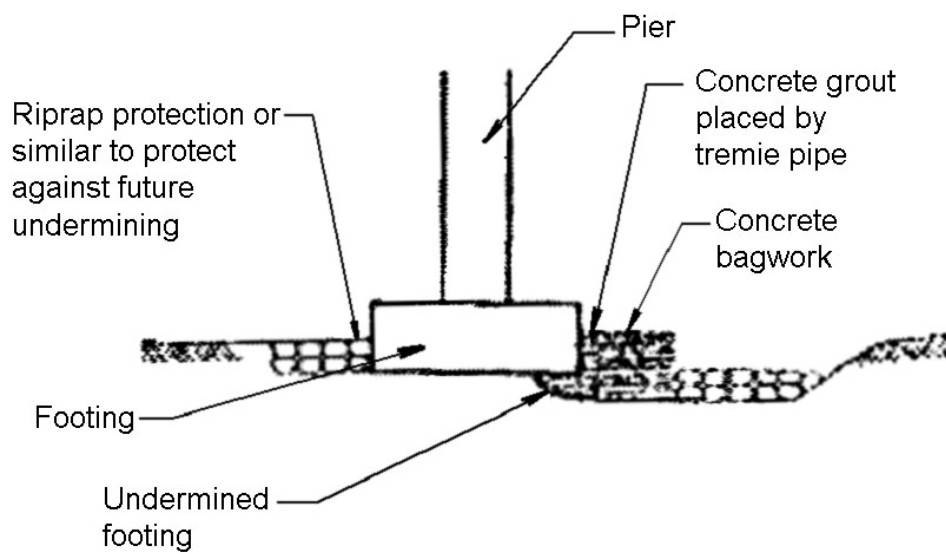


Figure 13.2: Structural Repairs – Grouting with Tremie Concrete (N.T.S.).

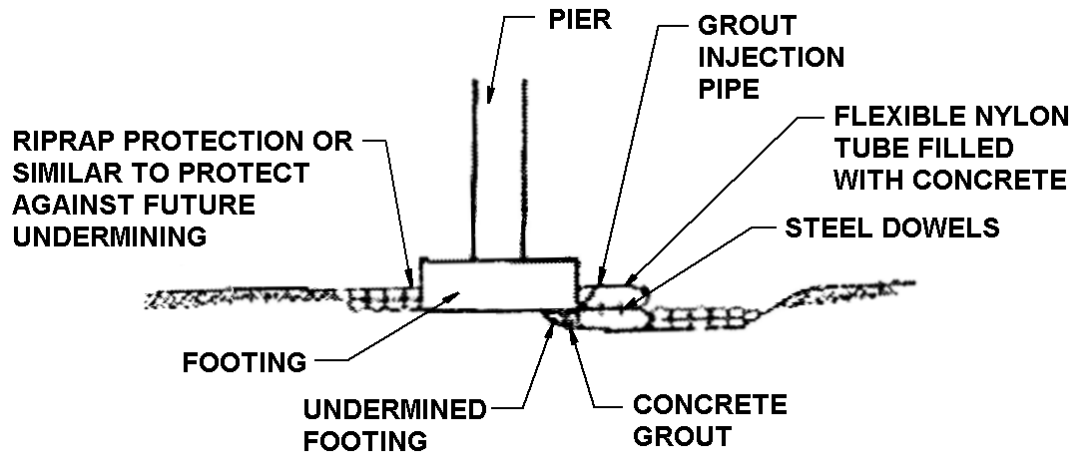


Figure 13.3: Structural Repairs – Grouting with pipe injection of concrete.

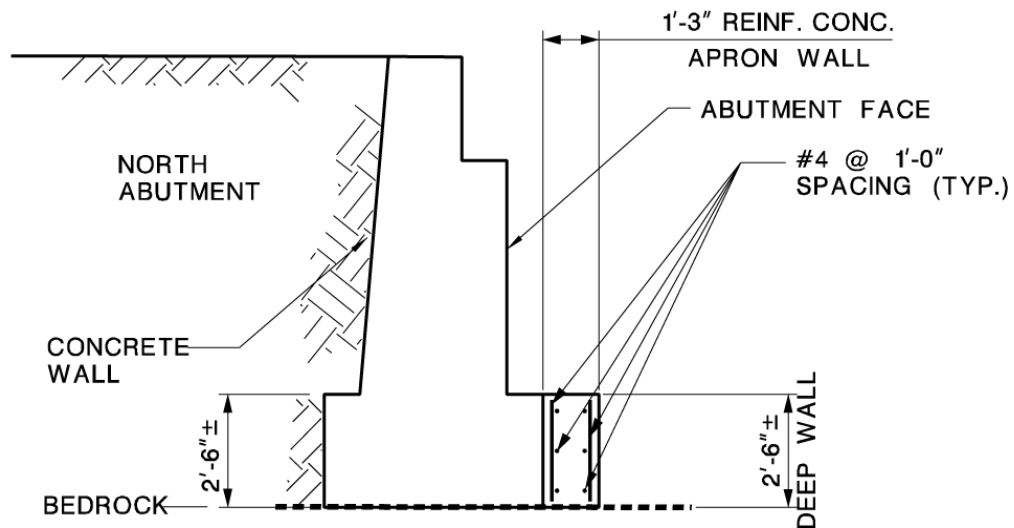


Figure 13.4: Concrete Apron Wall, Details at Abutment (N.T.S.).

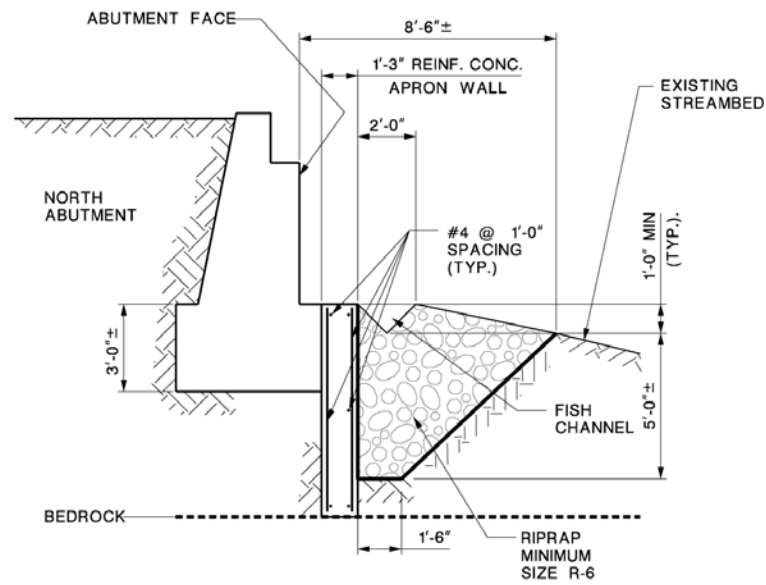


Figure 13.5: Concrete Apron Wall and Riprap, Details at Abutment (N.T.S.).

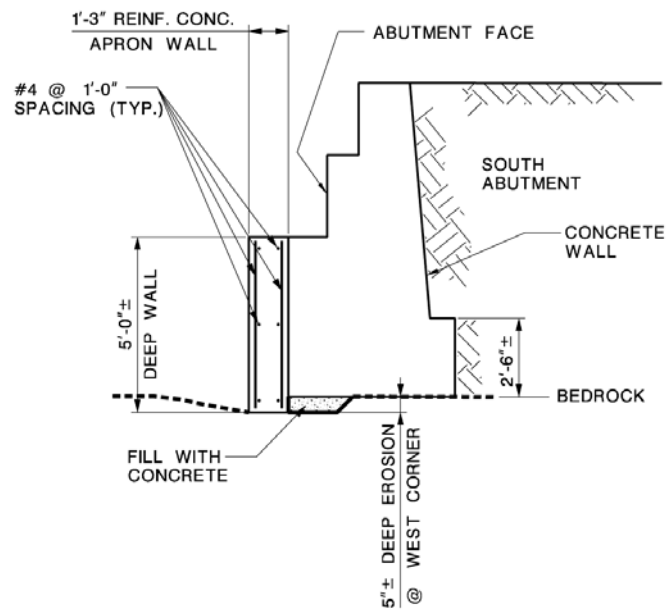


Figure 13.6: Concrete Apron Wall and Grouting of Scour Hole Details at Abutment (N.T.S.).



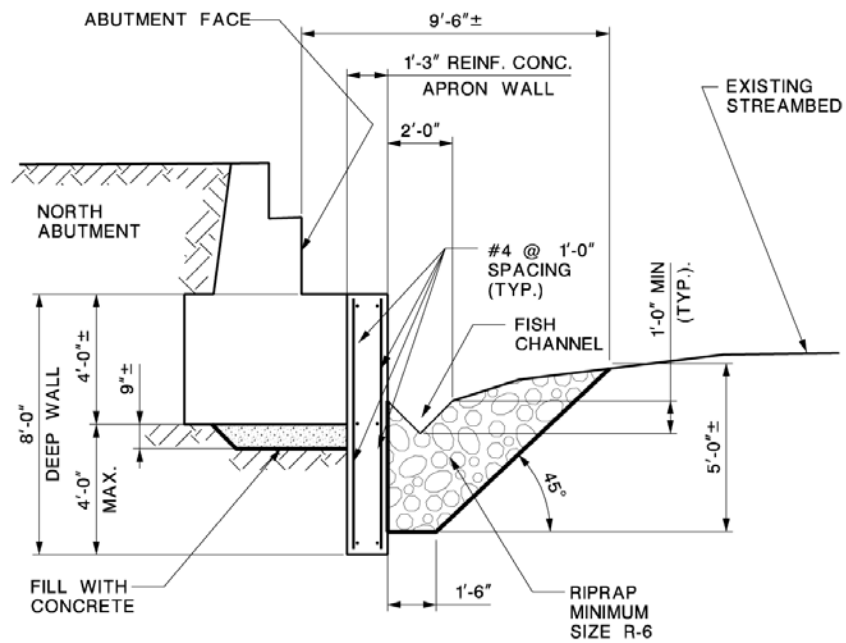


Figure 13.7: Concrete Apron Wall, Grouting Scour Holes with Armoring (N.T.S.).

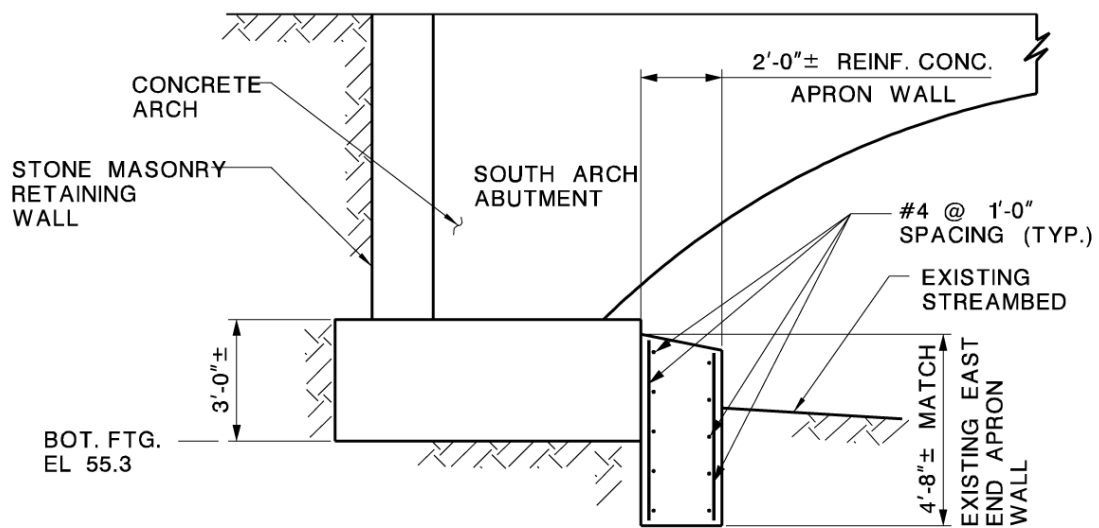


Figure 13.8: Concrete Apron Wall, Details at Abutment (N.T.S.).

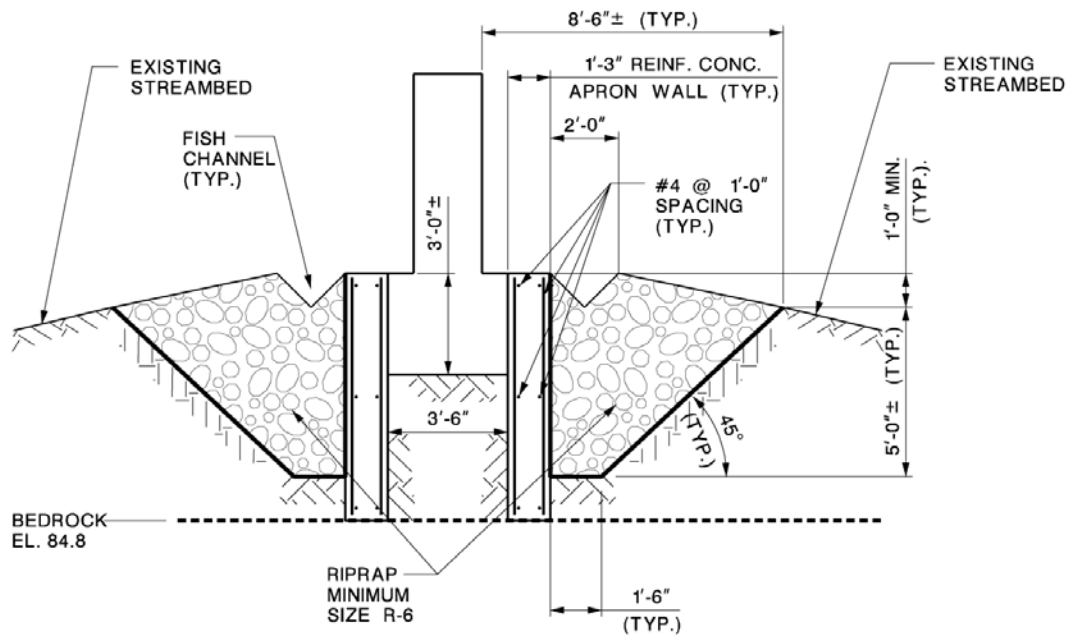


Figure 13.9: Concrete Apron Wall and Riprap, Details at Piers (N.T.S).

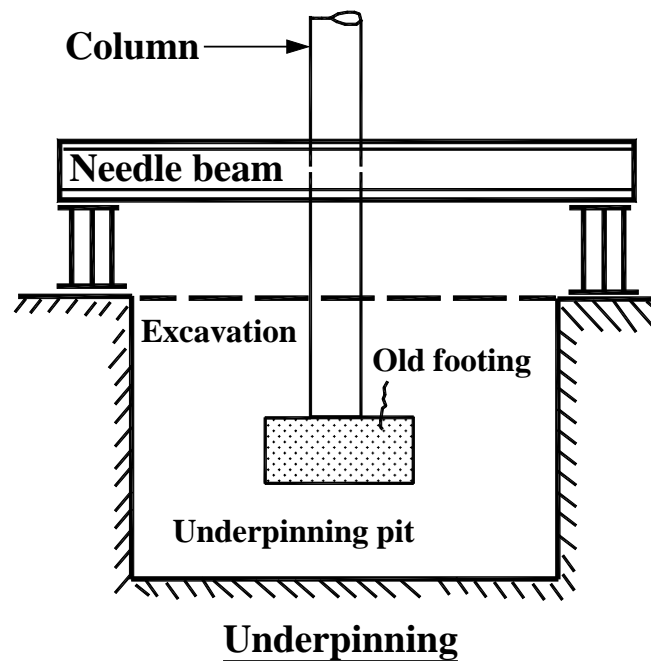


Figure 13.10: Conventional method of underpinning.

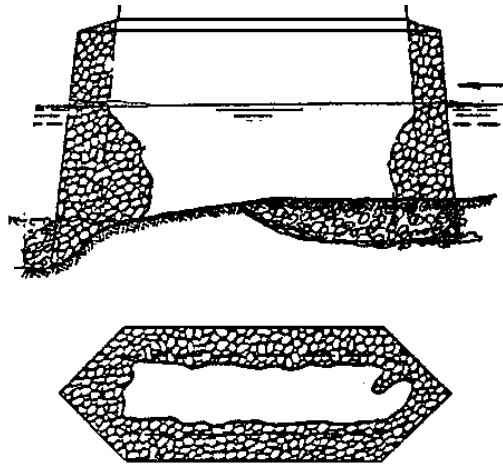


Figure 13.11: Jacketing the Foundation By Deepening and Jacketing with Concrete (N.T.S.).

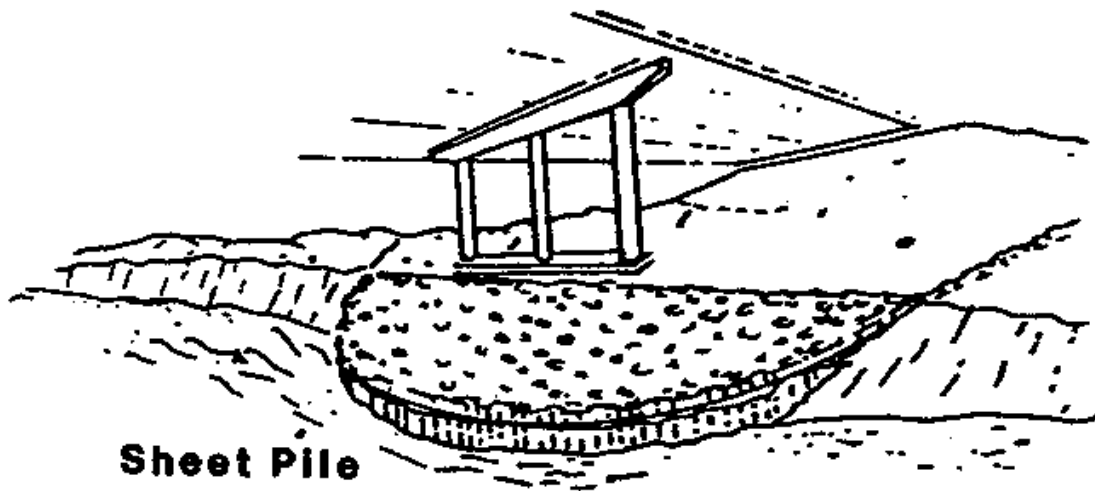


Figure 13.12: J Sheet-pile Cofferdam Enclosures and Cutoff Wall (N.T.S.)

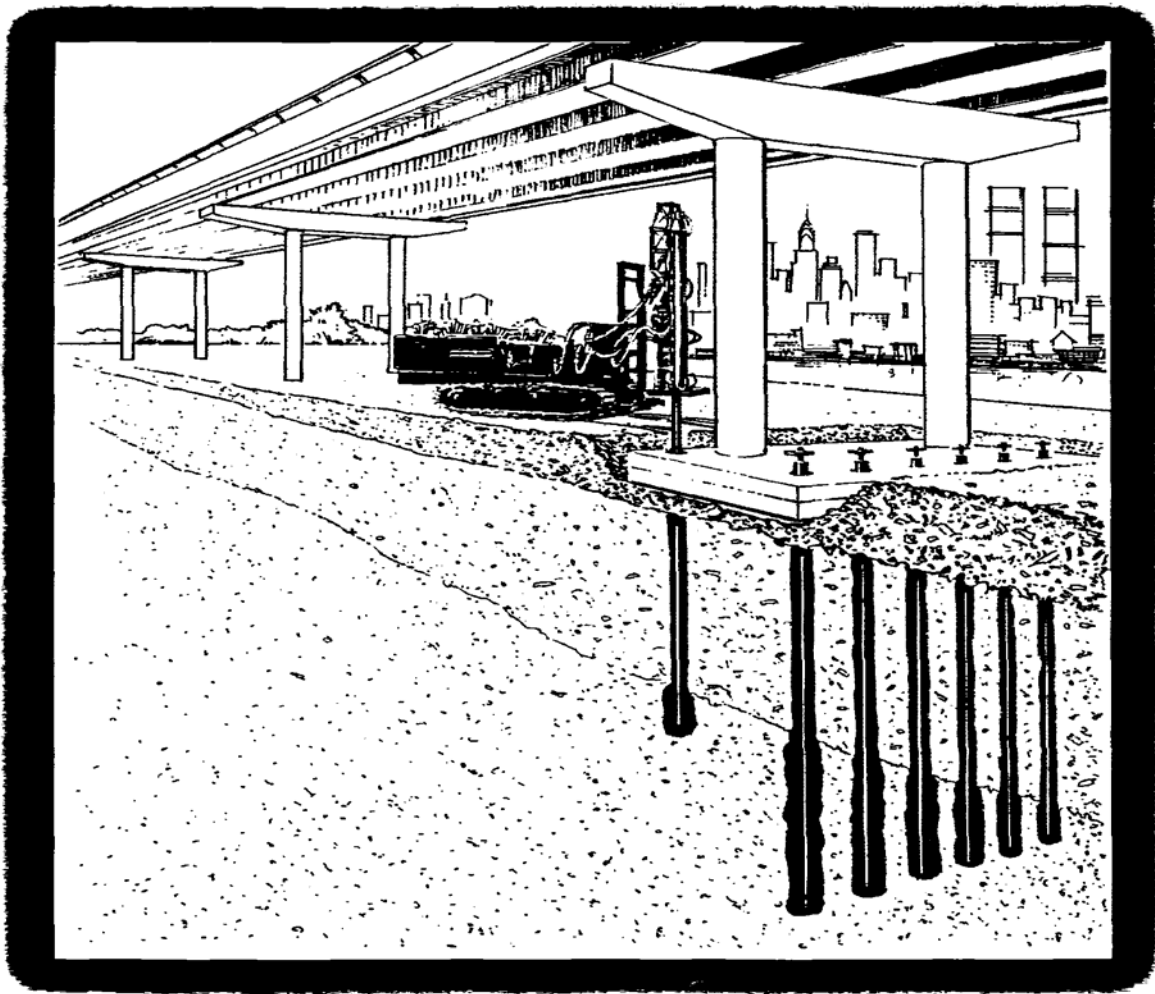
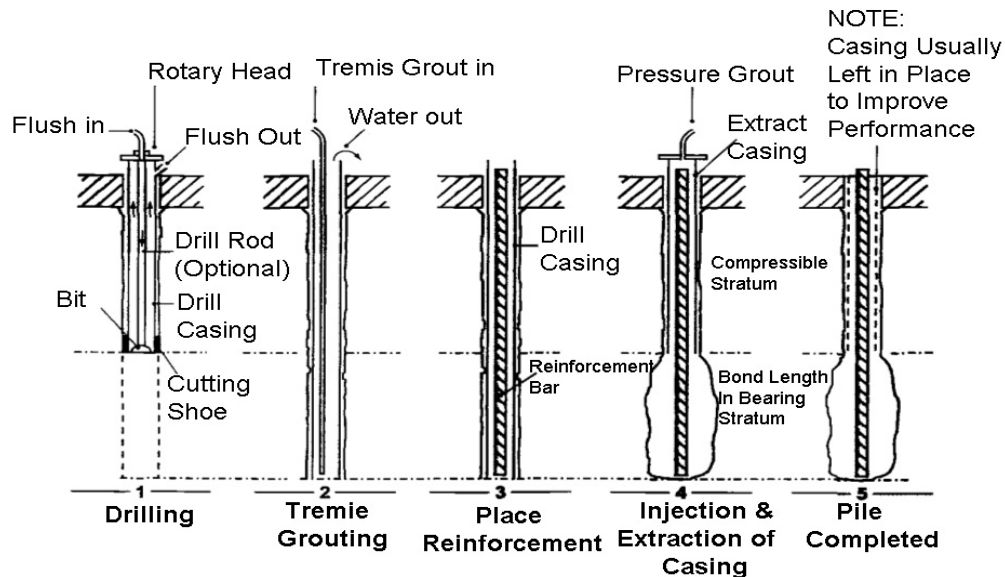


Figure 13.13: Restoration of Bridge Footings using Pinpiles.

## PIN PILES<sup>SM</sup>



Typical construction steps for PIN PILES<sup>SM</sup> in soil

### Common Uses of PIN PILES<sup>SM</sup>

- 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<sup>SM</sup>

- 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 13.14: Uses and Advantages of Pinpiles as Structural Countermeasure.

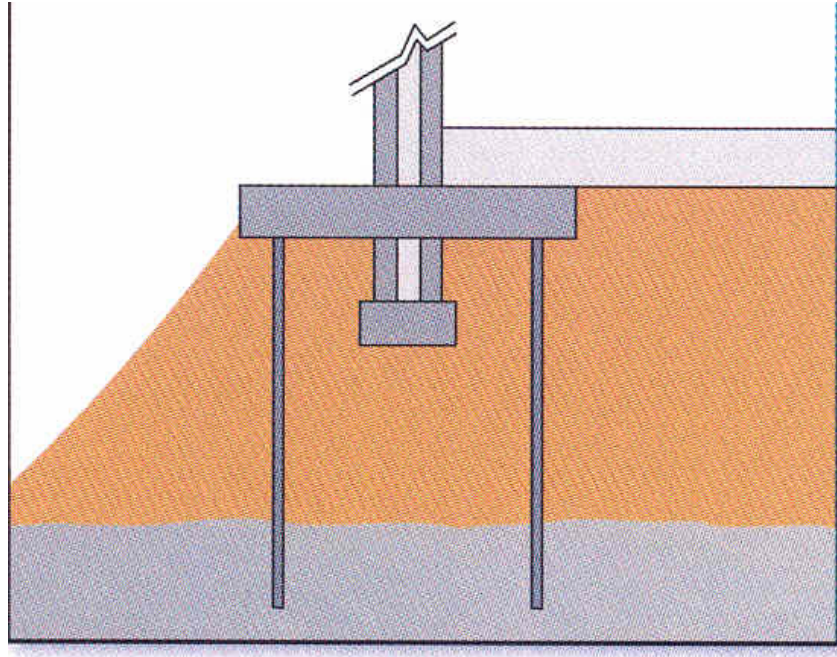


Figure 13.15: Underpinning method using mini piles and additional pile cap.

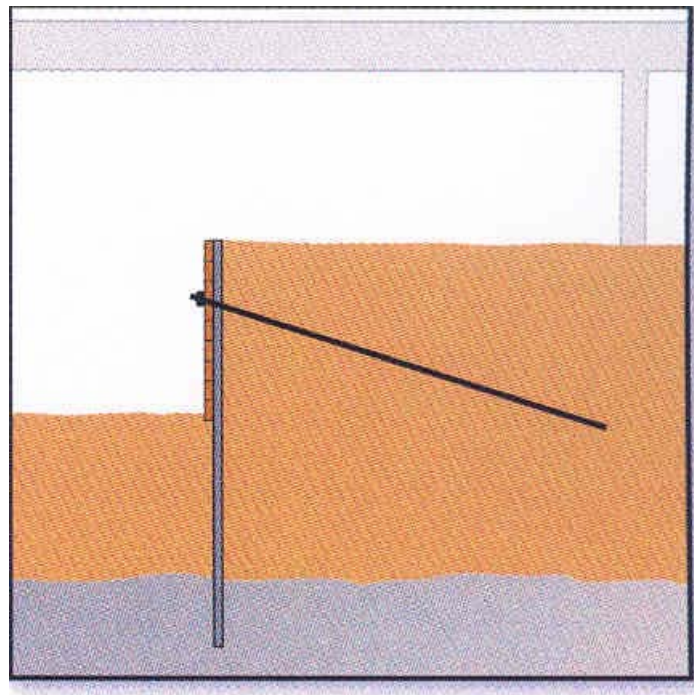


Figure 13.16: Timber Boards spanning across driven mini piles to stabilize soil against future erosion.



Figure 13.17: Exposed Pile Bents at Peckman's River Bridge on Route 46.

### Scour of Piles, Pile Groups and Caissons

Deep foundations such as piles and caissons are normally required in soft soils, where bearing capacity of soil is not sufficient to support a shallow footing. Such soils have greater scour potential.

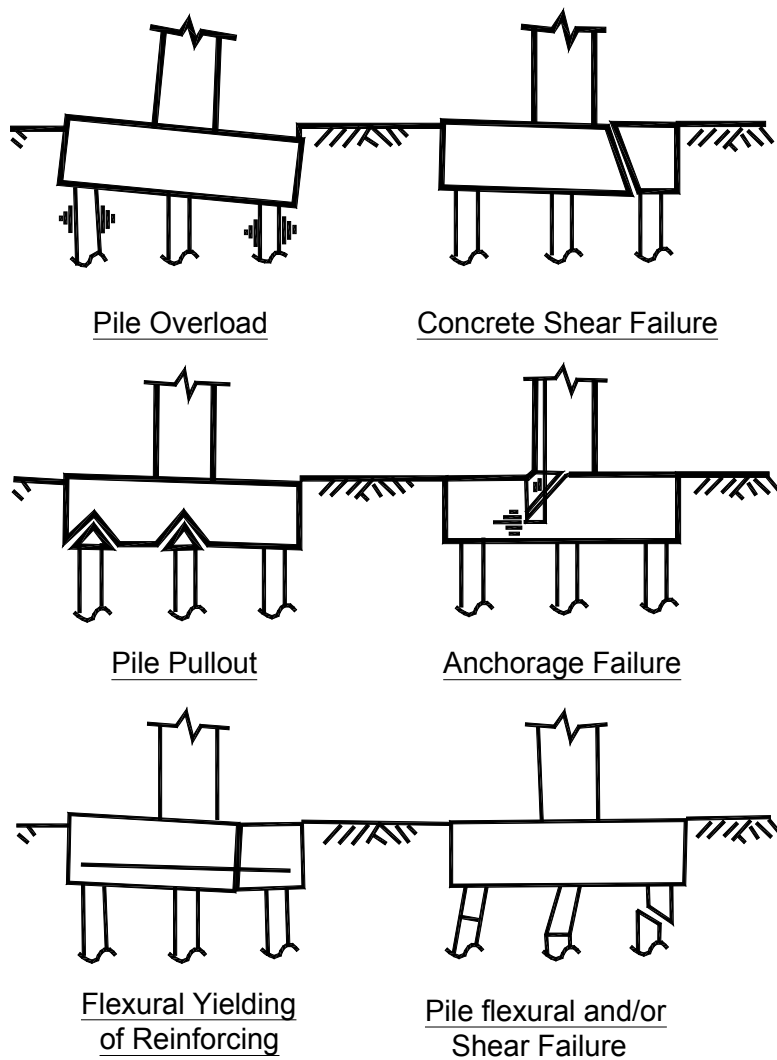
Piles are designed as bearing piles or friction piles and in either case, must be fully embedded in undisturbed soil at all times. If the surrounding soil is eroded or disturbed, the load bearing capacity of pile is reduced, leading to settlement and collapse of the pile cap. Figures 13.18 and 13.19 show possible modes of failure of piles resulting from soil erosion and sheet pile protection methods.

Raking piles, which are driven closer to the river, are more susceptible to erosion than vertical piles. The following steps to minimize the erosion of raking piles are required:

1. When using raking piles, they must not be placed at the edge of the river. Instead the span of bridge may be increased.
2. The top 5 to 10 feet of pile, which may be exposed, may be neglected in pile design length. The exposed length of steel or concrete pile should be checked for local buckling.
3. The pile cap should be placed below the calculated scour depth. However, this results in taller and full height abutments increasing the cost. Shielding of piles by driving sheet piling would prevent erosion.
4. Interference of raking piles with sheet piles is likely to occur. Allowing for sufficient clearance is possible but would reduce the width of waterway; making it difficult to

obtain a stream encroachment permit from DEP. Armoring or river training measures may be considered.

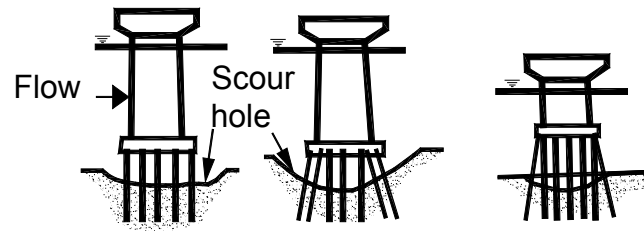
5. Caissons may be preferred over piles.
6. For long span bridges on harder ground and dense soils, drilled piers will be more resistant to erosion compared to caissons.
7. Integral abutment piles with single row of piles would require greater protection by providing shielding in the form of sheeting left in place.



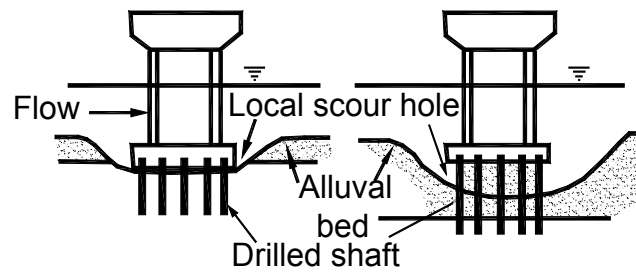
**Typical Pile Foundation failures**

Figure 13.18: Underpinning method using mini piles and additional pile cap.

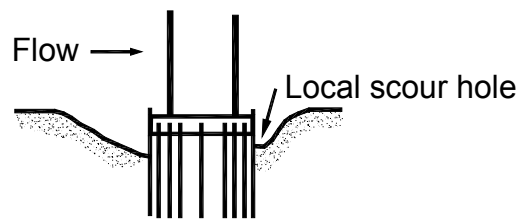




TYPICAL PILE DESIGN



DRILLED SHAFT FOR SCOUR



SHEET PILE PROTECTION

## Pile foundation

Figure 13.19: Scour at pile groups and protection.

## **CHAPTER 14**

### **ROCK RIPRAP AS TEMPORARY ARMORING**

#### **Description of Riprap**

A temporary countermeasure is used for emergency situations or as an interim countermeasure, until permanent countermeasures are installed. NJ experience with rock riprap, as a primary permanent armoring has not been very successful. For bridges and culverts, riprap is recommended as temporary/interim countermeasure until a permanent countermeasure is constructed. The use of riprap for river banks, retaining walls and wingwalls as primary countermeasure is still acceptable.

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. 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. 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.

#### **Modes of Failure**

In order to prevent the failure of riprap during floods, the following common causes of failure must be considered and controlled during the riprap design.

1. Inadequate rock size and gradation
2. Absence of filter layer
3. Internal slope failure
4. Riprap being placed at steep slopes
5. Poor toe design
6. Degradation of channel

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

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 and the presence of impermeable material that acts as a fault plane when subjected 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 curved 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 with the fault line on a horizontal plane occurs.
- 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)].

**Limitations on the Use of Riprap**

1. **Monitoring:** Riprap should 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 10 ft/sec., riprap should 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.

### **Traffic and Utilities Issues on the Use of Riprap**

1. **Site access:** Adequate access to the site should be provided for trucks to deliver riprap.
2. **Right of Way:** Construction easements and right of way access may be required for the duration of construction.
3. **Detours:** Detour, lane closure or night-time 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.

### **General Guidelines for Riprap**

NCHRP Report 568 entitled “Riprap Design Criteria, Recommended Specifications, and Quality Control” presents detailed guidelines on all aspects of riprap design and construction. The following general design guidelines apply to both piers and abutments for the design of riprap.

1. **Records of Foundations:** Records of construction 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 fieldstone or quarry stone should be used for riprap installations. The stone should be hard, angular or of approximately rectangular shape. The quality should be such that it doesn't disintegrate on exposure to water or weathering.
3. **Excavations:** The top layer of riprap should 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 as per design should be carried out in layers. The depth of riprap should be at least below the contraction scour depth. 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 should conform to OSHA standards.

4. **Equipment:** Excavation machinery, drilling equipment and small cranes for lifting heavy stones may be required.
5. **Maintenance:** Since riprap is buried, no maintenance is required. However, if riprap gets exposed or is scattered on riverbed except after heavy floods, remedial work should be planned. The following maintainability issues should be considered with riprap installations:
  - a. Under-bed installation increases durability.
  - b. More maintenance with 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
6. **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.
7. **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 based on HEC-18 procedures, should be prepared. Table 14.1 shows scour data at bridge abutments and piers required for the design of riprap. 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.

Table 14.1 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 Velocity
50 Year							
100 Year							
500 Year *							

If 500 year Discharge is not available, use  $Q_{500} = 1.2 \times Q_{100}$

The following flow parameters are required to calculate the size of rock,

- a. Flow velocity,  $V$
  - b. Velocity magnification factor,  $K$
  - c. Density of riprap
  - d. Water depth,  $d$
8. **Rock Size and Gradation:** The median size of riprap  $D_{50}$  shall be determined using guidelines in Chapter 4. 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 14.2.

Table 14.2: 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

9. **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 should be used unless otherwise recommended [Brown and Clyde (1989)].

Table 14.3 Rock riprap gradation

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

10. **Exceptions to depth of riprap:** Riprap may not have to be installed to the full scour depth in the following circumstances:
- 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.
  - 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. However, the size of stone 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.

**11. Durability:** The following durability issues should be considered with riprap installations:

- a. Broad band of failure threshold potential
- b. Catastrophic failure if riprap is exposed
- c. Abrupt failure of geotextiles.

**12. Effectiveness of riprap installations:** Effectiveness of riprap depends on

- a. The seal around piers
- b. Reduced tendency of rock dispersal
- c. Degradation of Granular filters
- d. More effective if tied into abutment countermeasure when pier is located within 3 pier diameters of abutment footings.

**13. Constructability of riprap installations:** Following constructability issues should be considered during riprap design.

- 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

**14. Costs:** Costs may vary according to local conditions. Current estimated cost is \$150 to \$200 per SF. Long distance freight charges for riprap may increase the unit cost by 10%. The following cost issues should be considered with riprap installations:

- a. Geotextile more expensive than granular filter
- b. Pre-excavation costs
- c. Disposal costs
- d. Less stone costs
- e. Traffic disruptions

**15. Riprap Detailing**

- 1. Construction drawings should 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 should show tables summarizing flood elevations, flood velocities and scour depths.
  3. Maximum side slope is 1V: 2H. Where excavation is difficult, 1V: 1H may be used with fractured rock.
16. **Construction Permits:** Stream encroachment and other applicable permits must be accounted for. Refer to Guidelines given for permit applications in the NJDEP Stream Encroachment Technical Manual.
17. **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.

### Design Guidelines for Riprap at Bridge Piers

The use of riprap blankets alone is recommended only for existing bridges that are due for replacement in a few years. Their use should be restricted to low and medium flood velocities not exceeding 10 feet per second. Following guidelines should be followed for the design of ripraps:

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  $2D_{50}$ .

$$D_{50} = \frac{0.692(KV)^2}{(S_s - 1)2g} \quad (14-1)$$

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)

$g = 32.2 \text{ ft/s}^2$

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

2. **Thickness of Mat:** The recommended thickness of a riprap mat is  $2D_{50}$ . The top of riprap should be flush with the bed at low flow. The minimum thickness should not be less than 12 inches.
3. **Layout of Riprap Mat:** Figures 14.1 and 14.2 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 at pier
  - b.  $2W$  ( $W$ =width of pier at base) or  $2W/\cos(\beta)$  when  $\beta > 15^\circ$  in Figure 14.2.
  - c.  $(1.5 \text{ foot} + y \cot \phi)$  from the face of the footing, where  $y$  is the design scour depth at any abutment or pier and  $\phi$  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 1.5 foot from the vertical face of the footing as shown in Figure 14.1.

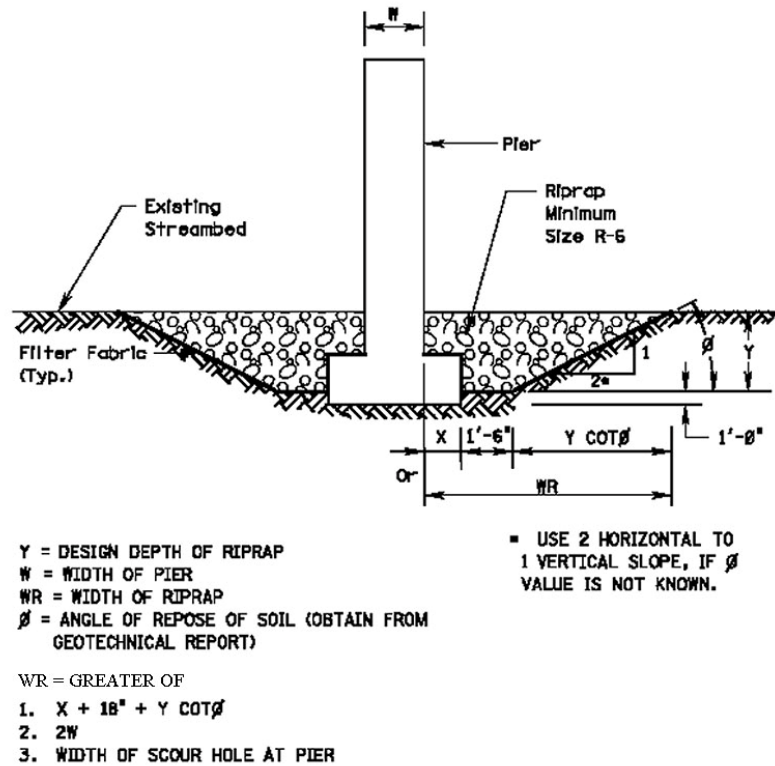


Figure 14.1 Pier Cross-Section Showing Riprap Details at Piers

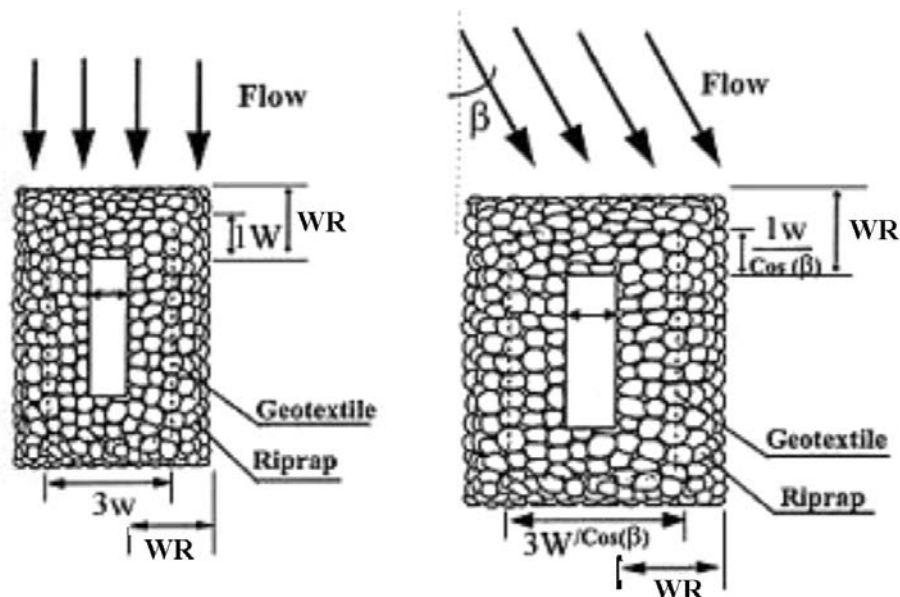


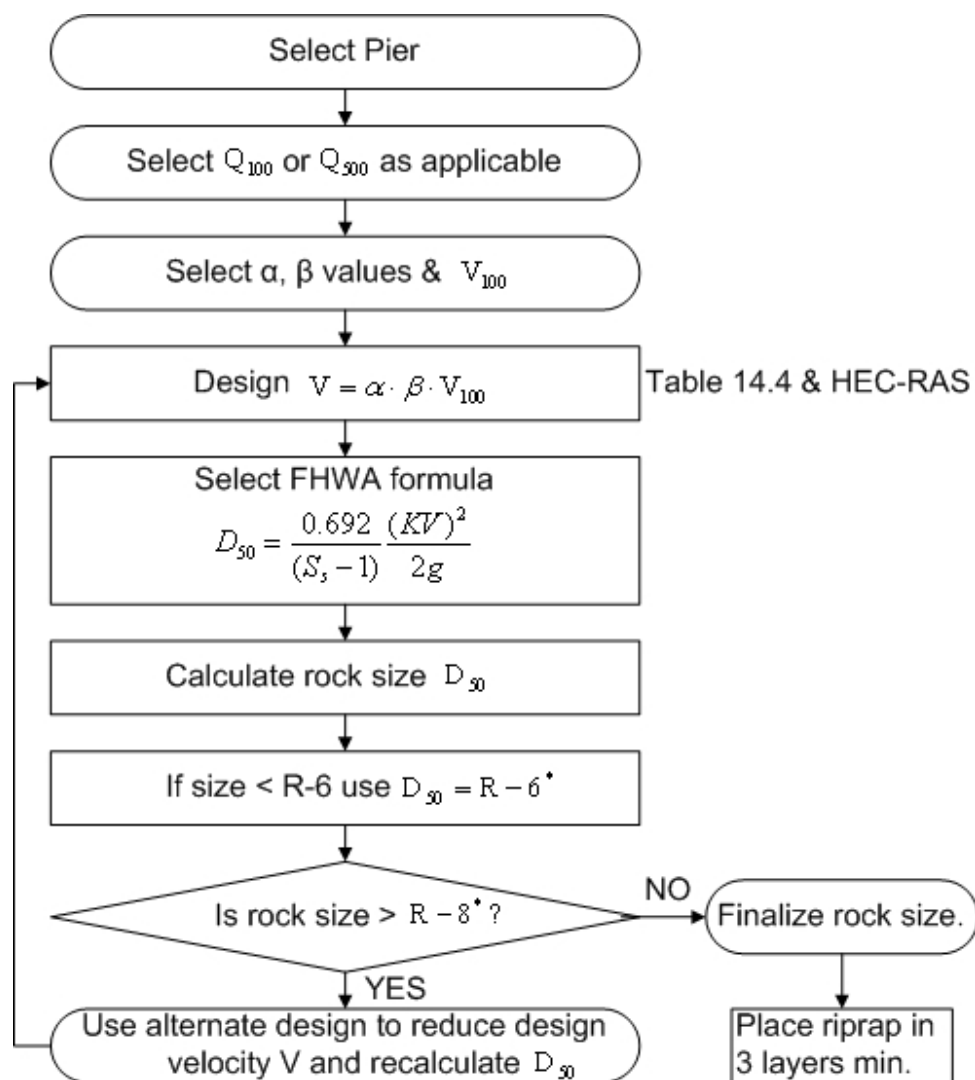
Figure 14.2 Extent of Riprap and Geotextile Filter at Piers

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 is 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 14.2.

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 10 of this Handbook.

5. **Gradation:** The distribution of riprap sizes as described under “General Guidelines for Riprap” should be used.
6. **Placement of Riprap:** Riprap around bridge piers can either be installed from a 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 can be done either by machine placing from a dragline or by buckets from bridge deck.
  - b. **Installation from Bed / Banks of Stream:** In this method, by hand placing and packing, a compact, mortar less 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.

Figure 14.3 presents a flowchart for the systematic design of riprap sizing based on the Equation 14-1 described above. Table 14.4 presents flow parameters required in the flowchart in Figure 14.3 for the calculation of riprap size at bridge piers.



\* NCSA Classification in Table 14.2

**NOTES:**

1. If rock exists at a depth lower than the design depth, place bottom of footing at 6 inches below rock surface.
2. Set design depth = ½ scour depth if only riprap countermeasure is used.

Figure 14.3 Flowchart for the design of riprap at bridge piers.

Table 14.4: Coefficients for evaluating riprap sizes at existing piers.

FLOOD EVENT	Velocity Multipliers				Shape Factor K	Shape Factor K
	Q100		Q500			
COEFFICIENTS	$\alpha$ Location factor	$\beta$ 100 year flood factor	$\alpha$ Pier location factor	$\beta$ 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

### Design Guidelines for Riprap at Bridge Abutments

Design guidelines for riprap at bridge abutments in this section are based on,

- “Stability of Rock Riprap for protection at the toe of abutments located at the floodplain” published in 1991
  - “Design of Riprap Revetment”, Hydraulic Engineering Circular -11.
  - Design Guidelines 8 and 12 of HEC-23.
1. **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).
  2. **Riprap sizing for abutments:** For New Jersey conditions, the following HEC-18 equation is recommended until better equations based on hydraulic tests are available,

$$\frac{D_{50}}{y_2} = \begin{cases} \frac{K_s}{(S_r - 1)} Fr_2^2 & Fr_2 \leq 0.8 \\ \frac{K_s}{(S_r - 1)} Fr_2^{0.14} & Fr_2 > 0.8 \end{cases} \quad (14-2)$$

Where:

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

$K_s$  = shape factor

= 0.89 for  $Fr_2 \leq 0.8$ , 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

### 3. 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  $2W/\cos(\beta)$  when  $\beta > 15^\circ$  ( $\beta$  is the same as defined in Figure 14.2).
- 3.)  $X+18''+y \cot \phi$  where  $X$  is the width of abutment footing,  $y$  is design scour depth at abutment and  $\phi$  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 1.5 foot from the vertical face of the footing. Figures 14.4 to 14.6 show a plan view of riprap layouts 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 width of the abutment toe and around the sides of the abutment to a point of tangency with the plane of the embankment slopes. The apron should extend from the toe of the abutment into the waterway a distance equal to twice the flow depth in the over bank area near the embankment, not exceeding 24.6 feet. Figure 14.7 shows the plan view of riprap apron for spill-through abutments.

Figures 14.8 to 14.10 show typical layouts of abutment riprap for abutments near a channel bank, stub abutment near the top of high channel bank and abutment near flood plain, as used by Maryland State Highway Administration. 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 14.5.

Figure 14.11 presents a flowchart for the systematic design of riprap sizing based on Equation 14-2 described above. Table 14.6 presents various parameters that are required in the flowchart of Figure 14.11 for riprap sizing.

Table 14.5: Riprap  $D_{50}$  Size and Blanket Thickness in Figures 14.8 to 14.10.

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

4. **Gradation:** The distribution of riprap sizes as described under “General Guidelines for Riprap” should be used.

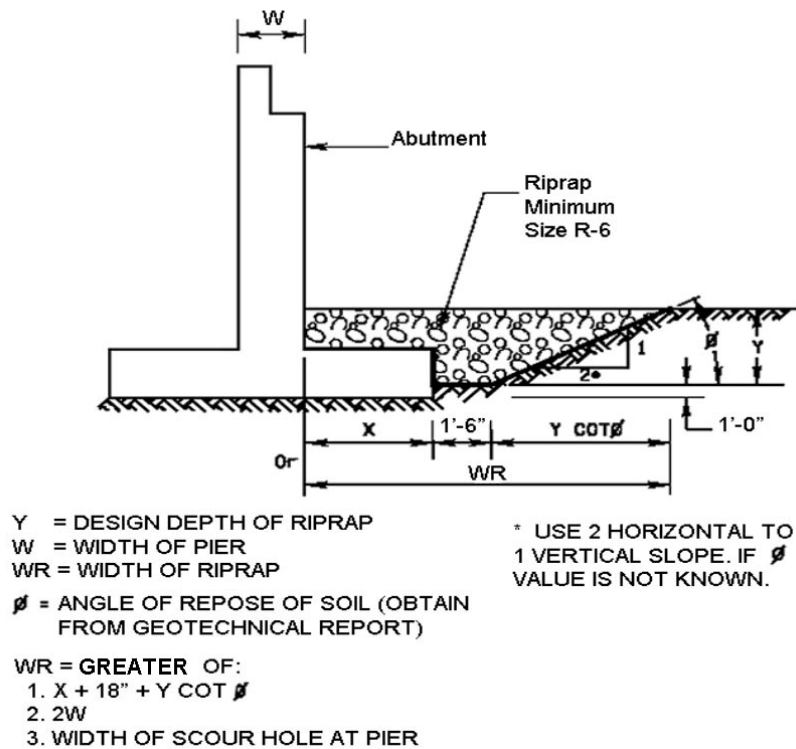


Figure 14.4: Riprap Details at Vertical Wall Abutments.

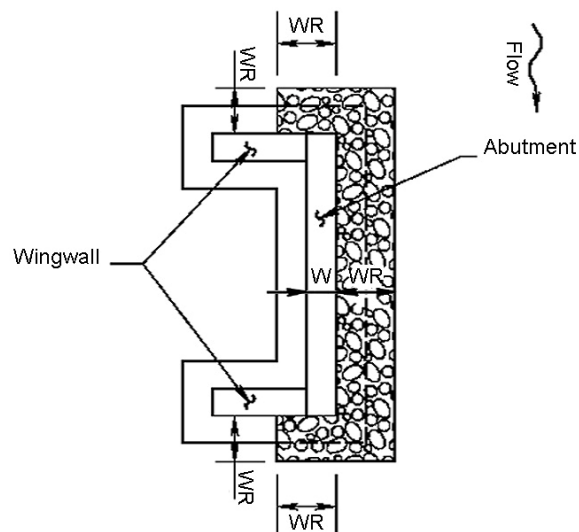


Figure 14.5: Plan of Riprap At Abutment - Wingwalls at 90 degrees.

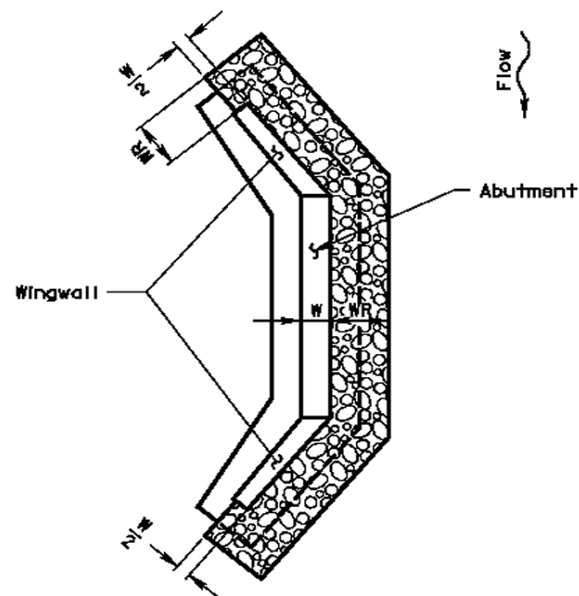


Figure 14.6: Plan of Riprap at Abutment - Wingwalls Splayed.

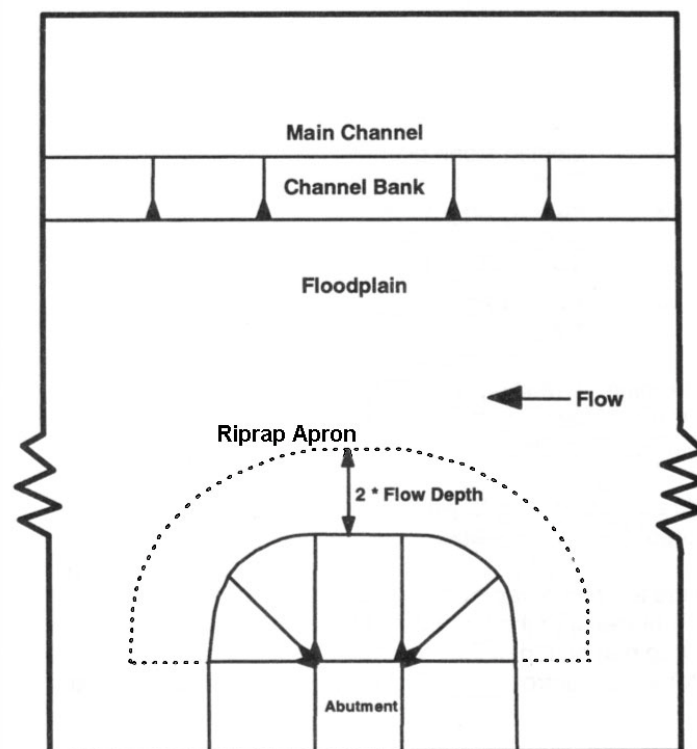


Figure 14.7 Plan view of the extent of rock riprap apron [HEC-23].

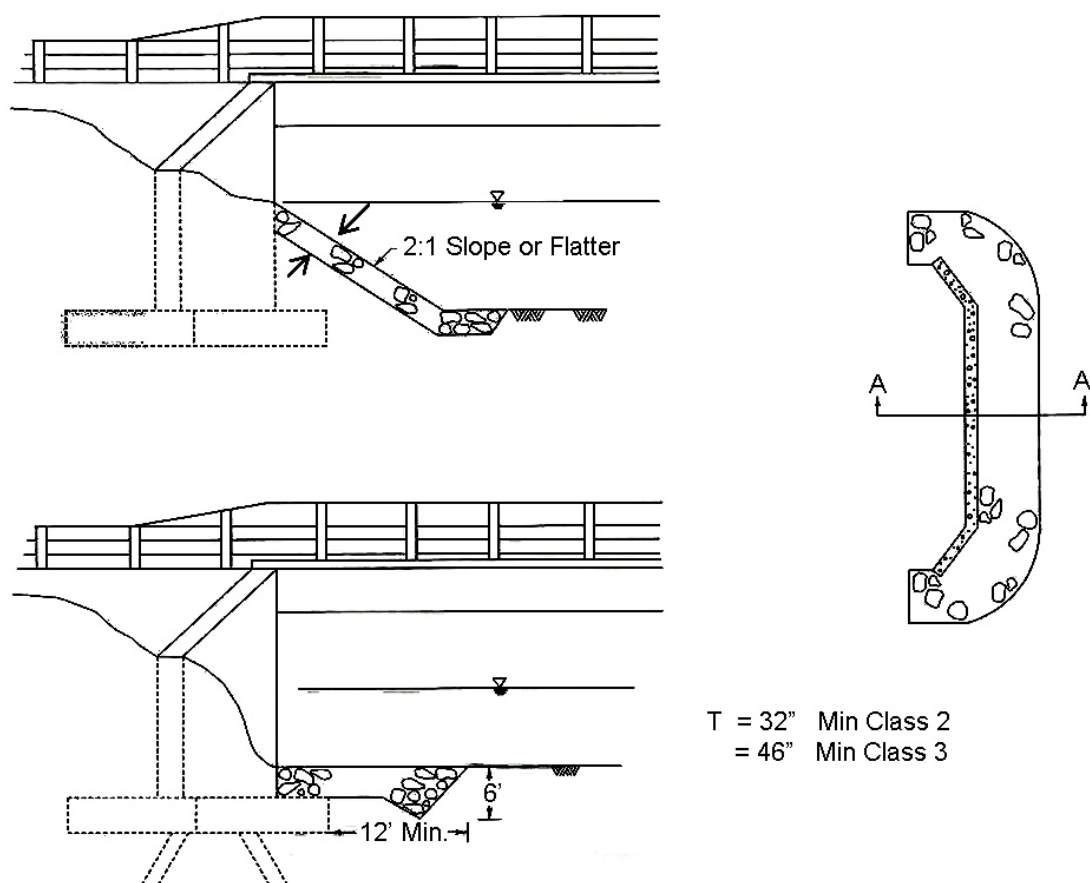


Figure 14.8 Plan & Typical Section A-A of Abutment near Channel Bank (Reference MDSHA, See Table 14.5 for  $D_{50}$  and Blanket Thickness)



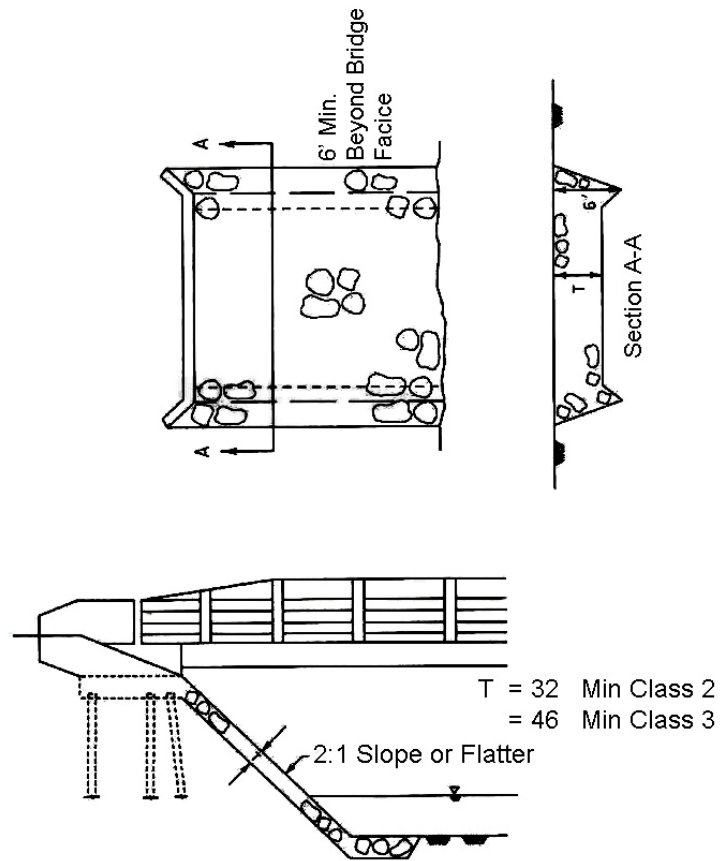


Figure 14.9 Plan & Typical Section of Stub Abutment near Top of High Channel Bank  
(Reference MDSHA, See Table 14.5 for  $D_{50}$  and Blanket Thickness)

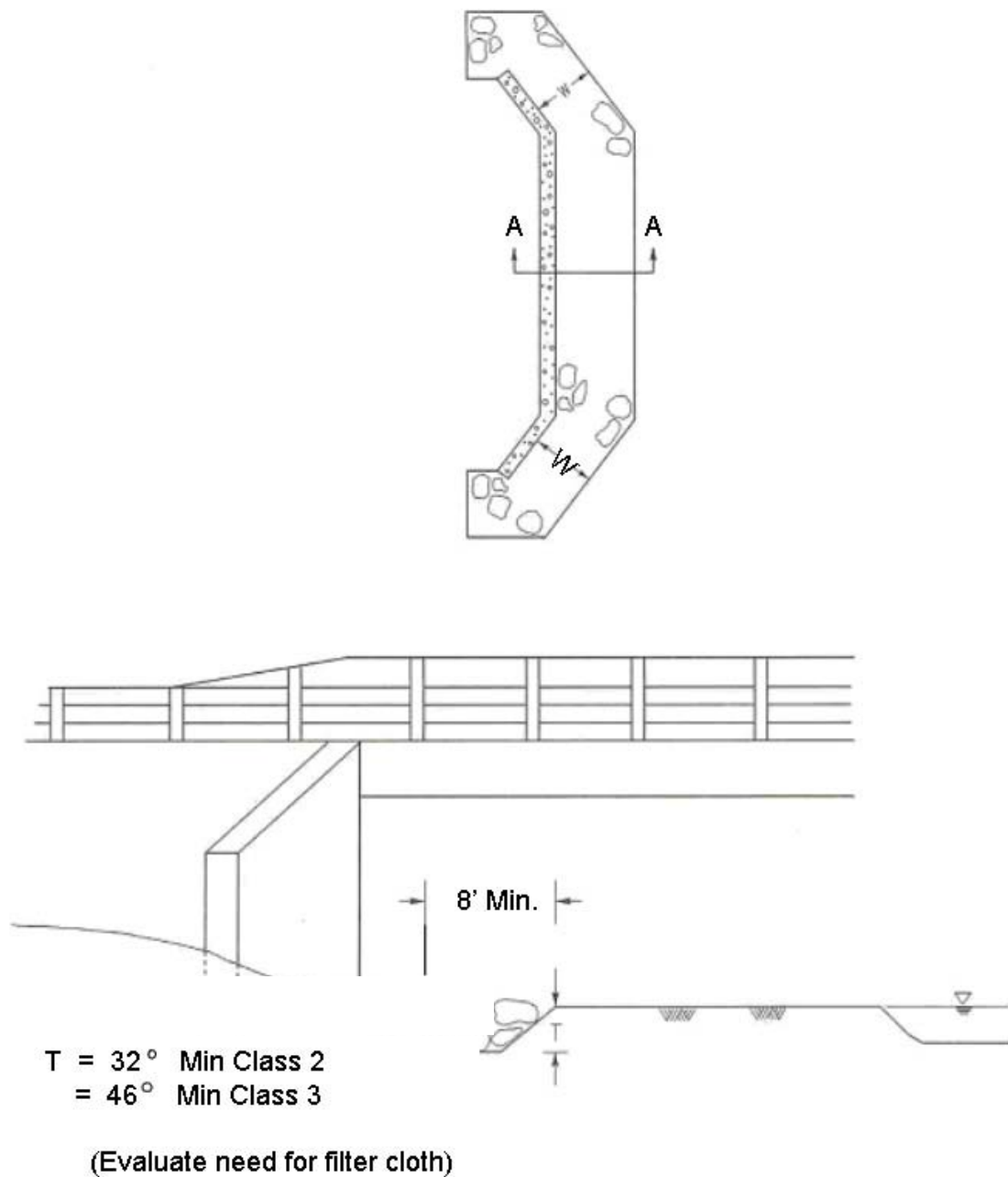
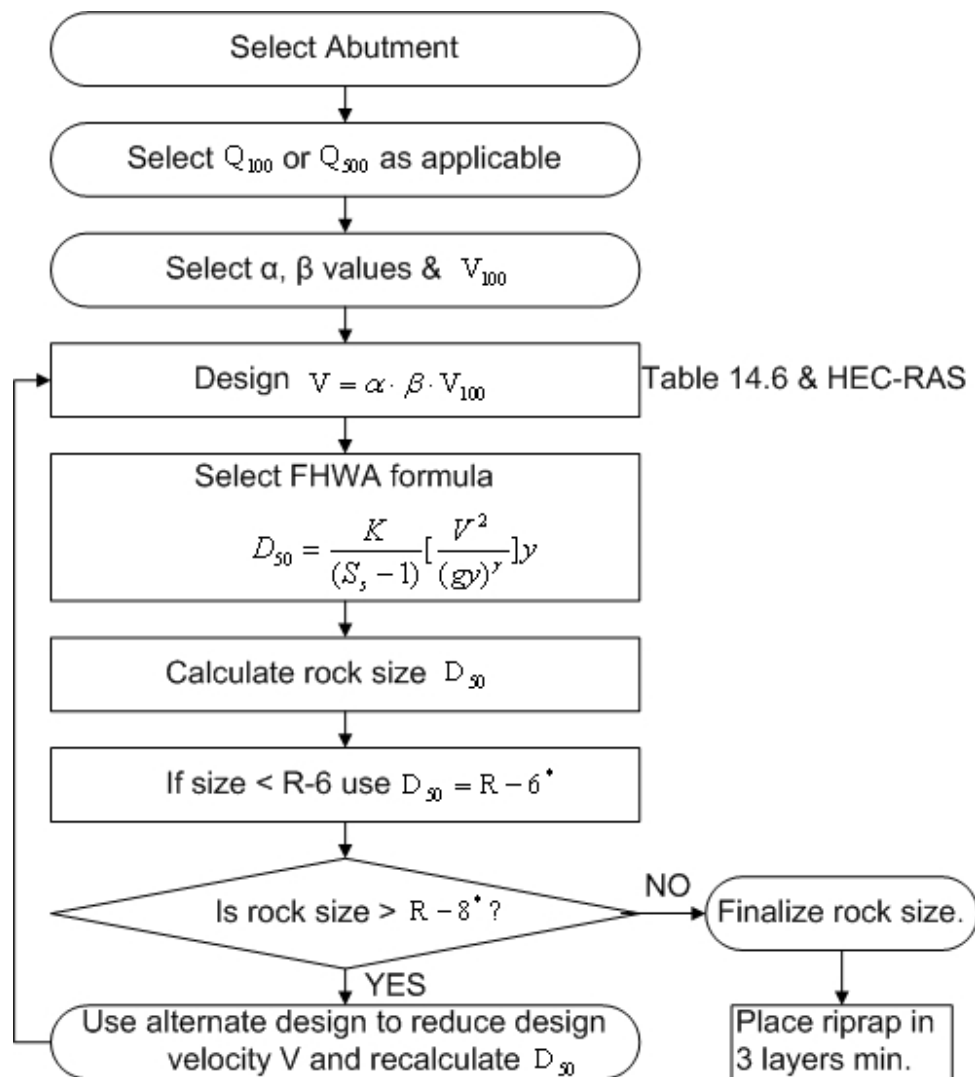


Figure 14.10: Plan & Typical Section A-A of Abutment in Flood Plain (Set Wall Back from Channel Bank with Low Flow Depths and Velocities for Worst Case Scour Conditions, Reference MDSHA, See Table 14.5 for  $D_{50}$  and Blanket Thickness).



\* NCSA Classification in Table 14.2

**NOTES:**

1. If rock exists at a depth lower than the design depth, place bottom of footing at 6 inches below rock surface.
2. Set design depth = ½ scour depth if only riprap countermeasure is used.

Figure 14.11 Flowchart for the design of riprap at bridge abutments.

Table 14.6: 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							
$\alpha$ Location factor	$\beta$ 100 year flood factor	$\alpha$ Pier location factor	$\beta$ 500 year flood factor	Spill- Through	Vertical -Wall	$\gamma$	Spill- Through	Vertical -Wall	$\gamma$
1.0	1.0	1.0	1.2	0.89	1.02	1.0	0.61	0.69	0.14

### Scaling Down Sizes of Armoring

With the view of obtaining an economical solution, certain scaling factors may be used 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 provides 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 may be scaled down by 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 may become 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 anchorage of armoring (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 river environment. For such situations, 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  $\zeta$  for application to the geology of soil

Weak soil	0.90
Sandy	0.85
Weathering rock	0.80

2. Scaling Factor  $\eta$  for application to width of the bridge opening,
 

Small width < 30 feet	0.90
Medium width > 30 but $\leq$ 50 ft	0.85
Large width > 50 but $\leq$ 70 ft	0.80
3. Scaling Factor  $\xi$  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  $\chi$ , for application to the remaining bridge life assessed,
 

For bridge replacement < 10 years	0.80
>10 but $\leq$ 15 years	0.85
> 15 years	0.90
5. Scaling Factor  $\psi$  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

$$\text{Scaling Factor (SF)} = \zeta \eta \xi \chi \psi \quad (14-3)$$

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

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

Example on the Use of Scaling Factors:

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 (14-3) = 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.

## Design Example for Riprap

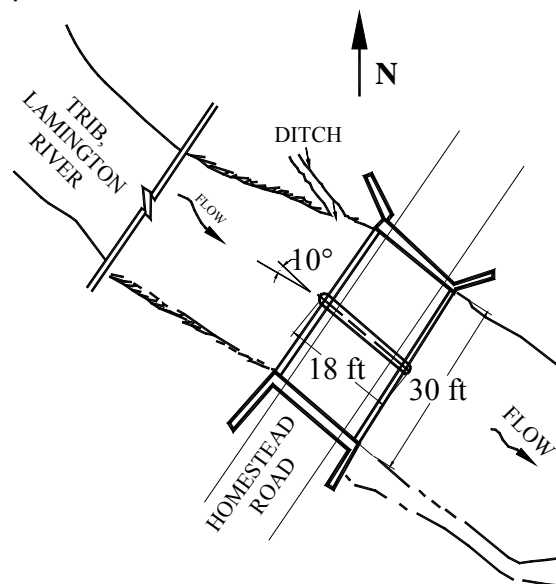
A typical New Jersey bridge is to have riprap installed. A scour critical bridge has a single round nosed pier and vertical wall abutments. Layout of the bridge site is shown in Figure 14.12. Elevations of pier and abutment are shown in Figure 14.13. Hydraulic, foundation and scour data for the bridge are as follows:

### Pier

Design scour depth at pier	= 5 ft
Width of pier footing	= 7'6" ft
Length of Pier (Width of Bridge)	= 18 ft
Width of pier	= 3'-6"
V100	= 5.08 ft/sec
V500	= 7.03 ft/sec
Specific Gravity of stone, Ss	= 2.24 for 500yr

### Abutment

Flow Depth, $y_0$	= 6.12 ft
V100 at abutment	= 5.54 ft/sec
V500 at abutment	= 7.64 ft/sec
Specific Gravity of Stone, Ss	= 2.24
Width of abutment footing	= 10 ft
Width of abutment	= 8 ft
Design scour depth at abutment	= 5'-6"



PLAN (NOT TO SCALE)

Figure 14.12: Plan

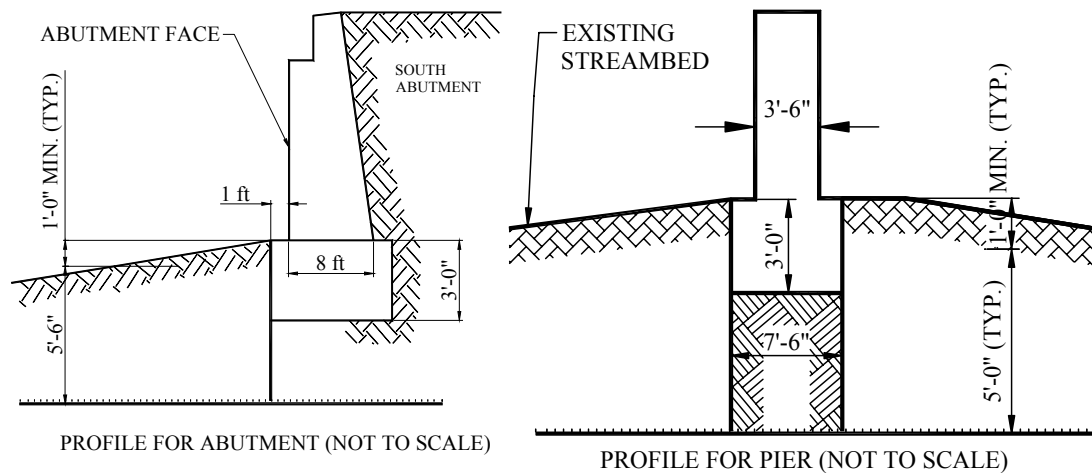


Figure 14.13: Elevations of Pier and Abutment Footings

### Riprap Design for Bridge Piers

#### 1. Design Velocity

$$V = \alpha \cdot \beta \cdot V_{100} = 1.2 \times 1.0 \times 5.08 = 6.096 \text{ ft/sec}$$

( $\alpha = 1.2$ ,  $\beta = 1.0$  from Table 14.4)

#### 2. Stone size as per Equation (14-1).

$K=1.5$  (For round nosed pier from Table 14.4)

$$D_{50} = \frac{0.692}{(S_s - 1)} \frac{(KV)^2}{2g} = \frac{0.692}{(2.24 - 1)} \left[ \frac{(1.5 \times 6.096)^2}{(32.2 \times 2)} \right] = 0.73' \approx 9"$$

Minimum  $D_{50} = 9"$

Place in minimum 3 layers.

Thickness of riprap mattress = 5 ft

(Contraction scour depth)  $> 2D_{50} = 18"$ , Hence thickness is sufficient.

From NCSA classification in Table 14.2, NCSA stone size = R-5, Use minimum R-6.

#### 3. Extent of Riprap Protection Design = 8'-6"

As per Figure 14.1,  $\Phi = 45^\circ$ . Hence, extent of riprap is Greater of:

$$\begin{aligned} WR &= X + 18" + Y \cot \Phi = 2' + 18" + 5' &= 8'-6" \\ 2W &= 2(3.5') &= 7' \end{aligned}$$

Figure 14.14 shows the design of riprap for bridge pier.

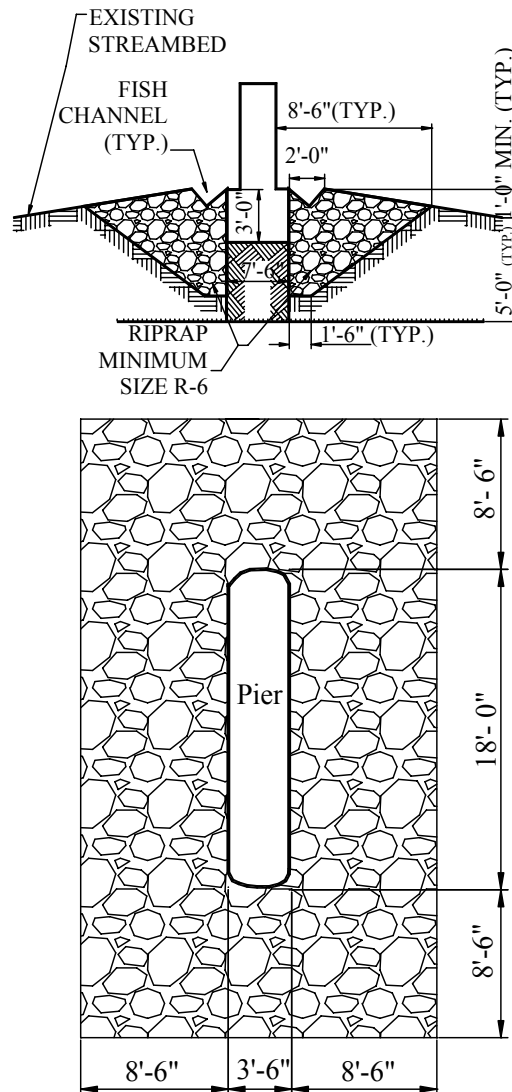


Figure 14.14: Countermeasure Details at Pier.

### Riprap Design for Bridge Abutment

#### 1. Design Velocity

$$V = \alpha \cdot \beta \cdot V_{100} = 1.0 \times 1.0 \times 5.54 = 5.54 \text{ ft/sec}$$

( $\alpha = 1.0$ ,  $\beta = 1.0$  from Table 14.6)

#### 2. Stone size as per Equation (14-2).

$$\text{Froude No.} = \frac{V}{(gy)^{0.5}} = \frac{5.54}{(32.2 \times 6.12)^{0.5}} = 0.39$$



Since  $0.39 < 0.8$  Select  $\gamma = 1.0$  as per Table 14.6

$K=1.02$  (For Vertical wall from Table 14.6)

$$D_{50} = \frac{K}{(S_s - 1)} \left[ \frac{V^2}{gy} \right] y = \frac{1.02}{(2.24 - 1)} \left[ \frac{(5.54)^2}{(32.2 \times 6.12)} \right] (6.12) = 0.78' = 9.41''$$

Minimum  $D_{50} = 0.78$  ft = Approx. 10 inch;

Thickness of riprap mattress = 5 ft

(Contraction scour depth)  $> 2D_{50} = 20''$ .

From NCSA classification in Table 14.2, NCSA stone size = R-5, Use minimum R-6.

### 3. Extent of Riprap Protection Design = 8'

As per Figure 17.4, choose maximum  $\Phi = 45^\circ$ ,

$$X + 18'' + Y \cot \Phi = 1' + 18'' + 5'-6'' = 8'-0''$$

Figure 14.15 shows the design of riprap for bridge pier. Place along the face of apron wall at abutments and wingwalls.

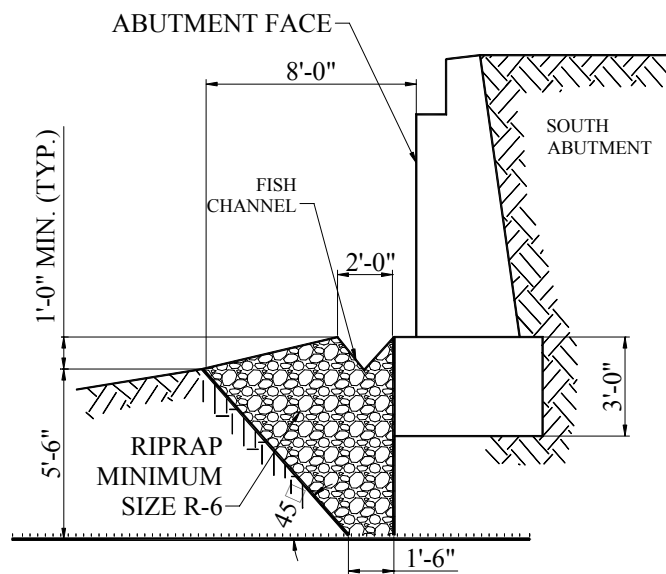


Figure 14.15: Countermeasure details at south abutment

**Note:** Riprap alone 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 15:**

### **CASE STUDIES ON SCOUR COUNTERMEASURES IN NEW JERSEY**

#### **Introduction**

The following steps are generally followed during planning and execution of retrofitting scour critical bridges:

- a. Stage 1 Screening Process
- b. Stage 2 In-depth Study and Selection of countermeasures
- c. Detailed Design of countermeasures
- d. Environmental Permit Approval Process
- e. Preparation of Construction Drawings and Technical Specifications
- f. Installation of countermeasures

This chapter discusses the following:

1. Technical specifications for scour countermeasures
2. Case Studies of In-depth Stage 2 Evaluation at NJ bridges and countermeasures applications, including Excel Spreadsheet Method of Scour Analysis
3. Case studies of environmental permit stage countermeasure design
4. Computation of increased pier scour depth due to debris accumulation

#### **Technical Specifications and Special Provisions**

Technical specifications deal with description of materials, method of construction, units of measurement of pay items and unit costs, and are required for preparing contract documents and for award of contracts. Technical specifications for proprietary countermeasures may be obtained directly from the vendors, if not available in NJDOT Standard Specifications.

The designer should use guidelines provided by the manufacturers only after they have been approved by the NJDOT engineer-in-charge for the project. Appendix IV provides the technical specifications for gabions, which are now being frequently used in place of riprap.

Any deviations from NJDOT Standard Specifications need to be approved as Special Provisions.

#### **Case Studies on In-Depth Scour Study of Three Bridges in New Jersey**

Any design of scour countermeasure should be based on a systematic and in-depth scour study. It is important to follow the required format of the multi-discipline analysis procedures. This is illustrated by case studies on in-depth Stage 2 Study of three NJ bridges below,

1. Two- span Homestead Road Bridge
2. Single-span Guinea Hollow Road Bridge
3. Water Street Bridge

The bridges were selected from review of Stage 2 Study of several bridges in NJ and were recommended for either repair or rehabilitation using countermeasures, or bridge replacement if countermeasures were not cost effective. The case study presented for Guinea Hollow and Water Street bridges is based on detailed study by Baig et al. (2002).

**i. Two- span Homestead Road Bridge located in Hunterdon County, NJ**

Figure 15.1 shows the plan of the two-span Homestead Road Bridge. A detailed in-depth scour study of the bridge has been carried out by NJDOT. Table 15.1 below shows coding of the waterway and the bridge during Stage I and Stage II of the study.

Table 15.1: Coding of the Waterway and the Bridge on Homestead Road.

Coding Guide for NJDOT	Description	Stage I Coding	Stage II Coding
Item 61	Channel and Channel Protection (Stream Stability, Condition of Channel and Slope Protection)	3	3
Item 71	Waterway Adequacy (Overtopping Flood Frequency)	6	7
Item 113	Scour Critical Bridges (North abutment and the pier is rated as unstable due to scour)	4	3*

\*Computed scour calculations for Stage II, Phase 3 show that the bridge is scour critical and the scour depth is below the bottom of the spread footing base. Hence, Stage I SI&A rating is revised to 3 based on scour analysis.

The “TABLE OF CONTENTS” of the In-depth Summary Report in Figure 15.2 indicates a typical extent of NJDOT In-depth scour study for the two-span Bridge. Figure 15.2 illustrates different non-structural items required including field survey, boreholes investigation, field inspection, underwater inspection, rating and soils laboratory test results for the calculation of scour depths.

Following in-depth scour study, new ratings need to be compared with SI&A Sheet ratings for Stage 1 Screening and Prioritization. An appropriate countermeasure needs to be designed after considering different alternatives. Cost estimates should be prepared for these alternatives.

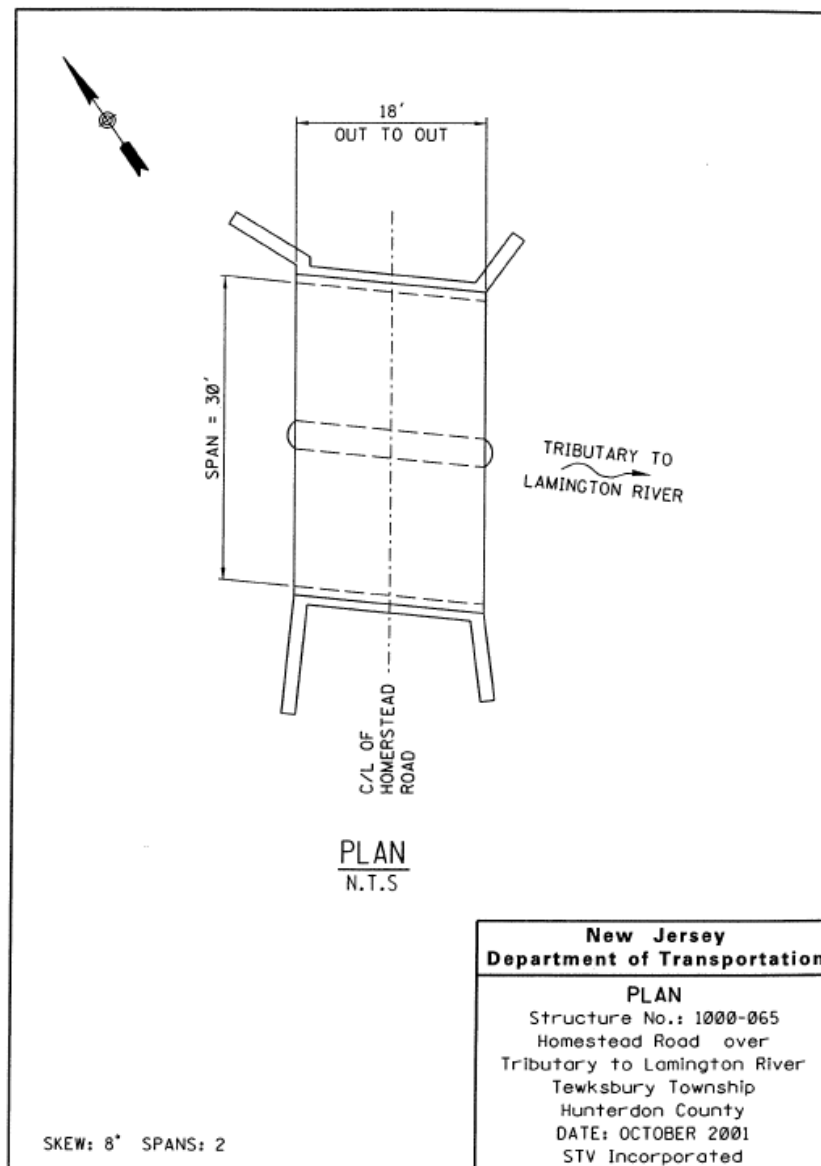


Figure 15.1: Plan of two span Homestead road bridge in Hunterdon county.

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Figure 15.2: A sample of typical contents of scour in-depth evaluation report as required by NJDOT.

Scour Depth Calculations: A detailed scour analysis should be carried out to evaluated scour depths contributed by different scour types. Scour analysis of the Homestead Bridge from Stage II scour study by NJDOT is reproduced below for the reference of readers.

---

**SCOUR EVALUATION: Homestead Road Bridge**

LOCATION: HUNTERDON COUNTY NJ

STRUCTURE NO. 1000-065

CLIENT: NJ DOT

MADE BY: MAK

DATE: 05/17/02

**DESIGN DISCHARGE Q (TOTAL)**

$$Q_{50}=1200\text{CFS}$$

$$Q_{200}=1505\text{CFS}$$

$$Q_{500}=2388\text{CFS}$$

$$D_{50}=(1.8+2.8)/2 = 2.3\text{mm}$$

WIDTH OF BRIDGE OPENING = 28FT

OUT TO OUT WIDTH = 17:18FT

SKEW ANGLE = 20 DEGREES

DISTANCE UPSTREAM OF BRIDGE= 20FT

SURVEY STATION # UPSTREAM OF BRIDGE = (RS 1340.77)

MANNINGS n VALUES:

n= 0.045 FOR CHANNEL

n= 0.1 FOR BANKS

$$g= 32.2 \text{ ft/sec}^2$$

TOTAL SCOUR=LONG TERM SCOUR + COUNTERACTION SCOUR + LOCAL SCOUR

**LONG TERM SCOUR**

NO EVIDENCE OF CONTINUING OF SIGNIFICANT LONG TERM AGGRADATION OR DEGRADATION DUE TO THE PRESENCE OF GRAVELS AND STONES IN CHANNEL.

NEGLECT SCOUR DUE TO AGGRADATION AND DEGRADATION.

CONTRACTIONS SCOUR ANALYSIS WITH AVERAGE D50

USE CRITICAL VELOCITY EQUATION, HEC 18, EQ. 5.1,PAGE 5.2

$$V_c=11.17(y)^{1.6}(D)^{1.0}$$

y=AVG.DEPTH OF UPSTREAM FLOW

$V_c$ = CRITICAL VELOCITY TO TRANSPORTATION  $D_{50}$  OR SMALLER MATERIALS

SELECT HEC-RAS OUTPUT SECTION

(RS 1340.77)

### **DETERMINE TYPE OF CONTRACTION SCOUR**

EVENT	y <sub>1</sub> FT	D <sub>50</sub> FT	V <sub>C</sub> =11.17(y) <sup>1.6</sup> (D <sub>50</sub> ) <sup>1/3</sup> FT/SEC	V FT/SEC	V < V <sub>C</sub>	MODE OF TRANSPORT
50 YR.	3.74	0.0075	2.72	5.57	NO	LIVE BED
100 YR.	5.67	0.0075	2.92	4	NO	LIVE BED
500 YR.	6.33	0.0075	2.97	0.656	NO	LIVE BED

### **LIVE BED CONTRACTION SCOUR:**

$$y_2 / y_1 = (Q_2 / Q_1)^{6/7} \times (w_1 / w_2)^{k_1} \quad \text{----- EQUATION 5.2, PAGE 5.10, HEC 18}$$

y<sub>1</sub> = AVERAGE DEPTH IN UPSTREAM MAIN CHANNEL

y<sub>2</sub> = AVERAGE DEPTH IN CONTRACTED SECTION

Q<sub>1</sub> = FLOW IN UPSTREAM CHANNEL

Q<sub>2</sub> = FLOW IN THE CONTRACTED SECTION

y<sub>i</sub> = AVERAGE SCOUR DEPTH

SELECT HEC-RAS OUTPUT SECTION

RS 10130

### **COMPUTE COEFFICIENT k<sub>1</sub>**

EVENT	y <sub>1</sub> FT	S <sub>1</sub> FT/FT	V <sup>*</sup> =(g × y × S <sub>1</sub> ) <sup>1/2</sup> FT/SEC	D <sub>50</sub> MM	ω FT/SEC	V <sup>*</sup> /ω	k <sub>1</sub>
50 YR.	3.74	4.13E-03	0.71	2.3	0.80	0.88	0.64
100 YR.	5.67	1.22E-03	0.47	2.3	0.80	0.59	0.64
500 YR.	6.33	2.02E-03	0.64	2.3	0.80	0.80	0.64

FOR VALUES OF k<sub>1</sub> SEE PAGE 5.11

### **EVALUATE LIVE BED CONTRACTION SCOUR**

EVENT	y <sub>0</sub> (FT)	Q <sub>2</sub> (CFS)	Q <sub>1</sub> (CFS)	W <sub>1</sub> (FT)	Width of opening (L <sup>*</sup> ) (FT)	Skew Angle (phi) (Radians)	W <sub>1</sub> =L × cos(phi) (FT)	y <sub>2</sub> /y <sub>1</sub> =(Q <sub>2</sub> /Q <sub>1</sub> ) <sup>6/7</sup> × (w <sub>1</sub> /w <sub>2</sub> ) <sup>k<sub>1</sub></sup>	y <sub>2</sub> =y <sub>1</sub> x(y <sub>2</sub> /y <sub>1</sub> ) (FT)	Y <sub>1</sub> =y <sub>2</sub> - y <sub>0</sub>
50 YR.	4.96	1200	1200	57.51	28.55	18.00	26.63	1.64	6.12	1.16
100YR.	2.25	1464	1333	64.53	28	18.00	26.63	1.63	9.22	6.97
500YR.	2	2263	1691	64.53	28	18.00	26.63	1.37	8.69	6.69

**LOCAL SCOUR AT PIERS:****SELECT** A STATION DIRECTLY UPSTREAM OF THE PIER. (STA. 13+21)USE CSU EQ. 6.1, P. 6.2  $y_s = 2K_1K_2K_3K_4(n/y_1)^{0.35}Fr_1^{0.43}$ K<sub>1</sub>=CORRECTION FACTOR FOR PIER NOSE SHAPE FROM FIG. 6.3 & TABLE 6.1K<sub>1</sub>=1.0 FOR ROUND NOSEK<sub>2</sub>=CORRECTION FACTOR FOR ANGLE OF ATTACK OF FLOW. TABLE 6.2 OR EQN.  
 $6.4 - (\cos(20) + L/a \sin(20))^{0.65} = 4.018^{0.65} = 2.47$ K<sub>3</sub>=CORRECTION FACTOR FOR BED CONDITION FROM TABLE 6.3-1.1K<sub>4</sub>=CORRN. FACTOR FOR ARMORING BY BED MATERIAL SIZE EQN. 6.5 & TABLE 6.4USE K<sub>4</sub>=1 FOR D<sub>50</sub><2MMMa= PIER WIDTH-2.0FT., L=LENGTH OF PIER-18.0FT., L/a=9,  
Fr<sub>1</sub>=FROUDE NUMBER DIRECTLY UPSTREAM OF PIER- $V_1/(gy_1)^{0.5}$ V<sub>1</sub>=MEAN VELOCITY OF FLOW DIRECTLY UPSTREAM OF PIER FT./SEC**COMPUTE LOCAL SCOUR DEPTHS @ PIER**

EVENT	K <sub>1</sub>	K <sub>2</sub>	K <sub>3</sub>	K <sub>4</sub>	y <sub>1</sub>	A	V <sub>1</sub>	Fr <sub>1</sub>	a/y <sub>1</sub>	$y_s = 2K_1K_2K_3K_4 \frac{(n/y_1)^{0.35}}{Fr_1^{0.43}}$
50 YR.	1	2.47	1.1	1	3.9400	2.000	5.0200	0.4457	0.6345	9.73
100YR.	1	2.47	1.1	1	5.9700	2.000	3.6700	0.2647	0.4188	9.00
500YR.	1	2.47	1.1	1	6.6400	2.000	5.0800	0.3474	0.3765	10.50

**SUMMARY OF SCOUR DEPTHS PIER**

EVENT	LONG TERM SCOUR FT.	CONTRACTION SCOUR FT.	LOCAL SCOUR FT.	TOTAL SCOUR FT.
50 YR.	0	1.16	9.73	10.90
100YR.	0	6.97	9.00	15.97
500YR.	0	6.69	10.50	17.19

Scour depths calculated by HEC-RAS output should be verified with scour depth calculations programmed in spreadsheet. Any discrepancy between the two approaches should be resolved by carefully reviewing data used in scour depth calculations. Figure 15.3 shows the plots of scour depths during the 100-year flood event for the Homestead Road Bridge.



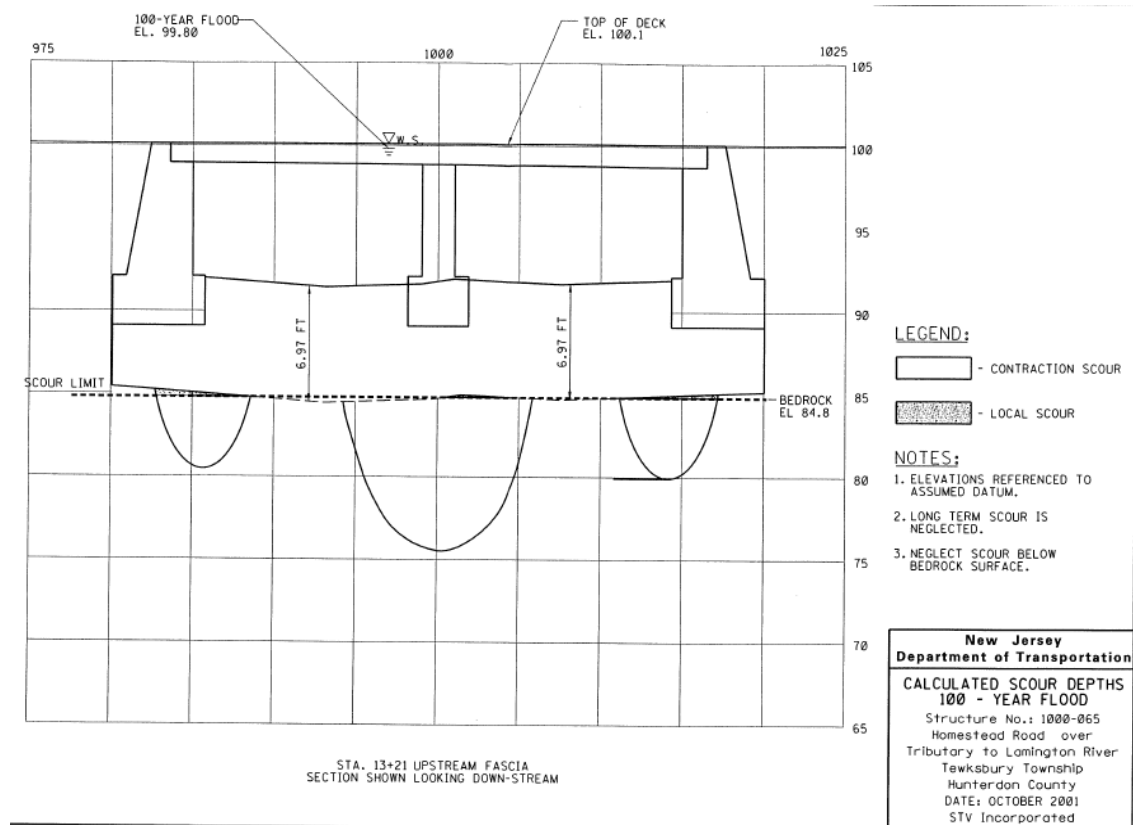


Figure 6. Calculated Scour Depths

Figure 15.3: Plots of Calculated Scour Depths during the 100 Year Flood Event for the Homestead Road Bridge.

## ii. Single-span Guinea Hollow Road Bridge

Based upon Stage II bridge scour evaluation (the same detailed procedure as for Homestead Road Bridge), the following SI&A codings for this structure are recommended:

Table 15.2: Coding of the Waterway and the Bridge on Guinea Hollow Road.

Coding Guides	Description	Stage I Coding	Stage II Coding
Item 61	Channel/Channel Protection (Stream Stability, Channel Condition, Slope Protection)	6	6
Item 71	Waterway Adequacy (Overtopping Flood frequency)	9	9
Item 113	Scour Critical Bridges (Abutments are rated as unstable due to scour)	4	3*

*\* Computed scour calculations for Stage II, Phase 3 show that the bridge is scour critical and scour depth is below the bottom of spread footing base. Hence Stage I SI&A rating is revised to 3 based on scour analysis.*

### ***Proposed Countermeasures and Approximate Cost***

Figure 15.4 shows plots of local and contraction scour depths for the single-span Guinea Hollow Road Bridge. It is observed that the total estimated scour depth is below the estimated bottom of footing elevations. Hence, placing of riprap adjacent to the apron on north and south abutments appears to be the most cost-effective method. Conceptual sketches, showing riprap depth and extent of riprap, are presented in Figure 15.5. The detailed cost-estimate of the work is as follows:

1. Repairs prior to implementing countermeasures, (including filling missing mortar joints, removing debris from channel bed, filling scour holes and gaps between the bottom of footing and top of soil next to south abutment, with concrete) =  
= \$ 5,000
2. Constructing Concrete aprons, cofferdams and de-watering = \$12,000
3. Providing riprap at abutments & wingwalls = \$29,000
4. Providing gabions at banks = \$29,000
- Approximate Total Cost = \$75,000

### ***Summary of Results for Guinea Hollow Road Bridge***

Spreadsheets were developed based on applicable methods and formulae. The total estimated scour depth is below the estimated bottom of footing elevations. The following additional factors indicate that the bridge is susceptible to scour.

- Evidence of undermining at both abutment footings
- Evidence of poor quality masonry walls with missing mortar in joints
- Evidence of large size scour holes close to the abutments

### ***Conclusions for Guinea Hollow Road Bridge***

The abutment walls are skewed to the direction of the flood water, and the footing elevation is placed above the computed scour depth. There is absence of river training measures and lack of foundation armoring. Strengthening the footing and installation of suitable armoring, such as riprap are recommended. Continued monitoring of the footings and of the main channel during routine biennial inspections will be required.

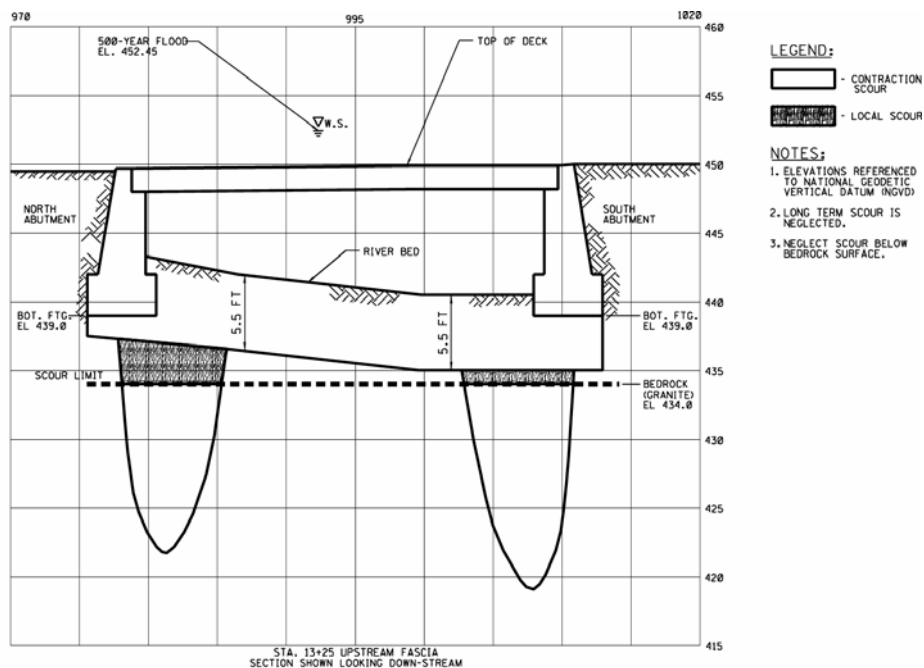


Figure 15.4: Calculated Scour Depths for 500-Year Flood – Structure No. 1000-41; Guinea Hollow Road Over North Branch Rockaway Creek.

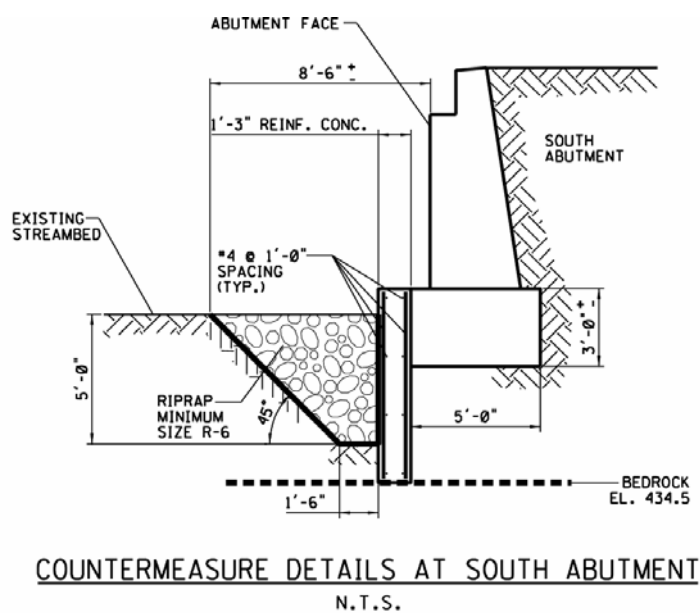


Figure 15.5: Scour Countermeasures – Structure No. 1000-041: Guinea Hollow Road Over North Branch Rockaway Creek.

### iii. Water Street Bridge

It has been observed from In-Depth Phase II study that the Water Street Bridge would need replacement but will require interim (temporary) countermeasures [Baig et al. (2002)]. Figures 15.6 and 15.7 show scour depths and temporary countermeasure details.

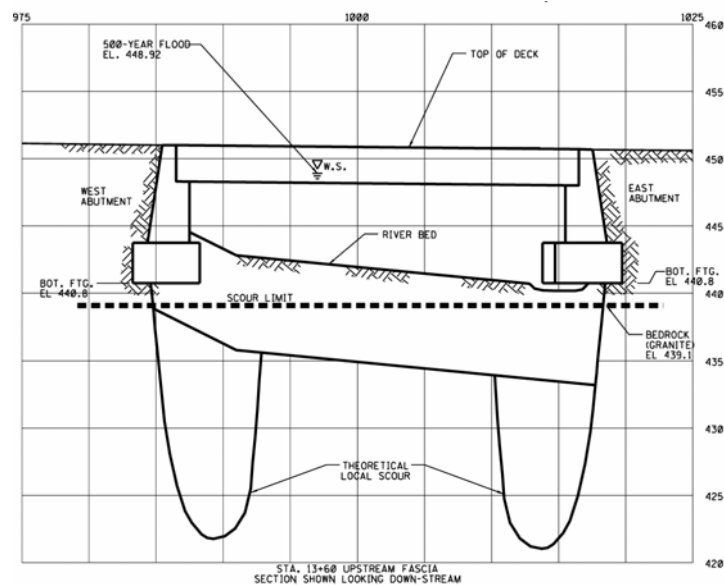


Figure 15.6: Calculated Scour Depths for 500 Year Flood – Structure No. 1000-042; Water Street Over Trib. to North Branch Rockaway Creek.

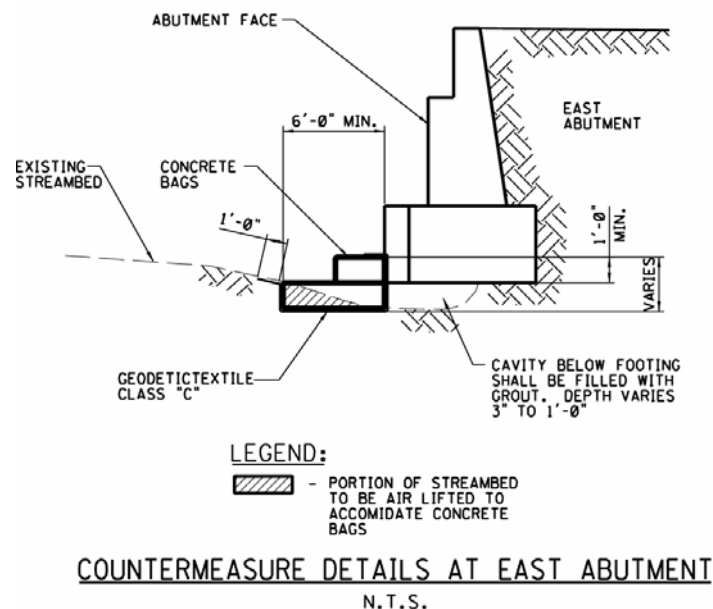


Figure 15.7: Scour Countermeasures – Structure No. 1000-042; Water Street Over Trib. To North Branch Rockaway Creek.

## Case Studies of Environmental Permit Stage Countermeasure Design

Case studies for Post In depth Study of three scour critical bridges are presented here. The bridges were classified as scour critical following Stage 1 evaluation. An in-depth Stage 2 study recommended conceptual design of countermeasures. Three scour critical bridges are:

1. Route 22 over Peter's Brook, located in Bridgewater Township, Somerset County
2. US Route 1 over Shipetaukin Creek, located in Lawrence Township, Mercer County
3. Route 31 over Spruce Run, located in Bethlehem and Lebanon Townships, Hunterdon County

The following alternates were considered for countermeasures and the findings are:

**Monitoring:** Installation of instruments for long distance monitoring was considered. It was not adopted due to expense.

### River Training Structures

Since local scour at the abutments is a concern, possible river training devices considered were:

- a. Guide banks (also known as spur dikes)

This type of structure involves the placement of fill to form embankments on the upstream side of the structure. This moves the point of maximum scour away from the abutments to upstream of bridge. The disadvantage would be placing significant amount of fill in the flood plain. It would also need to be combined with armoring.

- b. Installation of a grade control structure (such as a check dam)

It consists of sheet piles installed across the channel at downstream edge of bridge. It will prevent additional degradation of the channel in the bridge area and also provide vertical stability of streambed upstream of check dam. The disadvantages would be turbulence resulting from energy dissipation at drop downstream of dam and impacting the hydraulic characteristics of stream. Lateral erosion of banks downstream of dam will require additional use of armoring.

- c. Constructing relief bridge

Although it will reduce scour at the main bridge it is costly to implement. The additional cost may be equivalent to replacement cost of the original bridge. There may be difficulty with right of way since the bridge is in a built-up area.

### Structural Countermeasures

One type of structural modifications may require constructing new footings at a lower level or on mini piles. The disadvantages would be an extensive disturbance to streambed during construction and cost of the footings.

**Armoring:** The following types were considered:

- a. Gabion Mattress on geo-textile

- b. Rock Riprap on geo-textile
- c. Concrete apron

**Discussions on Selection of Countermeasures:** Tables 15.3 and 15.4 present comparative design of Gabions and Riprap for the three bridges for 100 years flood scenario. Riprap is unstable as compared to gabion mattress and is likely to move during floods. The depth of excavation below riverbed for riprap is far greater than for gabion mattress design. The use of concrete apron will be over-conservative for the magnitude of flood velocities. Hence, Gabion mattress design was recommended for construction.

In addition to the installation of Gabions, the following considerations are recommended:

1. Monitoring frequency be increased for scour critical bridges, especially during flood season. For greater water depths, underwater inspections should be carried out. For effective monitoring, installation of suitable instruments is recommended.
2. Gabions should be considered as temporary countermeasures until the bridge is replaced.
3. Armoring methods in addition to Gabions, as discussed in this handbook, should be considered as alternatives, before a final selection is made.
4. Structural countermeasures in the form of repairs of footings should be considered.
5. Any scour holes need to be filled up and any debris accumulation needs to be cleared.

Table 15.3: Comparative Study of Design of Gabions Mattress for 100 year floods.

Bridge Location	Average channel velocity upstream of bridge (ft/sec)	Average channel velocity d/s of bridge (ft/sec)	Gabion Thickness	Average channel velocity at abutment (ft/sec)	Gabion Thickness	Average channel velocity at pier (ft/sec)	Gabion Thickness	Remarks*
Route 1 over Shipetaukin Creek	-	-	-	3.39	6 inch	4.31	6 inch	6 inch thickness governs (Low flood vel.)
Route 31 over Spruce Run	11.5	-	12 inch	6.31	9 inch	7.42	9 inch	12 inch thickness Governs
Route 22 over Peters Brook	5.71	6.46	9 inch	-	9 inch	-	9 inch	9 inch thickness Governs

Notes: Use 3-inch layer of coarse aggregate under gabion mattress with a filter fabric.

Use native streambed material of 6-inch minimum thickness as top cover.

Current channel bed elevation should be maintained.

Length of gabion should extend twice the flow depth from face of abutment.

\* Use 12-inch thickness for avg. velocity not exceeding 14 ft/sec.

Table 15.4: Comparative Study of Design of Riprap for 100 Year Floods.

Bridge Location	Av. channel vel. At abutment (ft/sec)	Med. stone diameter (D50) of Riprap	Av. Channel velocity at pier (ft/sec)	Med. stone dia. (D50) of Riprap	Remarks
Route 1 over Shipetaukin Creek	3.39	3 inch	4.31	5 inch	Total thk. = 3 times D50 =15 inch min.
Route 31 over Spruce Run	6.31	9 inch	7.42	10 inch	Total thk. = 3 times D50 =30 inch min.
Route 22 over Peters Brook	> 5		> 6	10 inch	Total thk. = 3 times D50 =30 inch min.

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## APPENDIX II: FLOW DIAGRAMS FOR HYDROLOGIC AND HYDRAULIC ANALYSIS

The following flow charts may be followed in developing a comprehensive scour analysis. They are based on standard procedures from HEC-18 and other publications. These diagrams are included in Section 46 of NJDOT Bridge Design Manual and were originally developed by the authors of this handbook.

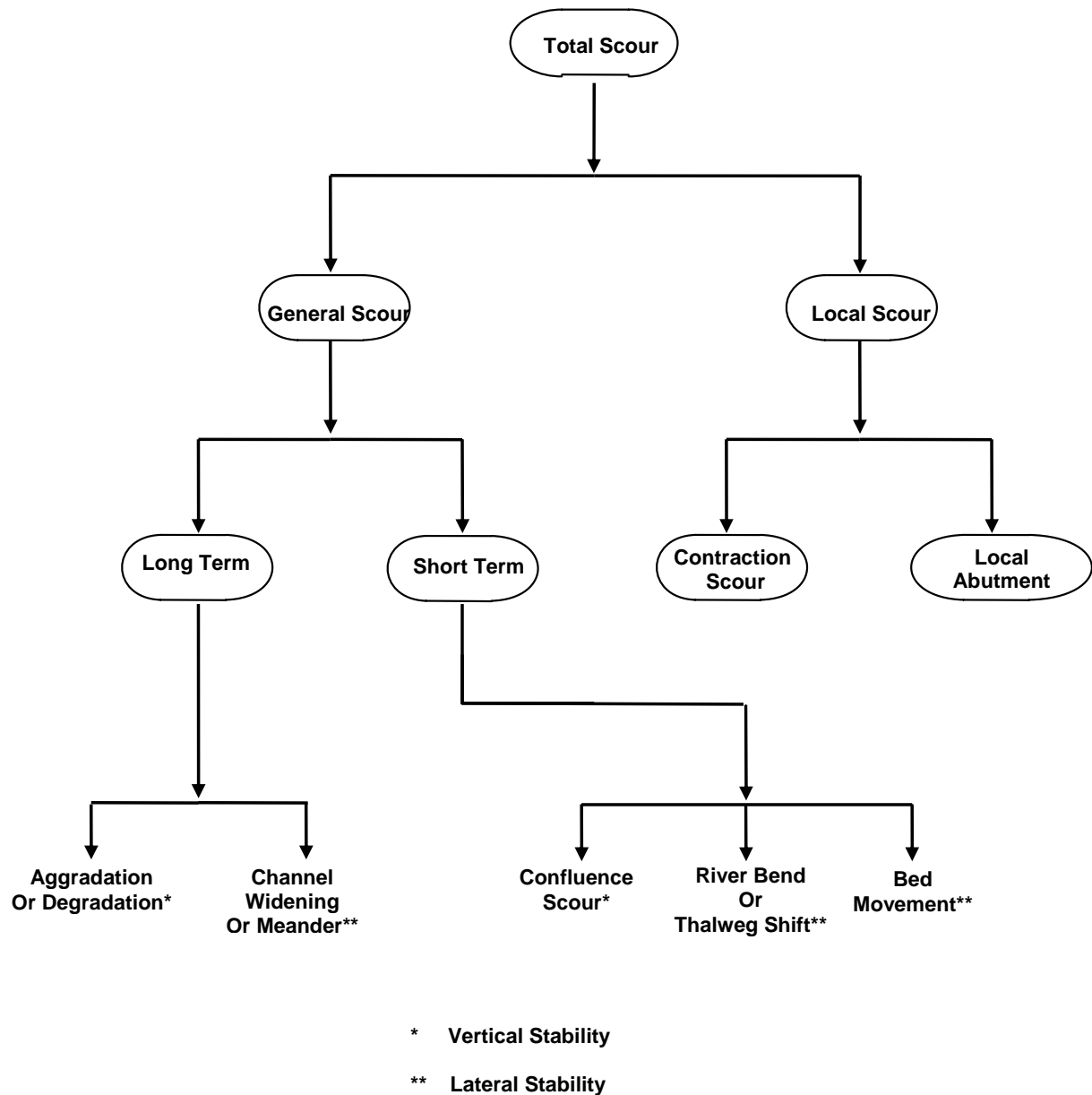


Figure A.II.1: Scour components on a bridge structure site.

### Notes:

1. If bridge inspection records of cross sections of the stream at the bridge site are maintained over many years, long-term scour can be calculated from streambed elevation changes. Projections of scour based on a long-term trend can be made.
2. Long-term scour is based on the concept that long-term streambed elevation changes can take place over the time scale of several years during the life of a bridge, due to aggradation or degradation. In the absence of reliable data for long-term scour, a minimum 1.0 ft. scour depth may be assumed.
3. Aggradation involves the deposition of material eroded from the channel or watershed upstream of the bridge. Common Countermeasures used for aggradation are channel improvements by dredging or cleaning.
4. Degradation involves the lowering or scouring of the longitudinal profile of a channel. The bends of meandering channels may move laterally in the vicinity of the bridge, causing the channel to widen and create lateral erosion and scour. Countermeasures for degradation are channel lining with concrete pavement, increasing a bridge opening width or vegetation planting.

### **Flow Diagrams For Detailed Scour Evaluation**

The flow charts in Figures A.II.2 and A.II.3 may be followed for a comprehensive scour analysis.

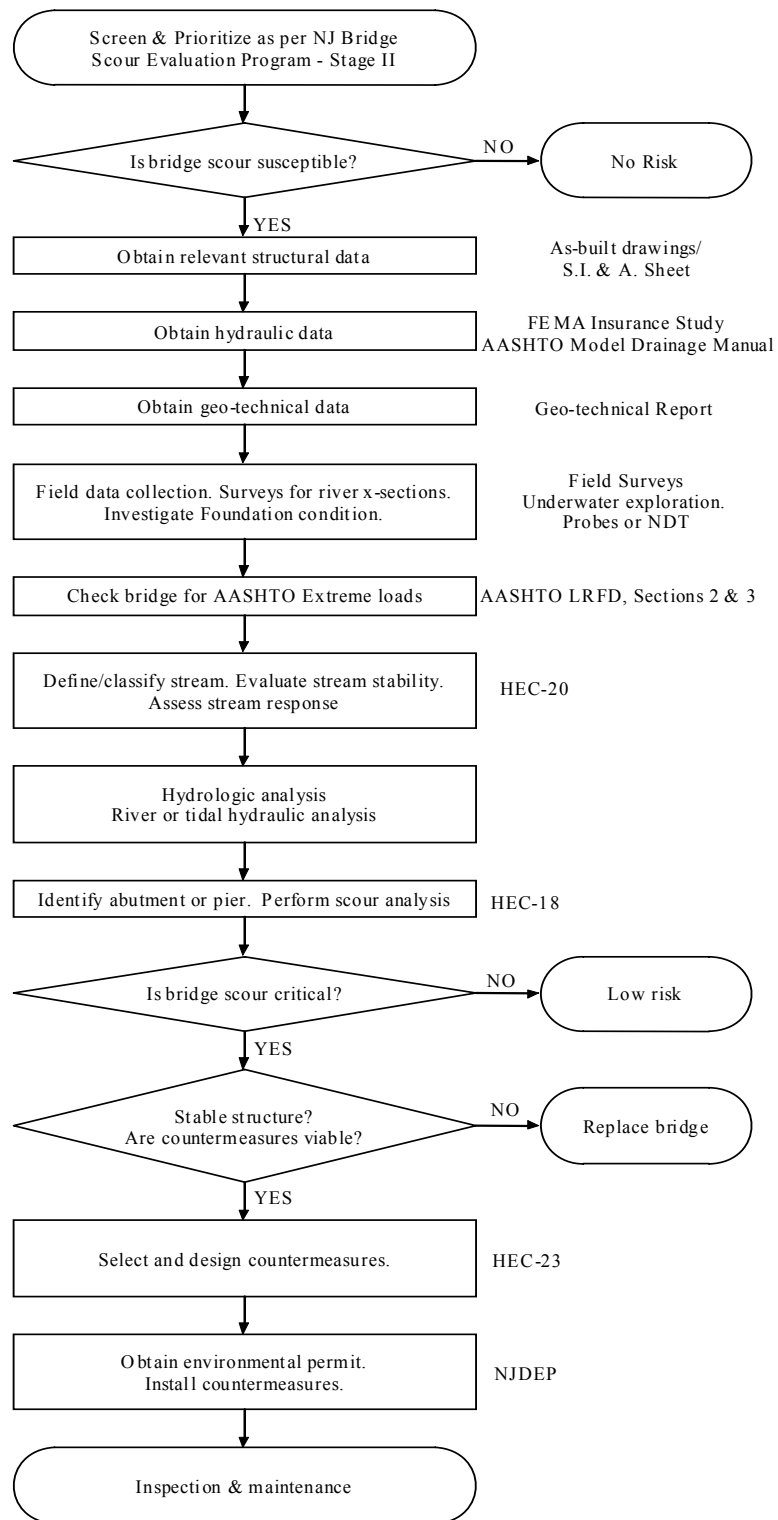


Figure A.II.2: Flow Chart for Scour Analysis of Existing Bridges and Bridges to be widened.

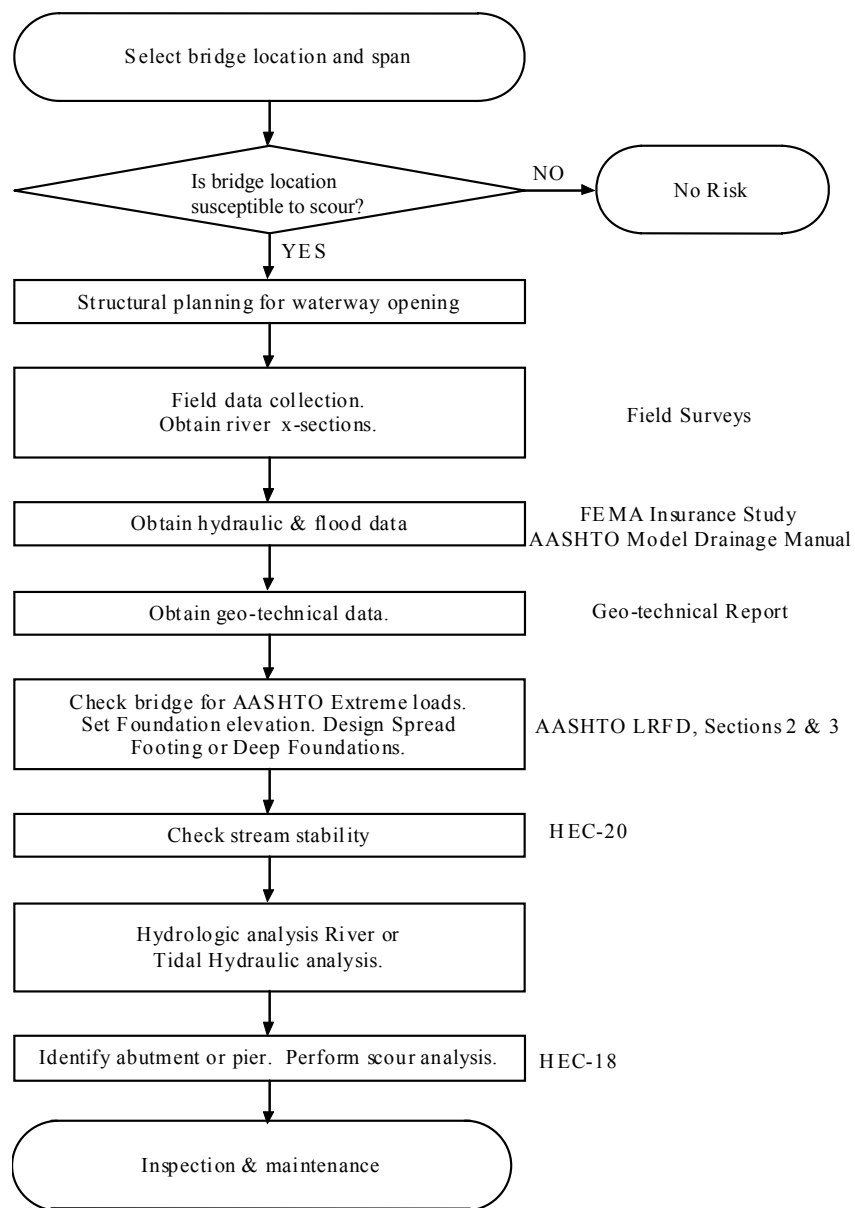


Figure A.II.3: Flow Chart for Scour Analysis of New Bridges and Bridges to be replaced.

## APPENDIX III: CONSTRUCTION DRAWINGS OF BRIDGES WITH COUNTERMEASURES

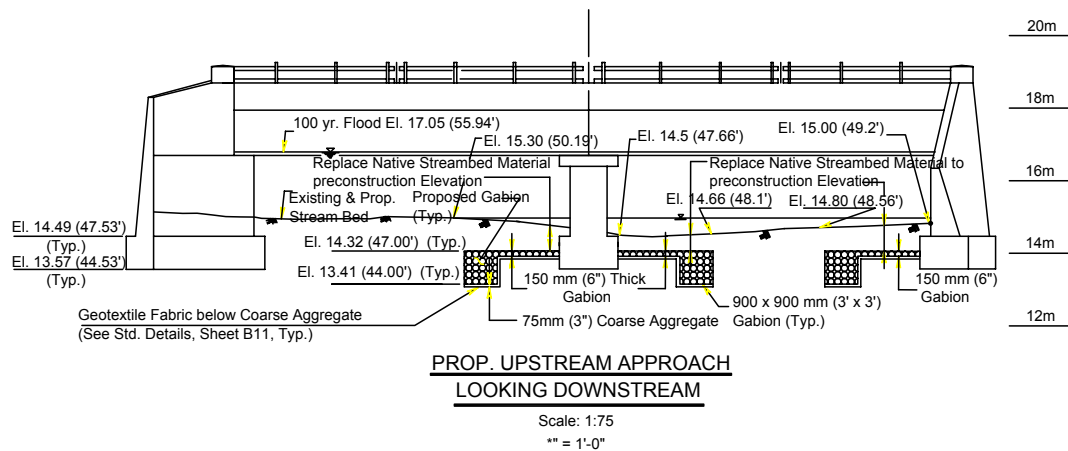


Figure A.III.1: Prop. Upstream Approach Looking Downstream, bridge # 1103151, Lawrence TWP. Mercer, Municipality County

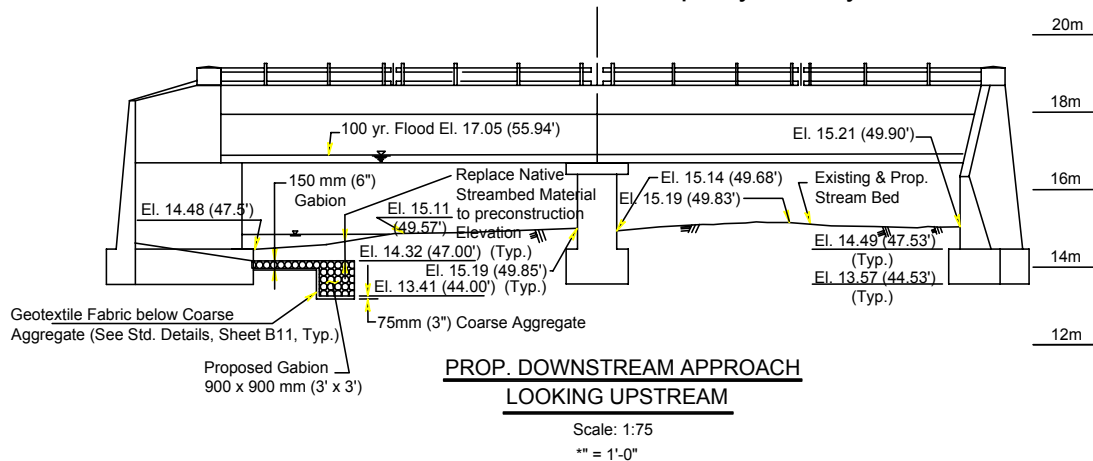


Figure A.III.2: Prop. Upstream Approach Looking upstream, bridge # 1103151, Lawrence TWP. Mercer, Municipality County

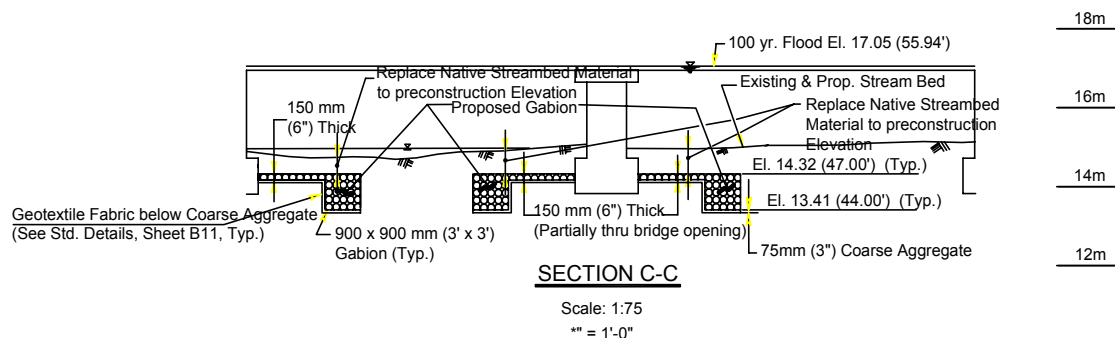
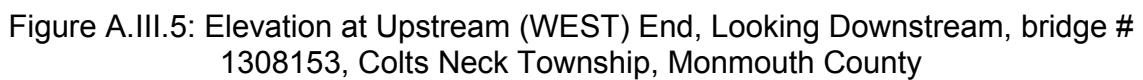
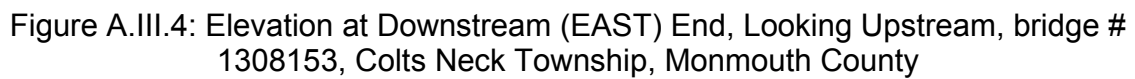


Figure A.III.3: Prop. Section C-C, bridge # 1103151, Lawrence TWP. Mercer, Municipality County





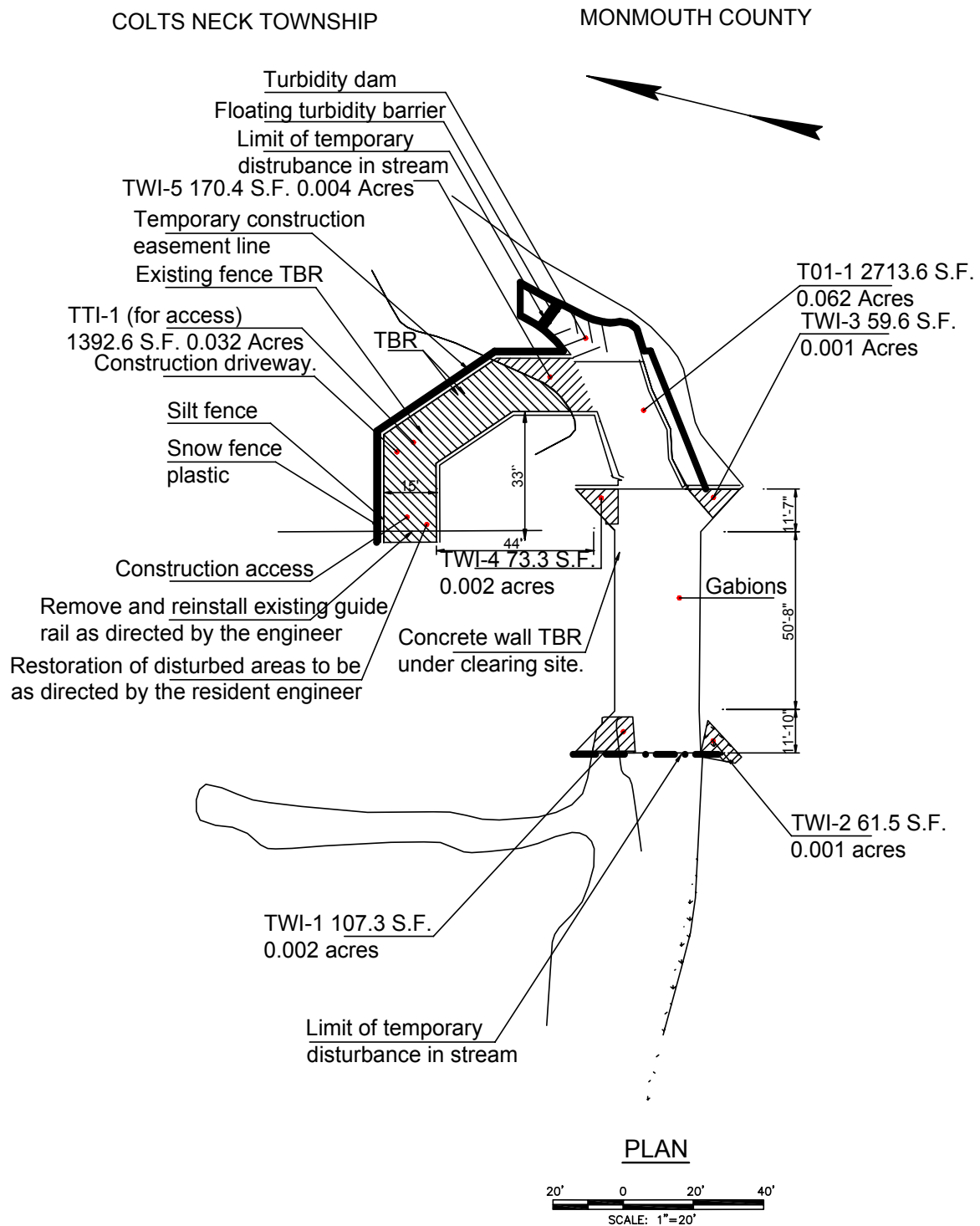


Figure A.III.7: Plan of bridge # 1308153o2gpin Colts Neck Township, Monmouth County.

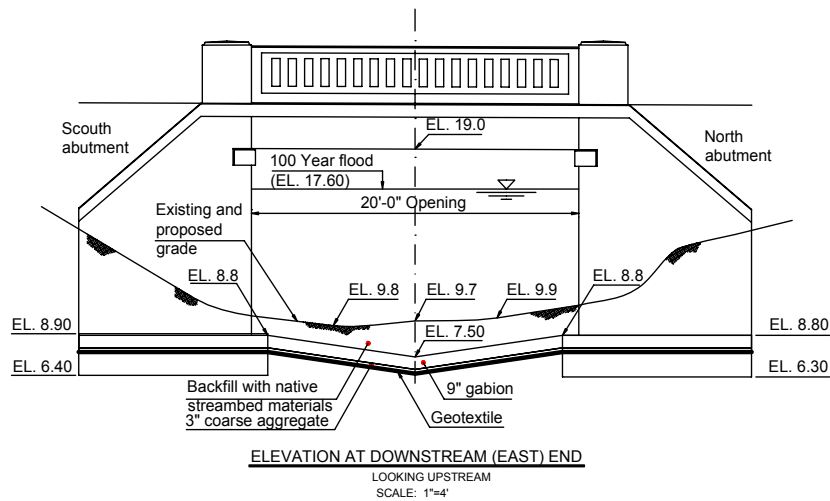


Figure A.III.8: Elevation at Downstream (EAST) End, Looking Upstream, bridge # 1310155, Route 35 over North Branch of Wreck Pond Brook, Monmouth County

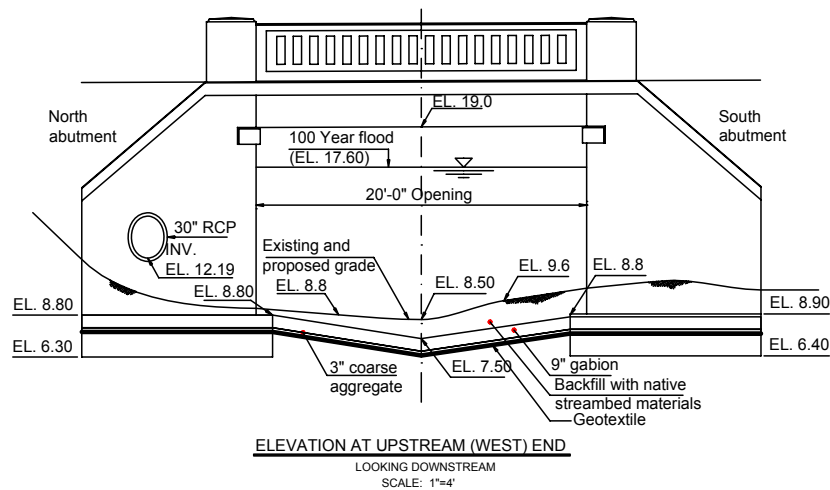


Figure A.III.9: Elevation at Upstream (WEST) End, Looking Downstream, bridge # 1310155, Route 35 over North Branch of Wreck Pond Brook, Monmouth County

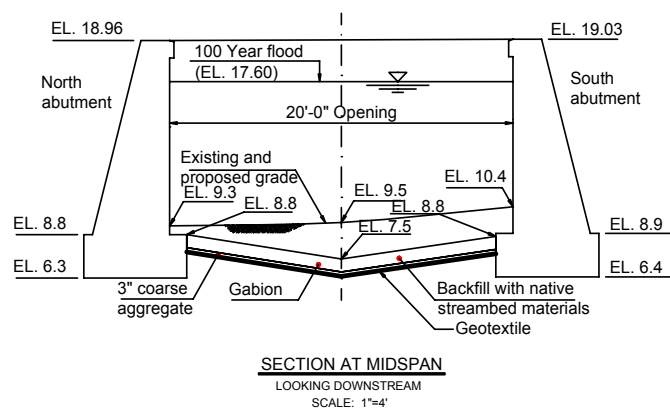


Figure A.III.10: Section at Midspan, Looking Downstream, bridge # 1310155, Route 35 over North Branch of Wreck Pond Brook, Monmouth County

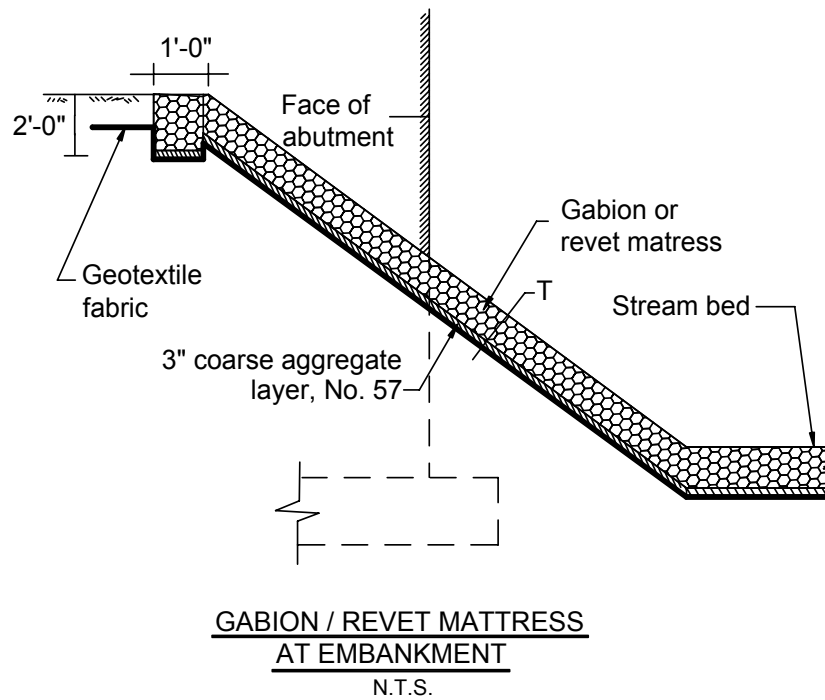


Figure A.III.11: Standard Details of Gabions at Embankments

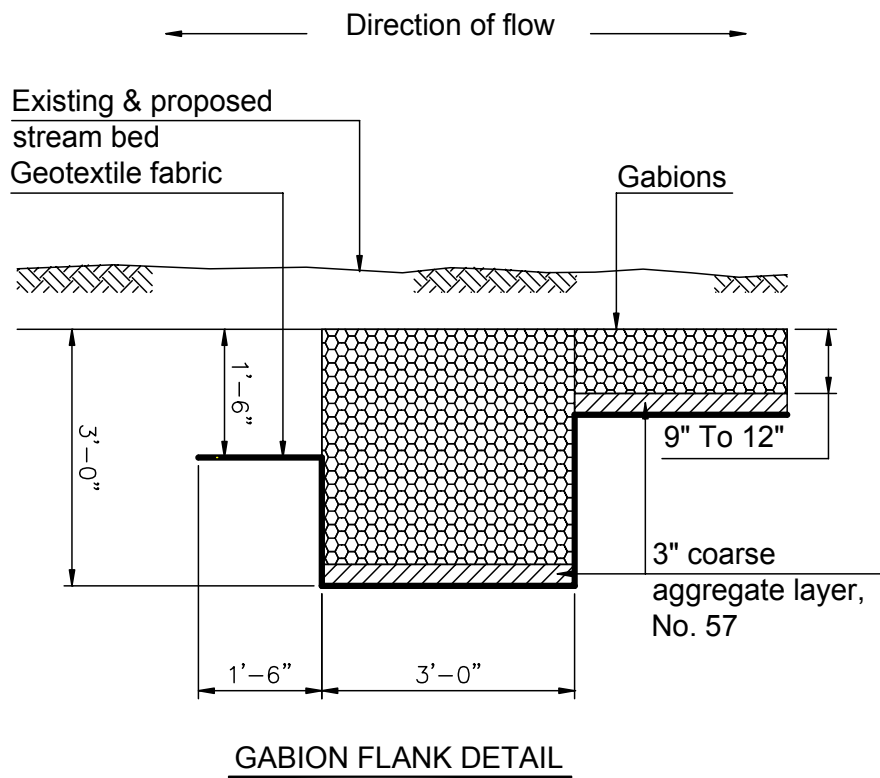
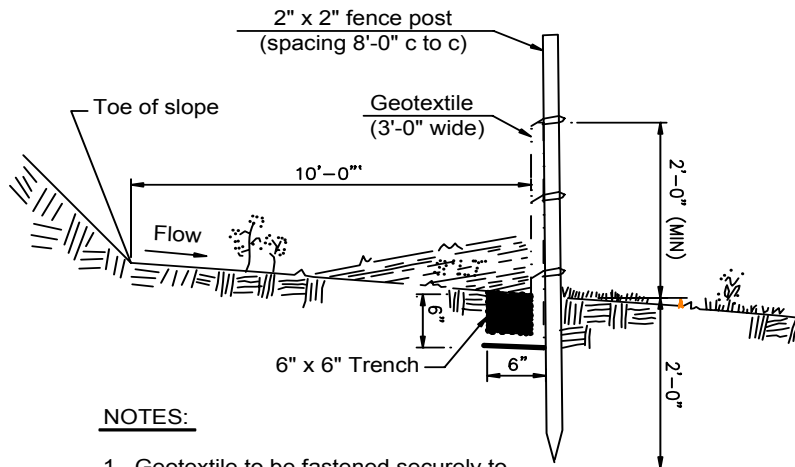


Figure A.III.12: Gabion Flank Detail

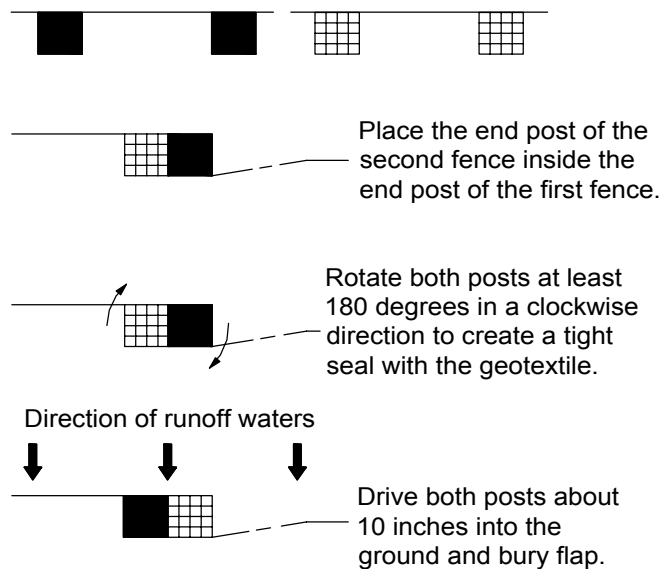


**NOTES:**

1. Geotextile to be fastened securely to fence post by use of wire ties or hog rings. Three fasteners per post.
2. Bury bottom 1'-0" of geotextile and tamp in place.
3. Ends of individual rolls of geotextile shall be securely fastened to a common post by wrapping each eand of the geotextile around the post twice and attaching as specified in note 1 above. Splicing of individual rolls shall not occur at low points.

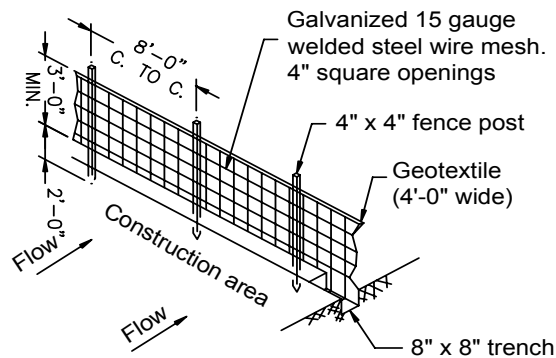
**SILT FENCE**

Figure A.III.13: Silt Fence



**ATTACHING TWO SILT FENCES**

Figure A.III.14: Attaching Two Silt Fences

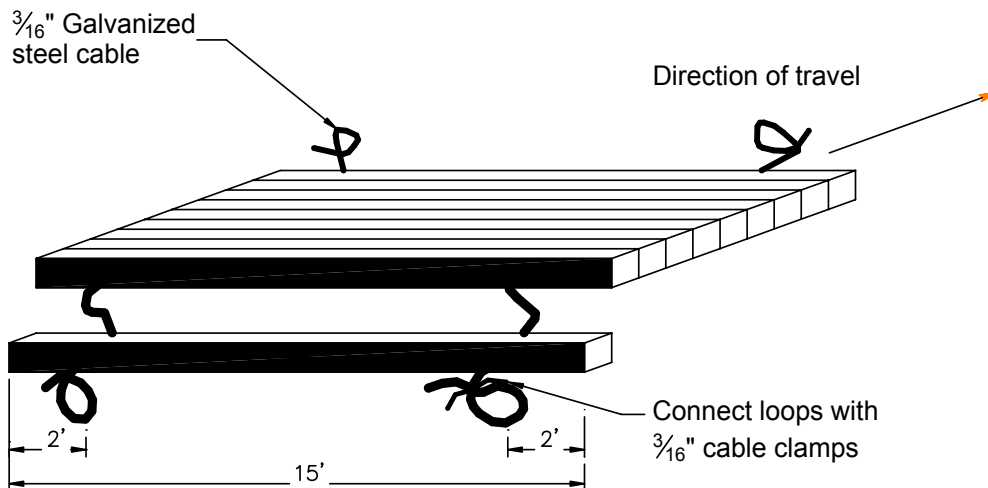


**NOTES:**

1. Geotextile to be fastened securely to wire mesh and fence post by use of wire ties or hog rings. 3 fasteners per post.
2. Bury bottom 1'-0" of geotextile and tamp in place.
3. Ends of individual rolls of geotextile shall be securely fastened to a common post by wrapping each end of the geotextile around the post twice and attaching as specified in note 1 above. Splicing of individual rolls shall not occur at low points.

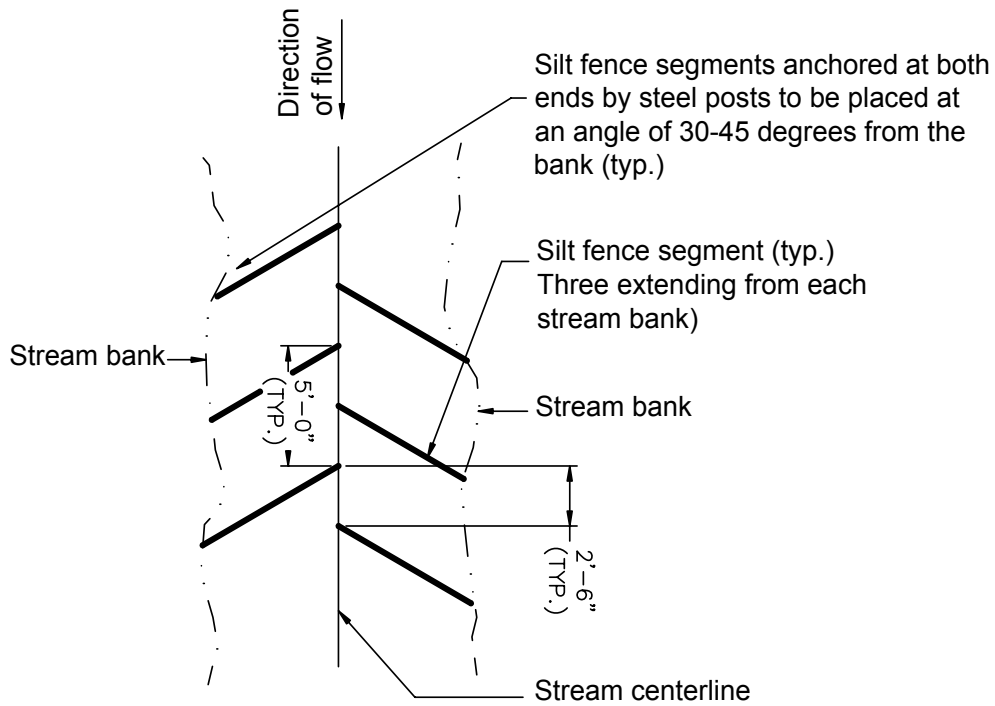
**HEAVY DUTY SILT FENCE**

Figure A.III.15: Heavy Duty Silt Fence



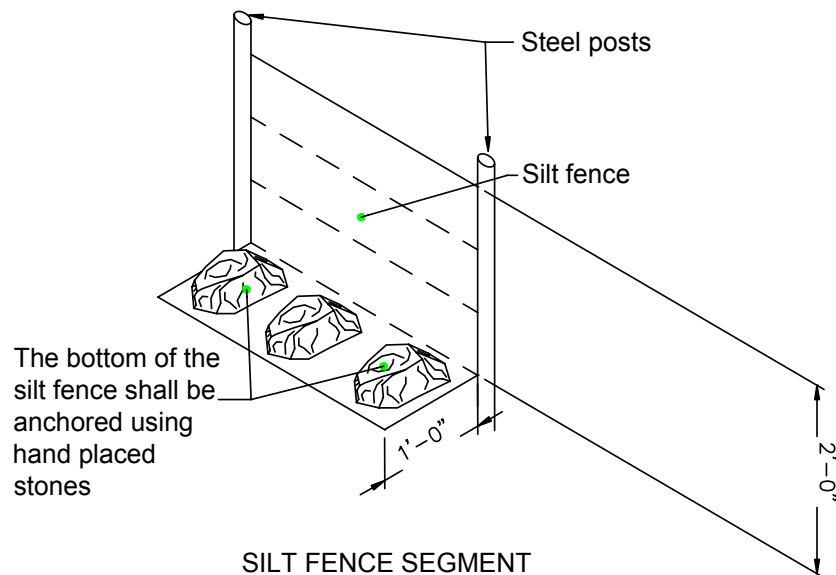
**CONSTRUCTION DRIVEWAY, WOOD MATS**

Figure A.III.16: Construction Driveway. Wood Mats



#### TURBIDITY DAM

Figure A.III.17: Turbidity Dam



#### SILT FENCE SEGMENT

ALL COST FOR "SILT FENCE SEGMENT" IS INCLUDED IN THE ITEM, "TURBIDITY DAM"

Figure A.III.18: Silt Fence Segment

Typical Construction Notes:

1. All elevations are referenced to the NJDOT bridge plans.
2. Construction equipment shall not be washed in the waterway or in the areas in which wash water will drain into the waterway.
3. Sensitive area-wetlands/state open waters. The contractor shall not encroach upon or store any equipment/vehicle/materials in wetland/transition areas/state open water areas beyond those approved in any issued permits or shown on the plans. The contractor is permitted to be only in those wetland/transition areas/state open water areas that are necessary to gain access to the work zone as depicted on the plans and in the work zone itself. All remaining areas of wetlands, transition areas or stream are not to be encroached upon. Therefore, the contractor must adhere to the locations of the silt fence and plastic snow fence and adhere to the limits of disturbance in stream. In addition, stockpiles, vehicles, and/or equipment shall not be located within 50 feet of a slope, drainage facility, wetland, or flood plain. A hay bale barrier or silt fence shall protect all stockpile bases.
4. All work to be done in accordance with the NJDOT standards for soil erosion and sediment control.
5. All soil erosion and sediment control practices to be installed prior to any major soil disturbance or in their proper sequence, and maintained until permanent protection is established. Plastic snow fence shall also be erected prior to construction and shall remain in place for the duration of construction. In areas where silt fence and snow fence are shown to be coincident on the plans, the snow fence shall be installed immediately behind the silt fence in relation to the toe of the slope.
6. Construction access road and other areas of temporary impact shall be restored to original grade and shall be replanted with material listed on estimate of quantity sheet upon completion of construction as directed by the resident engineer in consultation with the environmental team and the landscape and urban design unit. 72 hours notice shall be given before any plant material is brought on the project.
7. Pumpage of sediment-laden water from dewatering activities directly into the stream is prohibited. Dewatering basins or filter bags needed for dewatering activities should not be located in un-impacted wetland or transition areas or in floodplains. Water from dewatering basins should be returned to a portion of the stream that is protected by turbidity barrier.
8. Vegetation outside the limits of the construction driveway and other areas of temporary impact, as shown on the plans, shall be preserved, unless otherwise directed by the engineer.
9. Stream flow shall be maintained at all times. The countermeasures shall be constructed in a minimum of two stages with the stream being diverted to the portion of the channel not under construction. Regardless of how the construction is staged, floating turbidity barrier shall be used. No stream flow of storm run off shall be permitted to flow over disturbed areas.
10. Earthen beams shall not be used as cofferdams.



11. The following time restriction on all in-water construction activities will be required for the following natural resources (njac 7:13-3.6): warm water fish: march 15 through June 30
12. If the project is located in a geologic formation that has the potential for encountering acid-producing deposits during construction of access through the stream banks, the area must be identified and precautions taken to minimize exposure as described in the stream encroachment manual technical notes, especially section 2.5 technical notes concerning acid producing deposits. Should construction of the driveway require excavation, the contractor shall perform testing of the soil prior to construction of the construction driveway.
13. No change in plans or specifications shall be made except with prior written permission of the NJDEP.
14. A copy of the permit(s) shall be kept at the work site and shall be exhibited upon request of any person.
15. Any excavated material that will not be used as backfill must be disposed of in a lawful manner outside of any regulated floodplain, open water, freshwater wetland or adjacent transition areas, and in such a way as to not interfere with the positive drainage of the receiving area.
16. During the course of construction, neither the applicant nor its agents shall cause or permit any unreasonable interference with the free flow of the stream by placing or dumping any materials, equipment, debris or structures within or adjacent to the stream corridor. Upon completion or abandonment of the work, the applicant and/or its agents shall remove and dispose of in a lawful manner all excess materials, equipment and or debris from the stream corridor and adjacent lands.
17. All terms and conditions of the permits/waivers shall be adhered to.
18. The contractor shall comply with the state's underground facility protection act and notify the state's one call system and identify itself as the state's contractor and specify the route and mile point of the structure before performing work on the project. The one call system can be reached by calling 1-800-272-1000.
19. After completion of construction, temporary transition impact area/disturbed area shall be cultivated to a depth of 6 inches and restored to its original condition.
20. Any disturbed area that is to be left exposed for more than 30 days, and not subject to construction traffic, shall immediately receive a temporary seeding and straw mulching.

## **APPENDIX IV: TECHNICAL SPECIFICATIONS**

### **Guidelines**

1. For riprap, filter and geotextiles, see NJDOT Standard Specifications.
2. For gabions, see the specifications listed below.
3. For other types of armoring, obtain the latest set of specifications from the vendors and include as Special Provisions.

### **GABIONS**

#### **Description**

This work shall consist of the construction of gabion scour protection for slopes and channels.

#### **Materials**

- Gabions shall conform to Subsection 919.01.
- Coarse aggregate shall conform to Subsection 901.04.
- Stones for gabions shall conform to Subsection 901.16. However, the stone size shall be such that a minimum of two layers of stone is achieved in a full gabion. Gabion stone shall be 3 inches minimum to 8 inches maximum in diameter.
- Other materials shall conform to Subsection 616.02.

#### **Construction Requirements**

The slopes and ground surface shall be prepared according to Subsection 616.03. Excavation and backfill shall conform to Section 205. Native streambed material to be used as backfill shall be free of debris such as wires, cables, pieces of concrete and tires.

At least twenty calendar days before clearing site, a detailed plan of operation shall be submitted to the Engineer for review and approval. The plan should include a sequence of work; methods of construction and the procedures that will be used to comply with permit conditions. A separate plan for each site should be developed.

Gabions shall be constructed by assembling and filling gabion baskets on prepared slopes or channel bottoms upon which has been placed a layer of coarse aggregate No. 57 and geotextile fabric. The geotextile and coarse aggregate shall be constructed in accordance with Subsection 616.07.

The gabion baskets shall be assembled individually on site by erecting the sides, ends and diaphragms ensuring that all panels are in the correct position and the tops of all sides are satisfactorily aligned.

All kinks and bends shall be flattened. The four corners of the unit shall be connected first, followed by the internal diaphragms to the sides. All connections shall be accomplished using lacing wire or fasteners. Each length of lacing wire shall be 1.5 times the length to be laced but shall not exceed 5 feet.

For all connections made with lacing wires, one end of the lacing wire shall be secured by looping or twisting the end through the mesh openings on one side of the connection. After securing the end, the wire shall be laced with alternating single and double loops every other mesh opening (approximately every 4 inches to 6 inches) and the lacing end of the wire shall be securely fastened by looping and twisting the wire through the mesh openings.

The installation of fasteners shall be in accordance with the manufacturer's recommendations.

After initial assembly, the baskets shall be placed in their final position without damaging the geotextile fabric or the protective sheathing or coating of the baskets. The adjoining empty baskets shall be securely joined together using lacing wires or fasteners along the vertical and top edges of their contact surfaces. Gabions shall be placed and filled within the cofferdam in areas free of standing or running water.

Where directed by the Engineer, the basket mesh shall be cut, folded and wired together to suit site conditions. The mesh shall be cleanly cut and the surplus mesh removed or folded back and neatly wired

to the face of an adjacent gabion. The cut edges of the mesh shall be securely joined together using lacing wires or fasteners.

After the baskets are in place and joined together, they shall be stretched to effective alignment without deformation. Stretching and aligning the baskets shall be performed after several empty baskets are in place. The first basket in the line shall be partially filled with stone to provide the necessary anchorage. Stretching shall be performed using a come-along or other means of at least one-ton capacity.

Vertically stacked baskets shall be securely connected to the lower baskets (after the lower baskets are filled with stone) along the front and back edges of the contact surface using lacing wires or fasteners.

Open spaces between the placed stones shall be filled with smaller stones of the same type and quality as the gabion stone to minimize voids. Each cell shall consist of a minimum of two layers of stone. Stones at exposed faces of vertical structures shall be stacked by hand to provide a neat compact appearance. Care shall be taken when placing the stones to avoid damaging the protective sheathing or coating on the baskets.

Cells shall be filled in stages to avoid deformation of the baskets. At no time shall a cell be filled to a depth exceeding 12 inches more than adjoining cells. The cells shall be filled one inch to two inches above the level of the sides. Vertically stacked gabions shall be backfilled simultaneously with the gabion filling operation.

Gabions 3 feet high shall be filled in three layers, 12 inches per layer. Two connecting wires shall be installed after the placement of each layer. Gabions 18 inches high shall be filled in two layers, 9 inches per layer if the gabions are stacked vertically. Two connecting wires shall be installed after the placement of the first layer. Connecting wires shall connect the exposed face of the cell to the opposite side of the cell. An exposed face is any side of a cell that will be exposed or unsupported after the structure is completed. All connecting wires shall be looped around two mesh openings and the wire terminals shall be securely twisted to prevent loosening. Lacing wire or stiffeners shall be used as connecting wires.

After filling the baskets, the lids shall be pulled tight using a lid closer until the lid meets the perimeter edges of the basket. The lid shall be tightly laced and/or fastened along all edges, and the tops and ends of diaphragms. Baskets shall be well packed and filled without undue bulging and provided with secure lacing.

Gabions placed on slopes steeper than 1:2 shall be staked in accordance with the manufacturer's recommendations.

The upstream edges of gabions shall be rounded to prevent debris from snagging during high water events.

Any excavated material that will not be used as a backfill shall be disposed of according to Subsection 202.12.

**Method of Measurement.**

Gabions for scour protection will be measured by the cubic yard.

**Basis of Payment.**

Payment will be made under:

<i>Pay Item</i>	<i>Pay Unit</i>
GABIONS, SCOUR PROTECTION	CUBIC YARD

Separate payment will not be made for geotextiles and coarse aggregate placed under gabions. All costs thereof shall be included in the prices bid for the Pay Item "Gabions, Scour Protection".

Separate payment will not be made for excavation or backfill associated with gabion construction and channel grading except as related to management of high acid producing soil as specified at the end of Section 202 of the Special Provisions and except backfill required according to Section 204. Otherwise, all costs thereof shall be included in the prices bid for the Pay Item "Gabions, Scour Protection".

Separate payment will not be made for the rock excavation associated with the gabion construction. All costs thereof shall be included in the prices bid for the Pay Item " Gabions, Scour Protection".

Separate payment will not be made for removing and reconstruction of existing riprap for the purpose of extending edge protection/toe protection or gabion into it.

Separate payment will not be made for disposal of any excavated material that will not be used as a backfill except as related to management of high acid producing soil as specified at the end of Section 202 of the Special Provisions. Otherwise, all costs thereof should be included in the prices bid for the Pay Item " Gabions, Scour Protection".

Separate payment shall not be made for placing a minimum six-inch thick native streambed material cover over the gabion as shown on the plan. All costs thereof should be included in the prices bid for the Pay Item " Gabions, Scour Protection".