CHAPTER 11

Bridge-Scour Prevention and Countermeasures

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11.1 INTRODUCTION

Bridge-scour countermeasures are methods to protect bridges from scour and channel instability. Countermeasures include specific protection for piers and abutments, such as riprap, gabions, and other alternatives to riprap, the construction of guide banks at the ends of approach embankments encroaching on wide floodplains, grade control structures such as rock weirs, channel bank protection such as groins, and channel improvements such as channel straightening at the bridge site. The various bridge-scour countermeasures are described and categorized by scour type in Table 11-1. Design information for rock riprap is included in Appendix B.

The need for countermeasures can be avoided or reduced by appropriate bridge design. Good design practice comprises both the selection of a crossing site to reduce the likelihood of excessive scour and the design of the foundations and bridge superstructure to minimize the total depth of scour at the chosen site.

The results of a survey of bridge authorities in the United States conducted in 1995 (Parker et al. 1998) are shown in Table 11-2. The survey included more than 220,000 bridges and revealed that scour countermeasures were employed at 36,432 sites. Monitoring of scour depths was included as a form of countermeasure. Excluding monitoring, rock riprap is the most common countermeasure. Other commonly used countermeasures are rock gabions, extended footings, concrete pavement, grout-filled bags, and spurs. Table 11-2 lists a number of nonstructural countermeasures, including monitoring of scour depths, bridge closure, use of alarms, and imposing restrictions on vehicle use.

This chapter principally addresses countermeasures for local scour at bridge abutments and bridge piers, as well as countermeasures for general scour or channel degradation. The discussion is focused on commonly encountered countermeasures, especially riprap.

11.2 SCOUR PROCESSES

11.2.1 Mechanisms of Local Scour around Piers

The mechanics of flow at bridge piers is driven by strong pressure gradients caused by the stagnation of the flow on the upstream side of the pier, coupled with the nonuniform velocity distribution of the approaching flow, edge effects on the sides of piers, and flow expansion on the downstream side of the pier. The nonuniform velocities and pressures create flow separations and several three-dimensional vortex systems, which are scour-producing features that can fluctuate dramatically in size and intensity. Figure 11-1 shows a schematic representation of the dominant flow features at a rectangular pier.

The strong pressure gradient induced by the pier and the vertical velocity gradient causes a three-dimensional boundary-layer separation upstream and a system of three-dimensional vortices known as the horseshoe vortex system (Dargahi 1987; 1989). The nonuniform stagnation pressure zone on the upstream side of the pier forces high-velocity surface flow downward, where it impinges on the streambed at the base of the pier and rolls up into a horseshoe vortex system that is eventually carried to the pier sides. Under many flow conditions, the deepest scour has been observed to form under the area of flow impingement beneath the horseshoe vortex system (Melville and Coleman 2000).

A feature similar to the horseshoe vortex forms on the water surface upstream of the pier. The momentum gradient caused by the reduction in density at the air/water interface, in combination with the adverse pressure gradient of the pier, forms a flow reversal near the water surface. The general rotation in the surface roller is opposite to the rotation of the horseshoe vortex system. In addition, the deformation of the free surface by the roller instigates a wave that emanates from the pier nose. For relatively shallow flows, a weakening of the horseshoe vortex at the base of the pier has been attributed to the interaction of the surface roller.

Table 11-1 Bridge-Scour Countermeasures

Scour type	Measures	Purposes	Examples	Description
Channel instability – bank erosion	Armoring devices (revetment)	Prevention of erosion to the channel bank in the vicinity of the bridge; stabilization of channel alignment	Rock riprap	Dumped or placed broken rock
			Artificial riprap	Alternatives to rock riprap, including tetrapods, toskanes, akmons, and dolos
			Gabions and Reno mattresses	Wire-mesh baskets and mattresses filled with loose stone
			Precast concrete blocks	Concrete blocks of a cellular shape, possibly interlocking
			Cable-tied blocks	Concrete blocks or slabs interconnected with steel cables
			Grout-filled bags	Fabric bags filled with concrete
			Vegetation	Trees, shrubs, grasses
			Used tires	Used tires placed as a mattress or stacked against a bank
			Grouted riprap, concrete apron, grout-filled mats	Rigid revetments, grout-filled mats (fabric bags filled with concrete), and grouted riprap
	Flow-retarding devices	Reduction of flow velocity near the channel bank and inducement of deposition of sediment	Timber piles, sheet piles, Jack or tetrahedron fields	Permeable structures in a channel, generally placed parallel to the bank
			Vegetation planting	Trees planted to control bank erosion
	Flow-training devices	Reduction of flow velocity near the channel bank and inducement of deposition of sediment; stabilization of channel alignment	Groins (also known as spurs, dikes, jetties, wing dams, or deflectors)	Permeable or impermeable structures, projecting into the flow
			Hardpoints	Small spur-like structure of stone fill spaced along an eroding bank line
			Bendway weirs	Small spur-like structures, typically submerged at normal water levels, spaced along an eroding bank line
			Iowa vanes	Vertical plates installed in a streambed designed to break up secondary flow and mitigate the tendency to lateral migration of banks
Degradation and contraction scour	Check dams	Control of channel grade	Low dams or rock weirs	Constructed across the channel width
	Channel lining	Control of vertical erosion	Concrete or bituminous-concrete pavement	Reinforced-concrete or bituminous-concrete pavement covering channel bed and banks
			Boundary-armoring measures above	For contraction scour and not degradation

(Continued)

Table 11-1 Bridge-Scour Countermeasures *(Continued)*

Degradation and contraction scour	Bridge waterway area	Increase of bridge opening size or efficiency	Channel widening, relief bridges, or guide banks	
Aggradation	Channel improvement	Increased sediment transport to reduce sediment deposition at bridge crossing	Dredging, clearing of channel	
			Formation of a cutoff	
			Flow-control structures	
	Controlled mining	Reduction in sediment input at bridge site	Mining of bed sediment	
			Bar mining	
	Debris basin	Reduction in sediment input at bridge site	Debris basin	Constructed to trap sediment
Local scour	Armoring devices	Reduced local scour	Rock riprap	Dumped or placed broken rock
			Artificial riprap	Alternatives to rock, including tetrapods, toskanes, and akmons
			Gabions and Reno mattresses	Wire mesh baskets and mattresses filled with loose stone
			Cable-tied blocks	Concrete blocks or slabs interconnected with steel cables
			Grout-filled bags	Fabric bags filled with concrete
			Grouted riprap, concrete apron, grout-filled mats	Rigid revetments
	Flow-altering devices	Reduced local scour at piers	Sacrificial piles Iowa vanes	Piles or vanes placed upstream of bridge pier(s) to deflect flow away from the piers
			Horizontal collars	Thin horizontal plates attached to the base of the pier, to deflect flow away from the sediment bed
	Foundation modification	Reduced local scour	Underpinning	Extending bridge foundations to lower levels
			Extended footing	Slab footing to piers, which can inhibit local scour
	Guide banks	Improved flow alignment at bridge crossing; reduction in local scour at abutments		Straight or outward-curving structure, extending upstream from the end(s) of the approach embankment

Table 11-2 Distribution of Types of Scour Countermeasure in a 1995 U.S. Survey (after Parker et al. 1998)

Countermeasure	Number	Percentage
Dumped riprap	5,913	16.23
Self-launching riprap	72	0.20
Rock gabions	567	1.56
Other flexible revetment	37	0.10
Pavement	253	0.69
Grout-filled bags	97	0.27
Concrete-grouted riprap	27	0.07
Concrete-filled mat	51	0.14
Tetrapods	1	0.003
Extended footings	778	2.14
Cable-tied blocks	6	0.02
Vanes (pier or bed)	1	0.003
Sacrificial piles	22	0.06
Flow-direction plates	6	0.02
Jetties	43	0.12
Spurs	420	1.15
Retards	35	0.10
Check dams	83	0.23
Rock bank protection	79	0.22
Soil cement	7	0.02
Increase bridge span	2	0.005
Brace piles in transverse direction	5	0.01
Monitoring	27,770	76.22
Alarms	22	0.06
Bridge closure	111	0.30
Vehicle restriction	0	0.00
Other	24	0.07
Total	36,432	100.0

Three-dimensional spiral-edge vortices form downstream of flow separation lines on the corners of rectangular piers, on the sides of cylindrical piers, and at the upstream and downstream edges of round-nosed piers skewed to the flow direction. The vertically oriented vortices remain attached to the streambed just downstream of the separation from pier corners. These tornado-like flow structures transfer flow and sediment from the streambed upward and may be a primary mechanism of removal of dislodged sediment from scour holes. For clear-water scour conditions, Hjorth (1975), Parola (1993), and others reported that the primary mechanism of initial failure of armor protection was related to the flow at the edge separation points on rectangular piers, where conditions have been reported to be as much as an order of magnitude higher than those of the approaching flow.

Wake vortices form in response to the adverse pressure gradient of flow expansion, along with the highly nonuniform flow that is created along the shear flow zone on the pier sides. These vortices dominate the flow structure downstream of piers.

The pressure gradient that causes formation of the horseshoe vortex also deflects flow to the pier sides. The vertical nonuniformity of the boundary-layer flow approaching the pier in the presence of the stagnation pressure gradient causes a secondary flow throughout the flow depth as flow passes around the sides of the pier. Where the stagnation pressure gradient is relatively weak (wide piers in shallow flows), the vortical flow developed from flow curvature may be the most important feature in the formation of scour holes.

These large-scale vortical flow structures combine to increase the sediment entrainment and transport capacity by increasing near-bed flow velocity, turbulence levels, vorticity, and seepage gradients. In locations where surface water impinges on the streambed, near-bed streamlines and the sediment carried along them are deflected away from the region of the pier. Protection placed in this environment must resist the forces generated by these mechanisms and undermining by seepage and winnowing.

11.2.2 Mechanisms of Local Scour around Abutments

Although conceptually the pressure gradients and vertical nonuniformity that create the vortical flow at piers are also present at abutments, many factors associated with the generally large lateral extent of the flow field disturbed by abutments become important. In particular, the lateral extent of the flow disturbance is typically much larger than the flow depth. In contrast to the relatively uniform approach flow at piers, floodplain and main-channel geometry and roughness cause flow nonuniformity. The nonuniform lateral velocity distribution and extensive upstream adverse pressure gradients create upstream horizontal flow separation and recirculation zones. In laboratory experiments, where relatively low-roughness floodplains have typically been simulated, large separation regions called dead-water zones have been reported to form. Although the extent of the pressure gradients is larger, they are general much weaker than at piers. As a consequence, the dominant large-scale vortical structures include

- the flow curvature upstream that causes the primary vortex (see below) to form,
- the spiral edge vortices that form at flow separation from the edges, and
- the wake vortex systems.

Where abutments extend into the main channel or where bends force high-velocity flows against abutments, strong pressure gradients similar to those at piers may develop.

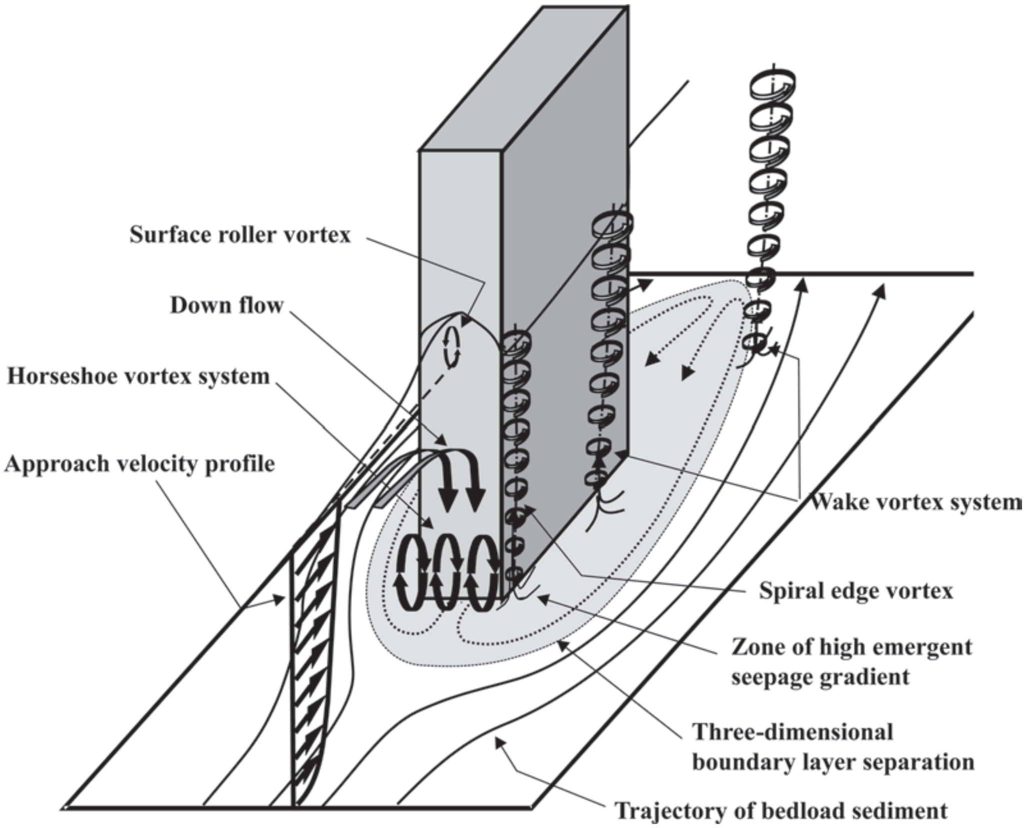

Fig. 11-1. Flow structures at a rectangular pier (modified from Parola 1995).

The pressure gradients caused by the flow at abutments force flow to contract toward the bridge opening. For short abutments that project into high-velocity flow, strong pressure gradients, similar to those at piers, cause a vortex to form that is similar to the horseshoe vortex. For relatively long abutments in relatively shallow flows, pressure gradients may be weaker; therefore, long vortices with their axes in the streamwise direction develop. The strongest of these vortices has been called the primary vortex (Kwan 1984). Although the pressure gradients induced by abutments are typically not as severe as those at piers, the effects of the pressure gradients are more extensive, causing flow curvature that extends far upstream of a bridge crossing. The flow curvature causes the formation of longitudinal vortex systems. Spiral-edge vortices, similar to those created at piers, form along the vertical edges of vertical-wall abutments and along spillthrough abutments at the point of flow separation. Initial scour-hole formation frequently is initiated at these locations. The initial movement of rock protection has been reported to occur at these locations in model studies by Pagan-Ortiz (1991).

11.2.3 Effects of River Morphology Development and Channel Contraction on Bridge Scour

Bridge-scour processes occur over a range of scales, from the local-scour scale around individual foundations described above, to the catchment scale extending to the limits of the catchment of the flow through the bridge section. In addition to local scour arising directly from the presence of individual bridge foundations, scour at a bridge site can arise due to contraction of flow width for the waterway section associated with the bridge structure (contraction scour), and also general river processes (general scour), including the respective processes of aggradation; degradation; scour in channel bends and channel confluences; scour due to the movement of both the channel thalweg and waves in the channel bed sediments; and lateral erosion arising from general bank erosion, channel widening, and channel migration processes. The total scour at a bridge site is given by the combination of the relevant components of general, contraction, and local scour.

Contraction scour can occur wherever the waterway section is laterally contracted by either natural channel morphology, such as a narrow neck in the river (a common bridge location), or imposed structures such as bridge foundations and associated road approach embankments. The width reduction causes increased flow velocities and bed shear stresses through the section, potentially increasing scour across the site as a whole. Analogously to local scour occurring at a bridge abutment, additional localized scour also typically occurs along boundaries at the entrance to a contraction where the flow is nonuniform. In practice, contraction scour and local scour can occur together and it may be difficult to distinguish between them.

Aggradation, involving the building up of bed levels, can be ascribed to sediment supply generally exceeding sediment transport capacity or to a raising of the base level. Degradation, the lowering of bed levels over a region larger than the immediate vicinity of the bridge site, is conversely the result of sediment transport capacity generally exceeding sediment supply or a lowering of the channel base level. With sediment supply for a reach provided by erosion of the catchment and waterway, and sediment transport capacity a function of sediment size, channel size and discharge, degradation and aggradation are strongly influenced by changes in hydraulic regimes, geomorphic channel controls, and also catchment land uses, including changes in mining, deforestation, agricultural, urbanization, and river management practices within the catchment. Both aggradation and degradation can proceed in upstream or downstream directions, and both can induce associated lateral channel instability. It is important to recognize that cyclic aggradation and degradation responses of bed levels can follow from a disturbance to a channel system.

For most flows inducing general sediment motion at a bridge site, sediment waves will be migrating through the site. Waves in sand beds are commonly classified as ripples, dunes, antidunes, or chutes, and pools. In gravel-bed rivers, waves occur mostly as gravel bars moving down the river. With heights of migrating dunes and bars in natural alluvial channels potentially up to the order of the mean flow depth (Melville and Coleman 2000), the passage of these waves can potentially influence bridge scour significantly with wave troughs temporally and locally lowering bed elevations as the waves propagate through a site. Bed roughnesses determined by these waves also significantly affect stage-discharge relationships during the passage of floods.

Lateral movement of the channel thalweg (the line of lowest bed elevation along the channel) is a natural process that alters local bed elevations, inducing scour, and can change the point and/or angle of attack for a flow at a bridge site. Bridges need therefore to be designed for the potential influence of the thalweg occurring in the vicinity of each foundation.

At a confluence of river channels, the individual streams typically meet toward the centerline of the confluence, plunge to the channel bed, and then return to the water surface along the sides of the confluence. The induced helicoidal secondary currents, similar to such currents formed in river bends, result in a deep scour hole with steep sides. Channel confluences can form randomly in time and space for braided reaches, with the resulting scour holes potentially reaching depths below the surrounding bed of up to five times the mean flow depth in the converging channels. Such scour holes have been noted to be contributing factors in a number of bridge failures (Coleman et al. 2000; Coleman and Melville 2001).

For flow around a curved reach or bend, the interaction between the vertical gradient of streamwise velocity and the curvature of the primary flow generally produces secondary currents leading to greater flow depths, velocities, and shear stresses at the outside of the bend. These result in channel deepening at the outside of the bend (bend scour) and, in conjunction with concurrent undermining of the outside stream bank, increased lateral erosion at the outside of the bend.

Lateral erosion, reflecting the dynamic nature of channel planform, can have significant consequences if not allowed for in the design of static bridge structures. In addition to being associated with bend scour, lateral erosion can be in the form of general channel-bank erosion, channel widening, or channel shift for meandering and braided rivers. The relative importance of lateral erosion processes is reflected by the conclusion of Simon (1994; 1995) that width adjustment processes may represent the dominant mode of morphology adjustment for rivers. Certainly, Parola et al. (1996) indicate lateral channel instability to be one of the most common factors underlying excessive bridge and abutment scour and approach endangerment.

General channel-bank erosion can result from weathering mechanisms such as freeze-thaw and desiccation, seepage effects, surface runoff, erosion by current flow, the action of waves, the sediment-transport capacities of flows exceeding the potential supply of sediment from the channel bed, and mass failure mechanisms for banks, including sliding along a deep failure surface, shallow slips, and block failures.

Channel widening can accompany aggradation as flows seek to increase in width when the bed aggrades and flow depths decrease. Degradation can also result in channel widening, owing to the removal of toe support for the river banks, or resulting from increased excess pore pressures on the declining limbs of flood hydrographs causing bank failure.

For meandering and braided (including anabranched) rivers, incremental channel shift is inherent. Meander migration is typically directed outward and downstream and is typically a relatively slow and somewhat methodical process. For braided reaches, dramatic channel avulsion can occur in the course of a single flood, particularly for rivers with little vegetation within the floodplain. Incremental channel shift can be exacerbated by human activities such as land-use practices, gravel mining (Coleman and Melville 2001), and the removal of riparian vegetation.

Where bank erosion is a significant source of floating debris, lateral erosion can also lead to increased floating debris loading on bridges in the river, exacerbating any local scour at bridge foundations.

In regard to the overall assessment of potential scour magnitudes for a bridge, analyses of case studies of scour-induced bridge failure (Coleman and Melville 2001) indicate that ranges of combinations of potential scour components need to be considered—for example, solely pier scour, or pier scour combined with bend and contraction scours. Floods on the order of bank-full flows should be considered to assess channel lateral migration and the influence of countermeasures on long-term lateral and vertical stream stability. Countermeasures should be designed to provide transport of sediment and debris through the bridge during bank-full events that is similar to that of the upstream and downstream reaches to prevent degradation or aggradation and lateral shift within the bridge opening. Case studies of scour-induced bridge failure (Coleman and Melville 2001) highlight, however, that both bridge-scour vulnerability and countermeasure design also need to be assessed for the occurrence of minor floods made more critical by present or potential river morphology at the bridge site.

11.3 PROTECTION AGAINST GENERAL SCOUR AND CONTRACTION SCOUR

11.3.1 Site Selection

The characteristics of a river can change considerably over short distances. Where multiple choices of bridge site are available, the following factors should be considered, although economic considerations may dictate selection of the shortest crossing point. Generally, sites exhibiting evidence of channel instability including degradation and aggradation, lateral movement and bank erosion, and hydraulic problems at other bridges in the area need to be assessed carefully.

11.3.1.1 Catchment Influences In bridge site selection, the potential influence on the site of changes within the catchment should be assessed, including history and patterns of water levels, flood magnitudes, earth flows, landslides, volcanic or earthquake activity, channel bars, channel confluence location, channel morphological controls, bank erosion, degradation, aggradation, and lateral channel instability. In particular, changes in human activities within a catchment can potentially influence vertical and lateral channel stability for the catchment. Such activities include agriculture, vegetation clearing, forestation, strip mining, urbanization, dam construction or removal, stream-bed mining or dredging, channel clearing, and channel realignment or containment.

11.3.1.2 Alluvial Fans Alluvial fans are inherently unstable, channels on the fan being potentially subject to rapid aggradation, degradation, and shift in channel location. Thus, bridges located on alluvial fans may be subject to continual problems due to channel instability. It is normally better to select a location at the apex of the fan where the channel is relatively more stable.

11.3.1.3 Influence of Channel Curvature Potential variation in river planform needs to be allowed for in the design of bridge foundations. Bridge locations on stable straight reaches or gentle bends, or at positions of geologically stabilized morphology, are often preferable. In meandering rivers, the choice is between a location at a bend or at a crossover point, although in some cases, the ideal location of a nodal point may exist, where the river has been flowing permanently irrespective of past river alignment changes. At a crossover the channel is wider, but may be more stable laterally than at a bend, where the channel is typically narrower and deeper. Bridge locations at stable bends necessitate designing the piers on the outside of the bend for the deepest channel scour, whereas the piers on the inside of the bend can be designed for less scour. If the bend can migrate, then all foundations need to be designed for the maximum scour in the bend. In straight reaches, the point of deepest scour can shift from side to side so that all piers need to be designed for the maximum scour. Foundations in the floodplain of a river need to be placed at the same level as those in the main channel if analyses indicate that the main channel can potentially move across the present floodplain.

11.3.2 Bridge Waterway Area

Wide, unimpeded bridge waterways are preferable. Constricted waterways induce contraction scour. The constriction can be lateral, due to the bridge foundations, or vertical, due to the bridge superstructure becoming submerged. It is important to ensure that the bridge is designed with adequate clearance between the maximum water level and the lowest level of the superstructure (including allowance for potential debris accumulations) to avoid superstructure submergence. In addition, it may be desirable to have the approach roadways at a lower level than the underside of the bridge superstructure, this allowing floodwater to overtop the approaches without intercepting the superstructure. The consequences of flow contraction can also be relieved by the construction of auxiliary (relief) bridges for floodplain flow.

In the design of new and replacement bridges where river morphology or existing structures severely contract flow, additional floodplain spans and relief structures should be considered. In the design of a replacement for a bridge structure that may be providing grade control, any increase in the bank-full flow capacity of the bridge opening may initiate degradation upstream and may thereby warrant the installation of appropriate grade control structures at the site to prevent such progressive degradation.

11.3.3 River Training Works

The cost of river training works at bridge sites can be significant. Training works are not normally required in relatively straight and stable reaches, where sites having the narrowest main channels and the smallest proportion of floodplain flow are preferable. In less stable rivers, sites requiring a minimum of river training are often preferable. For example, sites may be found where rock outcrops or other controls effectively limit lateral movement of the river channel.

11.3.4 Bank Protection

Lateral instability through general channel-bank erosion, bend scour, channel widening, or channel shift can result in erosion of abutments, breaching of bridge approaches leading to the bridge being outflanked, and scour of bridge foundations located outside of the main channel. Where bank failure is by rotational slip, lateral pressures on bridge foundations within the slip zone can further result in displacement or cracking of the foundations.

Lagasse et al. (2001) indicate that impermeable longitudinal stone dikes parallel to the bank line, or smaller rock toe-dikes, provide the most effective toe protection of all bank stabilization measures studied for very dynamic and/or actively degrading channels, the authors presenting design procedures for these dikes. Aside from such bank stabilization measures, measures to counter lateral erosion act to armor the boundary or retard or train the flow. Flow training measures include groins, hardpoints, bendway weirs, and Iowa vanes. The following focuses on countermeasures that are more commonly encountered or that hold potential.

11.3.4.1 General Comments on Armoring Measures Armoring measures, also known as revetments, are channel linings used to provide erosion-resistant surfaces. They can be flexible or rigid. Flexible revetments include rock riprap, artificial riprap (including akmons, dolos, and tetrapods), broken concrete, used tires, grout-filled bags, precast blocks, cable-tied blocks, gabions and Reno mattresses, and vegetation. Rigid revetments used include concrete pavement, grouted riprap, concrete-filled fabric mat, and cement-stabilized soil. Flexible revetments have the advantage of being able to adjust to local displacement of underlying materials without complete failure of the installation, although such deformations tend to be limited for mattresses of used tires or precast blocks. All revetment types must be designed to protect against

- slumping due to over-steepened slopes,
- undermining due to inadequate toe protection, and
- outflanking due to inadequate lateral coverage.

Any hardening of the outer bank of a bend to counter lateral erosion can potentially exacerbate vertical erosion in the bend by preventing the supply of sediment from bank erosion. There is evidence, however (Harvey and Sing 1989; Thorne et al. 1995), that hardening of the outer bank has no effect on maximum flow depth in the bend.

11.3.4.2 Rock Riprap and Broken Concrete Rock riprap is the measure most commonly used to protect banks from erosion. Guidelines and principles for the use of riprap as bank protection are given in Appendix B. Broken concrete has been used as an alternative to rock riprap in emergencies where rock of suitable sizes and quality has not been readily available.

11.3.4.3 Artificial Riprap, Grout-Filled Bags, Precast (including Cable-Tied) Blocks, and Used Tires Artificial riprap, typically fabricated of reinforced concrete in the form of standard units, may become cost-effective where rock riprap of a required size and quality is not readily available. Prefabricated units such as dolos, tetrapods, tetrahedrons, and toskanes (Fig. 11-2) are designed to give maximum interlocking, and thereby maximum protection of the underlying surface, using a minimum amount of material. Artificial riprap is widely used in the coastal environment and for riverbank protection. The most common mode of failure of these armor units is edge failure. Each form of armor unit will have unique design criteria, and reference should be made to the unit manufacturer. Specifications for tetrapods and toskanes are given in USACE (1984). Filter layers or bedding material may be required to achieve the desired hydraulic performance.

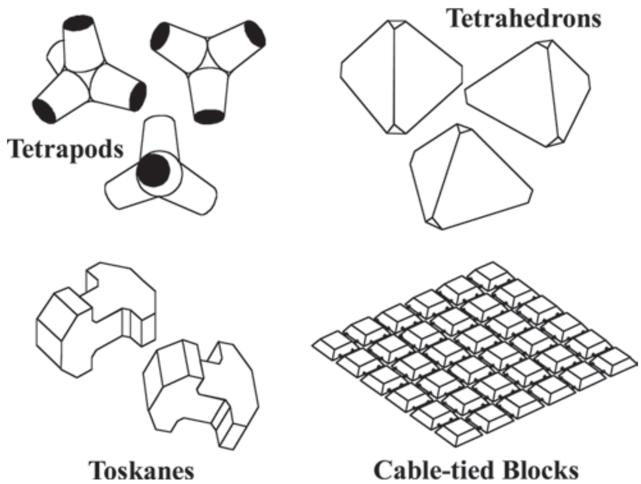

Fig. 11-2. Example of flexible armoring measures.

Stacked grout-filled bags form a flexible armoring countermeasure. These bags, however, provide little interlocking, with or without a geotextile, and are thereby subject to sliding and dispersion, leading to failure of the armoring measure, in the presence of degradation or a dune field. Lagasse et al. (2001) observe that grout-filled bags are installed only where rock of suitable size and quality has not been readily available. They comment that engineering judgment is typically used to select a bag size that will not be moved by channel flows. Installation practices critical to the success of grout-filled bag systems are discussed in Lagasse et al. (2001). Bags filled with sand instead of grout may offer additional advantages, including increased countermeasure flexibility, and possibly better interlocking.

Cable-tied blocks are flexible mats of interconnected smaller units (Fig. 11-2), typically concrete blocks or slabs interconnected with steel cables. The flexibility of the mat enables settlement of the mat edges, facilitating self-anchoring of mats in sand-bed streams. The blocks can be constructed in units with a preattached geotextile. The interconnected nature of the blocks allows stable scour protection to consist of smaller block units than for loose riprap. Failure modes for cable-tied blocks are found to include overturning and rolling up of unsecured leading edges and uplift of inner mats at higher velocities.

Lagasse et al. (2001) cite favorable reports of the performance of precast concrete blocks, where vegetation growing between the blocks can improve the appearance and stability of the countermeasure. They observe that each form of preformed block unit, including those that interlock and those held together by steel rods or cables, will have unique design criteria that should be available from the block manufacturer, these criteria having been formulated to ensure that intimate contact between the revetment and the protected subgrade is maintained under the desired hydraulic conditions. Lagasse et al. (2001) note that the significant influence of block protrusion into the flow on block stability necessitates construction inspection to ensure that blocks are installed within design tolerances.

Revetments of used tires in lieu of rock riprap have been successfully used for flow velocities up to 3 m/s on mild bends. These revetments can accommodate minor bank subsidence, but are somewhat unsightly and vandalism-prone and are typically expensive, owing to construction being labor-intensive. To aid revetment stability, tires should typically be tied together, with the revetment edges tied to the bank. They can also be packed with rock. In addition, the tires should fit together well; they can be assisted by vegetation planted in the tires (also aiding aesthetics); and they should resist uplift by being anchored to the bank at intervals and by having the sidewalls pierced to prevent flotation.

11.3.4.4 Concrete-Grouted Riprap, Concrete Pavement, and Grout-Filled Mats Concrete-grouted riprap is relatively cost-effective, making possible the use of rock of smaller sizes and wider gradings, although it scores poorly in terms of aesthetics and environmental acceptability. The decreased countermeasure flexibility negates the natural benefit of riprap being able to deform and armor developing scour, caused by toe undermining or bank settlement, for example. The reduced permeability of the armor layer arising from the grouting, although decreasing the need for filter layers beneath the countermeasure, is disadvantageous in that uplift from turbulence and confined groundwater can lead to failure of the rigid riprap layer in entirety.

Concrete pavement is similarly subject to problems of aesthetics, environmental acceptability, and susceptibility to uplift pressures. Weep holes can be used for relief of hydrostatic pressures for both concrete-grouted riprap and concrete pavement. Partially grouted riprap (in conjunction with underlying filter layers) can be used as a compromise measure, the grout acting to increase the stability of riprap installations without sacrificing all of the flexibility or pore-pressure-drainage advantages of loose riprap.

Grout-filled mats are continuous layers of fabric with pockets or cells that are filled with concrete. These mats, typically strengthened with cables, form a monolithic armoring countermeasure that is taken to be rigid in action. Grout-filled mats face the same problems as concrete-grouted riprap and concrete pavements, although porous mats may act to relieve hydrostatic pressures. Lagasse et al. (2001) present analyses of the hydraulic stability of these mats that make possible determination of mat thickness for a desired factor of safety against sliding of the unanchored mat.

In addition to potential failure due to uplift, these rigid revetments are subject to undermining by both hydraulic action (at the toe, the upstream and downstream edges, and also the upper edge if overtopped) and channel degradation. Grout-filled mats can also fail by overturning and rolling up of an unsecured leading edge, or uplift of the inner mat at higher velocities.

11.3.4.5 Gabions and Reno Mattresses Rock-and-wire gabions and mattresses comprise wire-mesh baskets and mattresses filled with loose stone, often connected together and often anchored to the channel boundary.

In comparison to more solid countermeasures, gabions are less susceptible to uplift forces, owing to the porous nature of the loose-rock fill material. In addition, should the countermeasure installation become unstable, the flexibility of the wire mesh enables gabions to mould themselves somewhat to restore stability of the installation. Gabions and Reno mattresses further allow the use of smaller rock than used for standard riprap protection (Simons et al. 1984), and can be used to protect steeper slopes, although gabions and mattresses are more expensive.

Damage to the wire mesh is a major reliability problem for gabions and Reno mattresses, potentially resulting in failure of individual gabions or even complete failure of the countermeasure installation as a whole. Wire damage may be from long-term corrosion or from abrasion due to the movement of either contained rock in highly turbulent flows or passing sediments in floods.

Design criteria for gabions can be determined from the unified formula (Table 11-3) of Pilarczyk (1995); this formula combines various design formulae for armoring countermeasure options. Alternatively, for stream slopes less than 2%, Maynord (1995) proposes use of the U.S. Army Corps of Engineers equation of B.11, where riprap size of which 30% by weight is finer, D_{30}, is replaced with the average filling rock diameter, D_m, which in turn is taken to be equivalent to half of a minimum gabion-basket thickness, $D_{n\min}$; that is, $D_{30} \rightarrow D_m = D_{n\min}/2$. For this procedure, a blanket thickness coefficient of $C_T = 1$ is adopted, and the stability coefficient C_s is taken to be $C_s = 0.1$. For additional detailed guidelines in regard to the materials and construction of gabions and Reno mattresses, the reader is referred to Parker et al. (1998).

11.3.4.6 Vegetation Through root action, dissipation of flow energy, and encouragement of sediment deposition, grasses and woody plants (trees and shrubs) act to armor surfaces to counter erosion and stabilize banks. Grasses can be used to protect upper banks that are subject to erosion due to rainfall, overland flow, and minor wave action. Woody plants offer better erosion protection owing to more extensive root systems. The U.S. Department of Agriculture (USDA 1996) provides U.S. guidelines for the use of vegetation as a method

Table 11-3 Unified Formula for Armoring Countermeasure Options (after Pilarczyk, 1995)

Factor		Relation
Unified formula	$\Delta D_n = \phi_c K_t \dfrac{0.035}{\theta_c} \dfrac{K_h}{K_{sl}} \dfrac{V^2}{2g}$ (11-6)	
	V = depth-averaged mean velocity	
Relative density, Δ	$\Delta = (S_s - 1)$	Rock riprap, Blocks, Blockmats
	$\Delta = (1-n)(S_s - 1)$	Gabions, Mattresses
	n = porosity of stones	
	S_s = specific gravity of stones	
Unit size, D_n	$D_n = 0.84 D_{r50}$	Rock riprap
	D_n = block thickness	Blocks
	D_n = basket thickness	Gabions, Mattresses
	D_{r50} = median size of riprap	
Stability factor, ϕ_c	$\phi_c = 1.0 \rightarrow 1.5$	Exposed edges
	$\phi_c = 0.50 \rightarrow 0.75$	Continuous protection
	$\phi_c = 0.75$	Common value for rock
Turbulence factor, K_t	$K_t = 1.0$	Normal turbulence (rivers)
	$K_t = 1.5$	Nonuniform flow with increased turbulence, Outer bends ($r/W > 2$)
	$K_t = 2.0$	High turbulence, e.g. sharp outer bends, ($r/W \leq 2$)
	r = bend radius	
	W = channel width	
Critical shear stress, θ_c	$\theta_c = 0.035$	Rock
	$\theta_c = 0.05$	Free blocks
	$\theta_c = 0.05 \rightarrow 0.07$	Blockmats, Mattresses
	$\theta_c \leq 0.10$	Rock-fill in gabions, Mattresses
Velocity profile factor, K_h	$K_h = \dfrac{2}{\log^2\left(1 + 12\dfrac{H}{D_n}\right)}$	For a fully-developed log velocity profile
	$K_h = \left(1 + \dfrac{H}{D_n}\right)^{-0.2}$	For a not-fully-developed velocity profile
	H = flow depth	
Bank slope factor, K_{sl}	$K_{sl} = \left(1 - \dfrac{\sin^2\alpha}{\sin^2\theta}\right)^{0.5}$	
	α = slope angle	
	θ = angle of repose	
Roughness height, k_s	$k_s = D_n$	Smooth units, e.g., concrete blocks
	$k_s = 1 \rightarrow 3D_n$	Rough units, i.e., rock

of streambank protection, including descriptions of principles, practice characteristics, design, construction materials, and appropriate techniques for streams, lakes, and estuaries. Willow trees have been successfully used in New Zealand (Acheson 1968) for protection of lower banks, where they establish quickly, withstand inundation, and are sufficiently dense to promote deposition of sediment. In terms of using vegetation as a countermeasure for bridge scour, care must be taken to ensure that any introduced vegetation does not adversely reduce channel capacity. In addition, Lagasse et al. (2001) recommend that vegetation not be seriously considered as a countermeasure against severe bank erosion where a highway facility is at risk.

11.3.4.7 Flow-Retarding Measures Flow-retarding measures (retards) are typically permeable structures, generally installed parallel to the bank and placed at the toe of the bank. They are thus best suited to protecting low banks or the lower portions of stream banks. They are designed to reduce flow velocity and control flow alignment, and thereby induce sediment deposition and prevent lateral erosion, creating an environment suitable for the establishment of vegetation. Retards include piles (typically timber or steel), fences, vegetation planting, and fields of jacks or tetrahedrons (with individual units possibly tied together using cables).

Key factors in the design of these measures include the availability of adequate floating debris and bed material to facilitate development of flow resistance and sediment deposition and the potential for bank revegetation to aid bank stability. The required permeability of retards is inversely proportional to the radius of curvature of the bend being protected, sharper bends requiring less permeable retards. Retards must be designed to withstand local scour processes and the potential impact of debris on structural loads and local-scour magnitudes.

Brown et al. (1981) conclude that retards are most successful for channels of widths less than about 100 m, flow velocities not frequently exceeding 1.5 to 1.8 m/s, and beds of sands with relatively large bed and suspended loads.

11.3.4.8 Groins and Bendway Weirs Groins (also referred to as spurs, dikes, wing dams, jetties, or deflectors) are structures that project from the bank into the channel (Fig. 11-3). Commonly constructed using rock, they may be permeable or impermeable and submerged or unsubmerged. They are designed to control flow alignment and reduce flow velocities near scour-threatened boundaries, thereby preventing lateral erosion, and possibly also inducing sediment deposition in the scour-threatened zone. They act through a combination of diverting flow around the structure, reducing flow along the bank as it passes through the structure, and redirecting flow passing over the weir. Groins are typically used to control meander migration. They can also be used to align wide, poorly defined streams into well-defined channels, reducing required lengths for any planned bridge crossings.

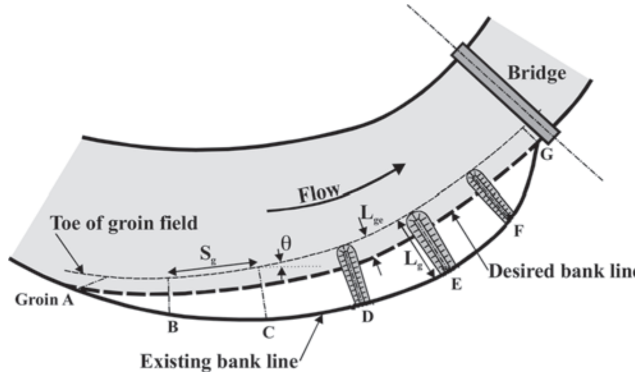

Fig. 11-3. Aspects of the design of a field of 7 groins.

The specification of standardized procedures for the design of groin fields is inappropriate owing to any given design being inherently site-specific. Some general principles, including published guidelines for groin spacing, are summarized in Table 11-4. Bendway weirs, also referred to as stream barbs, bank barbs, and reverse sills, essentially act as upstream-pointing submerged groins. Alternative guidelines (in terms of lengths, orientations, spacings, locations and numbers, heights, cross-sections, and construction) for these structures, particularly for larger structures or larger rivers, are given in Lagasse et al. (2001).

11.3.4.9 Hardpoints Hardpoints are small groin-like structures of stone fill placed along an eroding bank line. They are distinguished from groins because they protrude only short distances into the channel, typically acting individually to provide localized protection against scour. Lagasse et al. (2001) indicate that hardpoints are most effective where streamlines and bank lines are approximately parallel and velocities within 15 m of the bank line are less than approximately 3 m/s. These structures can be effective where bank erosion is mainly caused by a wandering thalweg, but close spacings required by the short lengths typically render these structures uneconomic for protection of meander bends.

11.3.4.10 Submerged (Iowa) Vanes Iowa vanes are submerged vertical plates installed in the streambed to deflect flow and control sediment deposition and erosion (Odgaard and Wang 1987). These vanes have been successfully used to control erosion in river bends (Odgaard and Kennedy 1983), ameliorate shoaling problems in rivers (Odgaard and Spoljaric 1986), and control sediment at lateral diversions (Barkdoll et al. 1999). The reader is referred to these papers, and particularly Odgaard and Wang (1991a; 1991b), for design guidelines and principles for the use of Iowa vanes in these situations.

11.3.5 Degradation, Contraction, Thalweg, and Sediment-Wave Effects

Each process of degradation, contraction scour, thalweg effects, or sediment-wave effects can lead to undermining

Table 11-4 Principles and Guidelines for the Design of Groin Fields (see Fig. 11-3)

Factor	Design criteria		
Groin length, L_g (normal to flow or bank)	Where the bank is irregular, groin length should be adjusted to provide even curvature of the thalweg Generally, $L_{ge} < 0.15W$ (W = channel width) for impermeable groins, and $L_{ge} < 0.25W$ for permeable groins, where L_{ge} = effective groin length = distance between arcs describing the toe of the groin field and the desired bank line		
Groin orientation	Orientation affects groin spacing, scour depth at the tip of the groin, and degree of flow control achieved Groins oriented normal to the flow are most economical because they provide maximum protrusion for a given groin length The first (upstream) groin should be angled downstream		
Groin spacing, S_g	$S_g = L_{ge} \cot\theta$; where θ is the flow expansion angle downstream of the groin tips ($\approx 17°$ for impermeable groins, and increasing with permeability greater than 35%, Lagasse et al. 2001)		
	Reference	S_g/L_g	*Applicability*
	Acheson (1968)	3–4	Depends on curvature and channel slope
	Ahmad (1951)	4.3	Straight channels
		5	Curved channels
	Copeland (1983)	2–3	Concave banks
	Grant (1948)	3	Concave banks
	Maza Alvarez (1989)	5.1–6.3	Straight channels
		2.5–4	Curved channels
	Neill (1973)	4	General practice
	Richardson et al. (1988)	2–6	Depends on flow and groin characteristics
	Strom (1962)	3–5	General practice
	Suzuki et al. (1987)	<4	Straight channels
	United Nations (1953)	1	Concave banks
		2–2.5	Convex banks
Groin plan shape	Straight groins are preferred Top widths of impermeable groins should be at least 1 m		
Longitudinal extent of groin field	Field and aerial surveys of the extent of scour are a good basis for determination of the necessary extent of a groin field Protection downstream of a bend is especially important because meander bends propagate downstream		
Groin height	To avoid bank overtopping, impermeable groins generally do not exceed bank height Similarly, a sloping crest height (downward away from the bank line) is advantageous for impermeable groins Permeable groins should allow floating debris to pass over, unless the design requires trapping of light debris		
Groin side slopes	Side slopes should be 2:1 (H:V) or flatter		
Groin permeability	Permeability up to about 35% does not affect the length of channel bank protected Impermeable groins give better flow control, but induce greater end scour and, if submerged, can induce bank erosion High-permeability groins are preferred for mild bends and regions requiring small flow velocity reductions		
Bed and bank contact	Adequate bed contact is necessary to avoid undermining of the groin, especially at the toe, where a launching apron is advantageous Adequate bank contact is necessary to avoid outflanking of the groin		
Erosion protection	Riprap protection to the upstream and downstream faces and the end of the groin is recommended (possibly aided by filter layers)		

of flow- and grade-control structures and bank protection, a need for bridge relocation, and bridge failure due to undermining of the foundations. Successful protection of piers and abutments involves providing adequate foundation depths, by underpinning of existing foundations if necessary, and allowing adequate setback of abutments from slumping banks where appropriate. Aside from such considerations of foundation design or modification, the countermeasures for these vertical-erosion processes act either to maintain stable bed levels through the bridge site or to ease the passage of flows past the bridge.

11.3.5.1 Check Dams Check dams are low dams or weirs constructed across the entire width of a channel. These dams act to establish a fixed grade point, maintaining bed levels at the bridge site and controlling any upstream migration of degradation. The structures are usually constructed of rock riprap (rock weirs), timber piles, gabions, concrete, or sheet piles, the first two materials being more for lower dams and channel widths less than 30 m. Typically, check dams are installed immediately downstream of the bridge they protect, although they can extend through the bridge site. For severe cases of degradation, two or more check dams in succession can be used, where a single higher dam may inhibit fish movement or cause severe scour downstream. Figure 11-4 shows the degradation protection for the Oreti River road bridge in New Zealand, the protection consisting of a rock weir downstream of the bridge and rock mattresses installed through the bridge site. Check dams are widely used in New Zealand to maintain bed levels through a bridge site, although increased focus on possible adverse ecosystem impacts is hindering the ready use of this countermeasure at present.

The dam height (relative to the bridge foundations) required to protect a bridge will depend on the identified causes of degradation and the morphology and hydrology of the river at both bridge and catchment scales. To be successful, check dams must not be undermined by piping or seepage around or beneath the dam or erosion upstream or downstream of the structure. In this regard, Parker et al. (1998) observe that the degree of degradation that such structures can withstand before they fail (Fig. 11-5) remains to be determined. Check dams can also initiate erosion of the banks and bed downstream of the structure. Such erosion, which can potentially undermine the dam, or lead to the river outflanking the dam, can be countered by energy dissipation measures for flows over the dam, including stilling basins (Lagasse et al. 2001). Alternatively, several lower weirs, a constructed artificial rapid, or an armored riverbed can be used. Any bank erosion caused by check dams can be controlled by the bank protection measures discussed above. Means of calculating potential scour depths downstream of check dams are discussed in Melville and Coleman (2000) and Lagasse et al. (2001). Breusers and Raudkivi (1991) recommend, however, that predictions of scour downstream of low weirs be derived from specific model tests, owing to possibly significant variations in the predictions of currently available analytical expressions.

11.3.5.2 Channel Lining (Paving) Parker et al. (1998) note that pavements and asphalt paving are limited in application to ephemeral rivers in arid environments. In general, riprap or concrete channel lining (paving) in the vicinity of a bridge has proved unsuccessful at stopping degradation, the lining being subject to undermining. A check dam may be used to protect such a lining, in which case the lining essentially becomes redundant. Channel-lining armoring measures for contraction, thalweg, and sediment-wave effects are discussed further in Sections 11.3.4.1 to 11.3.4.6.

11.3.5.3 Channel Widening, Relief Bridges, and Guide Banks Relief flow paths and widening of the bridge opening and the channel in the vicinity of a bridge can act to alleviate contraction-scour lowering of bed levels at the bridge site (Section 11.3.2). Guide banks (Section 11.5.4) and measures acting to retard flows along upstream floodplains (Section 11.3.4.7) can also be used to reduce flow contraction effects and resulting scour by improving the alignment and efficiency of flows through the bridge opening (Lagasse et al. 2001). Where appropriate, use of streamlined and solid

Fig. 11-4. The rock weir and rock mattresses protecting the Oreti River road bridge (looking downstream).

Fig. 11-5. An undermined and failed concrete check dam designed to control degradation on a Taiwanese river (looking upstream).

foundations can also ease potential debris accumulations and contraction effects.

11.3.6 Aggradation

Potential consequences of aggradation range from bridges being buried by sediment, to a need for bridge relocation, to increased loading on bridge structures (particularly during flooding), to increased likelihood of bridge overtopping and flooding of surrounding areas (with possible associated erosion) owing to a reduced waterway. Associated channel widening and lateral instability can also lead to bridge failure or erosion of bridge approaches, and can result in greater volumes of in-stream floating debris, increasing the potential for blocked waterways and for increased hydraulic loads and scour at bridges downstream.

If aggradation threatening a site is a temporary phenomenon, which in time will dissipate or pass downstream, then the sediment pulse can be dredged or simply allowed to migrate through the site if it is judged that the pulse will not endanger the infrastructure.

For longer-term aggradation, ideally the cause of the aggradation can be remedied, although typically the required measures prove to be complex, extensive, and very costly. Alternatively, and in lieu of diverting the river to accommodate or redesigning the bridge and bridge approaches, the river morphology resulting from the aggradation, active countermeasures are adopted. These countermeasures aim to increase sediment-transport capacity in the vicinity of the bridge site through the use of bridge or channel modifications, or reduce the volume of sediment supplied to the site through the use of structures to trap sediment upstream of the bridge, channel-bed mining or bar removal to control the bed level at the bridge, or general channel maintenance. Any countermeasure method adopted needs to be appropriate to the cause of the associated problem; otherwise it may not be successful.

Ongoing in-channel dredging and removal of channel vegetation can be used to increase flow capacity and consequent sediment-transport capacity. The frequency of such measures will be dictated by comparison of monitored rates of aggradation with tolerable rates determined for the bridge site. Any such measures will require assessments of possible associated pollution and ecosystem impacts.

Alternative control structures reducing the width of the channel can give increased flow velocities and sediment-transport rates.

Construction of a cutoff downstream of a bridge will increase channel slope, inducing higher sediment-transport rates upstream of the cutoff, thereby moderating aggradation at the bridge while the river adjusts to the changed alignment. Cutoffs must be designed with considerable study to correctly assess the magnitudes and locations of potential degradation, aggradation, and lateral erosion. The viability of such a channel realignment solution essentially depends upon the volume of the source aggradation material and the hydraulics and sediment-storage potential of possible alternative channel realignments.

Sediment traps and dams in the catchment upstream of the bridge can be used to reduce the supply of sediment to the bridge site, thereby moderating aggradation at the site. The performance of any in-channel trap must ensure that potential degradation downstream of the trap does not endanger the bridge structure. Sediment traps and dams can be expected to require some degree of ongoing maintenance. Johnson et al. (2001) observe that design of any sediment trap must consider

- appropriate width and depth to enable sediment to settle out of the flow and deposit,
- location facilitating ease of access for sediment-removal equipment,
- location enabling collection of sediments otherwise causing aggradation at the bridge site, and
- environmental impacts, including possible pollution and ecosystem impacts caused by trap maintenance procedures.

Channel-bed mining and bar removal provide very effective means of controlling aggradation or even inducing degradation at a bridge site, although such measures must continue as long as aggradation is occurring. The rate of sediment removal must be monitored to ensure that adverse degradation does not result at the bridge site.

In terms of a general channel-maintenance solution, equipment retained at the bridge site can be used to train the aggrading channel, pushing deposited material across to form terraced riverbanks. This maintenance process must continue as long as aggradation is occurring, and can result in the potentially dangerous situation of large channel levees of ungraded material containing flows over a channel bed elevated above the surrounding countryside. Such a scenario in the South Island of New Zealand has resulted in the Franz Josef community being threatened by the potential of flows being released by flood-induced breaching of the levees elevated above the township. Other measures must then be considered to alleviate the aggradation problem.

In general, any potentially adverse influence of the hydraulic and morphologic impacts of these countermeasure solutions on the bridge site and the general river system must be considered before the countermeasure can be adopted. A useful reference analysis of the potential benefits, disadvantages and costs of aggradation countermeasure options for a bridge in northern Pennsylvania, is presented by Johnson et al. (2001).

11.3.7 Bend and Confluence Scour

Vertical erosion arising from either bend scour or confluence scour can lead to undermining of flow- and grade-control structures and bank protection, a need for bridge relocation, and bridge failure due to undermining of the foundations.

Channel lining (paving) at the outside of the bend using armoring measures discussed above (Sections 11.3.4.1–11.3.4.6) can be used as a countermeasure for vertical erosion arising from bend scour. Flow training measures discussed above (Sections 11.3.4.8–11.3.4.10) can alternatively be used to redirect flow through the bend, although it must be ensured that such measures do not simply relocate any adverse scour to a foundation away from the outside of the bend.

In the absence of foundation designs allowing for confluence scour magnitudes (Melville and Coleman 2000), foundations can be protected by maintenance of bed levels using check dams running through the bridge site and encompassing the foundations. Use of check dams extending across the entire width of the channel reflects the variable nature of confluence locations for braided rivers. Where constructing a check dam would prove prohibitively expensive for a wide braided river, monitoring of river planform development could alternatively be adopted as a form of countermeasure (Coleman et al. 2000). Protection measures at an individual foundation would then only be instigated when observed patterns of channel development indicate the possibility of a confluence scour hole impacting the foundation. Protection measures adopted for individual foundations could include armoring or flow training measures discussed above (Section 11.3.4).

11.3.8 Debris and Ice Jams

The accumulation of debris at bridge foundations typically increases the scour. Streamlined pier shapes are less likely to cause debris to accumulate, whereas pile bents are particularly prone to debris accumulation. Similarly, ice jams can exacerbate scour. At locations where ice jams and debris accumulations are expected, relief structures and roadway overtopping can be used effectively to reduce flooding as well as scour.

Piers and low-elevation superstructure may disrupt the flow of debris and ice through bridge openings. Velocities near the bridge may be significantly higher than those estimated when ice or debris accumulates on piers and superstructures blocking large areas of flow. Consequently, estimates of the potential impact of debris and ice blockage and their effect on flow direction and velocity should be considered when determining the type and size of countermeasures necessary to protect a bridge. Diehl (1997) provides a method for assessing the potential for debris accumulation on bridge elements that may be adapted for design of countermeasures at locations where debris is likely to be a problem. Bridges located on actively incising or widening streams are highly susceptible to blockage by debris because of the high input and transport of trees delivered through bank erosion (Diehl 1997; Parola et al. 1998).

11.4 COUNTERMEASURES FOR LOCAL SCOUR AT BRIDGE PIERS

11.4.1 Introduction

There are two categories of methods of protection of bridge piers against scour: armoring devices, such as riprap and alternatives to riprap, and flow-altering devices, such as sacrificial piles, horizontal collars, and deflector vanes. Alternatives to riprap include artificial riprap, such as toskanes and dolos, cable-tied blocks, grout-filled bags, and foundation extensions such as extended footings. The last can be effective in reducing local scour if the top level of the footing is at or below the undisturbed bed level, but can increase scour if the footing is at a higher level.

11.4.2 Pier Shape Design

The local scour at circular bridge piers is unaffected by changes in flow direction, rendering circular piers preferable to all other shapes where changes in flow alignment are likely. For piled foundations, the scour increases with the number and closeness of the piles. Therefore, it is better to develop bearing capacity using fewer and deeper piles. The local scour at piers with slab footings, pile bents, and piers founded on caissons may be exacerbated if the footing, pile cap, or caisson is at or above the undisturbed bed level. For piled foundations, it is preferable to construct pile caps above normal water level to minimize their influence on scouring.

11.4.3 Riprap Protection at Piers

The most commonly employed method of protecting bridge piers against scour is the use of a layer of riprap around the piers. Figure 11-6 shows riprap protection at a model-scale bridge pier (diameter 200 mm) prior to testing. The model riprap is crushed rock with median diameter 50 mm. The principle behind this technique is that large stones that are heavier than the bed sediment are able to withstand the higher shear stresses that occur around a bridge pier. A number of studies and reports dealing with riprap protection at bridge piers have been published, including Engels (1929); Gales (1938); Sousa Pinto (1959); Maza Alvarez (1968); Bonasoundas (1973); Neill (1973); Quazi and Peterson (1973); Posey (1974); Hjorth (1975); Breusers et al. (1977); Dargahi (1982); Farraday and Charlton (1983); Worman (1987); CBIP (1989); Parola and Jones (1989); Worman (1989); Breusers and Raudkivi (1991); Parola (1991, 1993); Austroads (1994); Chiew (1995); Parola (1995); Richardson and Davis (1995); Croad (1997); Lim and Chiew (1997); Parker et al. (1998); Lauchlan (1999); and Melville and Coleman (2000).

11.4.3.1 Failure Mechanisms for Riprap Placed at Bridge Piers The following four failure mechanisms of riprap layers at bridge piers were observed during laboratory

Fig. 11-6. Riprap protection at a model-scale bridge pier prior to testing (after Lauchlan 1999).

studies, including those of Parola (1993), Chiew (1995), and Lauchlan (1999):

- *Shear failure*. Shear failure occurs where the riprap stones are entrained by the flow, because they are unable to resist the hydrodynamic forces induced by the flow.
- *Winnowing failure*. The action of turbulence and seepage flows erodes the underlying bed material through voids between the riprap stones, a process that is more likely to occur in sand-bed rivers than in coarser bed materials. A filter is often recommended to resist winnowing failure.
- *Edge failure*. Scouring at the periphery of the riprap layer undermines the riprap stones. Riprap is vulnerable to edge failure in conditions where there is insufficient lateral extent of the protective layer.
- *Bed-form undermining*. The migration past the pier of the troughs of large dunes undermines the riprap layer, which settles as a consequence. Bed-form undermining is the controlling failure mechanism at bridge piers founded in riverbeds subject to migration of dunes, especially sand-bed rivers, according to Lim and Chiew (1997), Parker et al. (1998), and Lauchlan (1999). Figure 11-7 is a schematic diagram showing the failure mechanisms for a dune bed for riprap placed at the bed surface and riprap placed below the bed surface, respectively.

11.4.3.2 Riprap Design for Pier Protection Riprap design for pier protection against scour involves consideration of the following characteristics of the riprap layer, as illustrated in Fig. 11-8:

- Median size (D_{r50}) and gradation of the riprap material;
- Vertical thickness (t_r) of the riprap layer;
- Plan layout and horizontal coverage of the riprap layer, B_r and L_r;

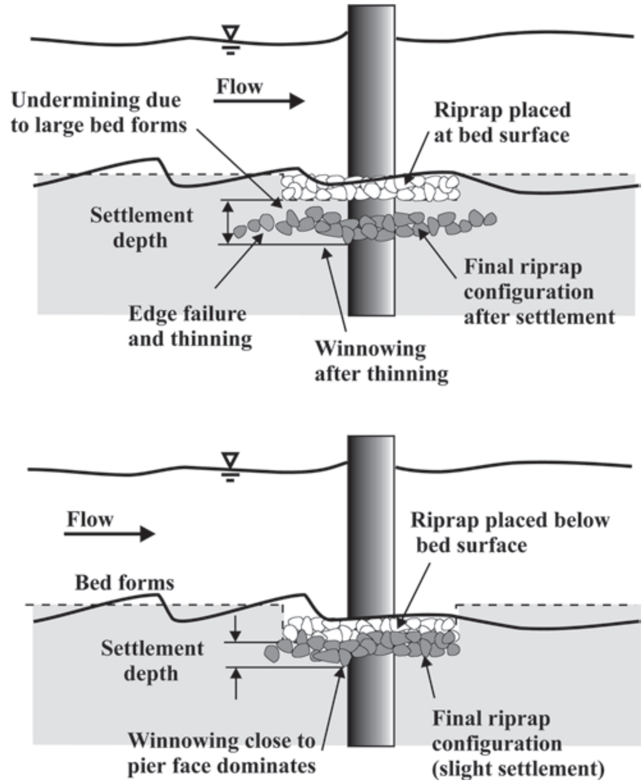

Fig. 11-7. Failure mechanisms for riprap protection at bridge piers (after Melville and Coleman 2000).

- Placement depth (Y_r) of the surface of the riprap layer below the sediment bed level; and
- Need for, and design of, a filter layer beneath the riprap.

11.4.3.3 Riprap Size Some of the equations that have been suggested for sizing riprap at bridge piers are given in Table 11-5. Most of these equations can be expressed in the form

$$\frac{D_{r50}}{H} = \frac{C}{(S_s - 1)^x} \mathsf{F}^y \tag{11-1}$$

where

F = $U/(gH)^{0.5}$;
H = flow depth;
U = mean flow velocity;

and C, x, and y are coefficients, y typically varying between 2 and 3. It is apparent that riprap stone size depends strongly on flow velocity, but is less dependent on flow depth. In using these equations, U and H can be taken to be the depth-averaged velocity and the depth of the flow approaching the pier under consideration.

Melville and Coleman (2000) show that the riprap size equations predict widely varying stone sizes. Several of the

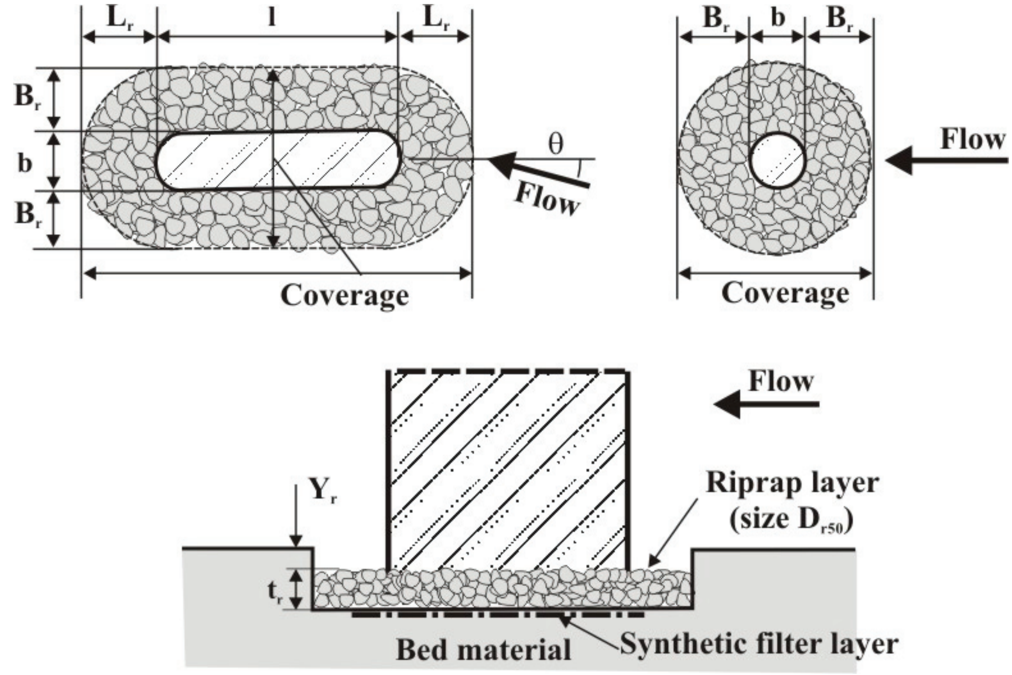

Fig. 11-8. Definition diagram for placement of riprap protection at bridge piers.

equations are based on laboratory data, whereas others have not been validated with data. Significant among the former group are the equations derived from the laboratory studies of Parola (1990)—see Parola and Jones (1989); Parola (1993; 1995); and Richardson and Davis (1995)—and those of Quazi and Peterson (1973) and Lauchlan (1999). Parola (1990) investigated the stability of riprap layers around circular and rectangular piers under clear-water conditions. The riprap was either placed flush with the bed or mounded above the bed. The failure criterion was related to the exposure of any part of the second layer of a three-stone thick layer. Parola (1993) presented an equation for cylindrical piers (N_{sc} = 1.4) and three equations for rectangular piers, depending on the riprap size relative to the pier width. Comparison of the former equation with standard riprap size relations (see Appendix B) indicates that riprap placed at a bridge pier needs to be about 2 to 3.6 times larger than the size required for stability in uniform undisturbed flow for the same flow conditions (Parola et al. 1995). Lauchlan (1999) examined the stability of riprap layers at circular and rectangular piers under live-bed conditions, finding no significant difference in riprap stability for the two pier shapes. The riprap was adjudged to have failed if the scour depth exceeded 20% of that at an unprotected pier under the same conditions. Lauchlan (1999) also investigated the effect of placing riprap below the sediment bed surface as a means of counteracting the influence of bed-form undermining. Quazi and Peterson (1973) formed a sediment bed of riprap stones and determined the flow velocity at which the stones were just stable for a round-nosed pier. Richardson and Davis (1995) recommend the Parola and Jones (1989) equation with an additional factor (f_2) for pier location in the channel. This equation is also suggested for use by Parker et al. (1998). The equations based on laboratory data, discussed above, are compared in Fig. 11-9 over the range F = 0 to 0.6 and for specific gravity of riprap S_s = 2.65.

Given the different experimental methods and, in particular, diverse failure criteria among these methods, the riprap size predictions of these equations are acceptably consistent and give reasonable estimates of stone size for design. A conservative combination of the rock size estimates given by the plotted equations is obtained by using the upper envelope generated by the Lauchlan (1999) relation together with the appropriate Parola relation.

11.4.3.4 Riprap Gradation Although the exact size distribution of riprap is not critical, it is important that the riprap should be well graded. Richardson and Davis (1995) state that the maximum rock size should not exceed twice the median size of the riprap; that is, $D_{r\max} \leq 2D_{r50}$. Croad (1997) gives an additional criterion, $D_{r50} \leq 2D_{r15}$. The grading curve envelope (upper and lower limits) shown in Fig. 11-10 encompasses most of the recommended gradings (Gregorius 1985).

11.4.3.5 Lateral Extent Recommendations for the areal extent of riprap protection at bridge piers, based on

Table 11-5 Equations for Sizing Riprap at Bridge Piers

Reference	Equation		Symbols
Bonasoundas (1973)	$D_{r50}\,(\mathrm{cm}) = 6 - 3.3U + 4U^2$	(11-7)	D_{r50} = riprap stone size for which 50% are finer by weight The equation applies to stones with $S_s = 2.65$ U = mean approach flow velocity (m/s)
Quazi and Peterson (1973)	$N_{sc} = 1.14\left(\dfrac{D_{r50}}{H}\right)^{-0.2}$	(11-8)	N_{sc} = Critical Stability Number = $U^2/[g(S_s\text{-}1)D_{r50}]$ F = Froude number of the approach flow = $U/(gH)^{0.5}$ H = mean approach flow depth
Breusers et al. (1977)	$U5\ 0.42[2g(S_s 2\ 1)D_{r50}]^{0.5}$	(11-9)	S_s = specific gravity of riprap stones
Farraday and Charlton (1983)	$\dfrac{D_{r50}}{H} = 0.547\,\mathsf{F}^3$	(11-10)	
Parola and Jones (1989)	$\dfrac{D_{r50}}{H} = \dfrac{0.346\,f_1^2}{(S_s - 1)}\mathsf{F}^2$	(11-11)	f_1 = factor for pier shape: f_1 = 1.5 (round-nose), 1.7 (rectangular)
Breusers and Raudkivi (1991)	$U = 4.8(S_s - 1)^{0.5}\, D_{r50}^{1/3} H^{1/6}$	(11-12)	
Austroads (1994)	$\dfrac{D_{r50}}{H} = \dfrac{0.58\,K_p\,K_v}{(S_s - 1)}\mathsf{F}^2$	(11-13)	K_p = factor for pier shape: K_p = 2.25 (round-nose), 2.89 (rectangular) K_v = velocity factor, varying from 0.81 for a pier near the bank of a straight channel to 2.89 for a pier at the outside of a bend in the main channel
Richardson and Davis (1995)	$D_{r50} = \dfrac{0.692\,(f_1 f_2 V)^2}{(S_s - 1)\,2g}$	(11-14)	f_2 = factor ranging from 0.9 for a pier near the bank in a straight reach to 1.7 for a pier in the main current at a bend
Chiew (1995)	$D_{r50} = \dfrac{0.168}{\sqrt{H}}\left(\dfrac{U}{U_*\sqrt{(S_s - 1)g}}\right)^3$ $U_* = \dfrac{0.3}{K_D\ K_H}$	(11-15)	K_H = flow depth factor $K_H = 0.783\left(\dfrac{H}{b}\right)^{0.322} - 0.106 \quad 0 \le \left(\dfrac{H}{b}\right) < 3$ $K_H = 1 \quad \left(\dfrac{H}{b}\right) \ge 3$ K_D = sediment size factor $K_D = 0.398\ln\left(\dfrac{b}{D_{r50}}\right) - 0.034\left[\ln\left(\dfrac{b}{D_{r50}}\right)\right]^2 \quad 1 \le \left(\dfrac{b}{D_{r50}}\right) < 50$ $K_D = 1 \quad \left(\dfrac{b}{D_{r50}}\right) \ge 50$
Parola (1993; 1995)	Rectangular: $N_{sc} = 0.8 \quad 20 < (b_p / D_{r50}) < 33$ $N_{sc} = 1.0 \quad 7 < (b_p / D_{r50}) < 14$ $N_{sc} = 1.2 \quad 4 < (b_p / D_{r50}) < 7$ Aligned Round-nose: $N_{sc} = 1.4$	(11-16)	b_p = projected width of pier
Croad (1997)	$\dfrac{U}{A\sqrt{(S_s - 1)gD_{r50}}} = 1.16\left(\dfrac{H}{D_{r50}} - 2\right)^{1/6}$ $D_{r50} = 17\,D_{b50}$	(11-17)	A = acceleration factor: A = 0.45 (circular and slab piers), A = 0.35 (square and sharp-edged piers) D_{b50} = median size of bed material Equation given for factor of safety = 1.25, as recommended by Croad (1997)
Lauchlan (1999)	$\dfrac{D_{r50}}{H} = 0.3S_f\left(1 - \dfrac{Y_r}{H}\right)^{2.75}\mathsf{F}^{1.2}$	(11-18)	S_f = safety factor, with a minimum recommended value = 1.1 Y_r = placement depth below bed level

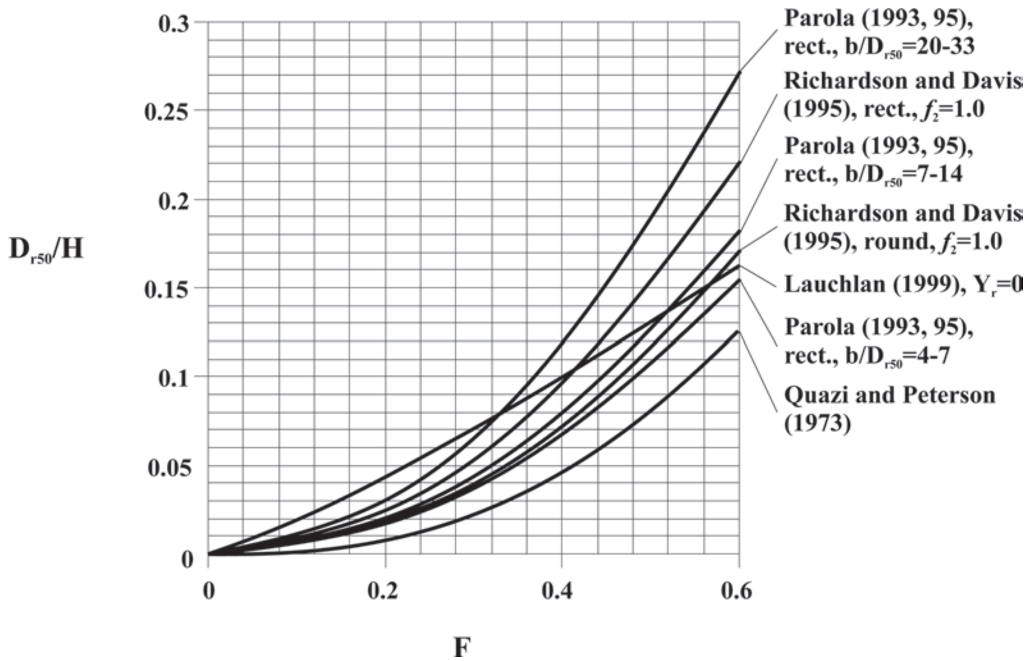

Fig. 11-9. Comparison of equations for sizing riprap at bridge piers.

laboratory testing, have been made by Sousa Pinto (1959); Maza Alvarez (1968); Bonasoundas (1973); Ruff and Nickelson (1993); Chiew (1995); Parola (1995); Croad (1997); Parker et al. (1998); and Lauchlan (1999), among others. These recommendations range from placing riprap only at the nose of the pier to completely surrounding the pier with a riprap layer extending up to 3b (where b is pier width) from the pier face in all directions. For rectangular piers, Parker et al. (1998) suggest the equation

$$B_r = L_r = \frac{1.5b}{\cos\theta} \qquad (11\text{-}2)$$

where:

θ = angle of attack of the flow (see Fig. 11-8).

Oblong-shaped piers can be treated similarly. An equivalent coverage for a circular pier is to use a circular stone mat of diameter 4b, where b is the pier diameter.

11.4.3.6 Layer Thickness A range of recommendations for riprap layer thickness (t_r), typically from $t_r = 2D_{r50}$ to $3D_{r50}$, have been made. Thicker riprap layers impede the winnowing process and are able to resist disintegration through rearmoring. Chiew (1995) showed that thicker layers resist higher flow velocities, whereas laboratory testing by Lauchlan (1999) indicated an approximate 70% reduction in local scour pertaining to an increase in thickness from $1D_{r50}$ to $3D_{r50}$.

11.4.3.7 Placement Level Richardson and Davis (1995) and others propose that the surface of the riprap layer be placed at the streambed level. Neill (1973) and Breusers et al. (1977) recommend placing the riprap below the expected general scour level. Lauchlan (1999) found that placing riprap at some depth within the bed significantly improved the performance of the layer under live-bed conditions in sand-bed streams. The term $(1\text{-}Y_r/H)^{2.75}$ in her equation (Table 11-5) reflects this advantage, as shown in Fig. 11-11. Riprap placed deeper is inherently more stable, especially in sand-bed rivers, because the stones

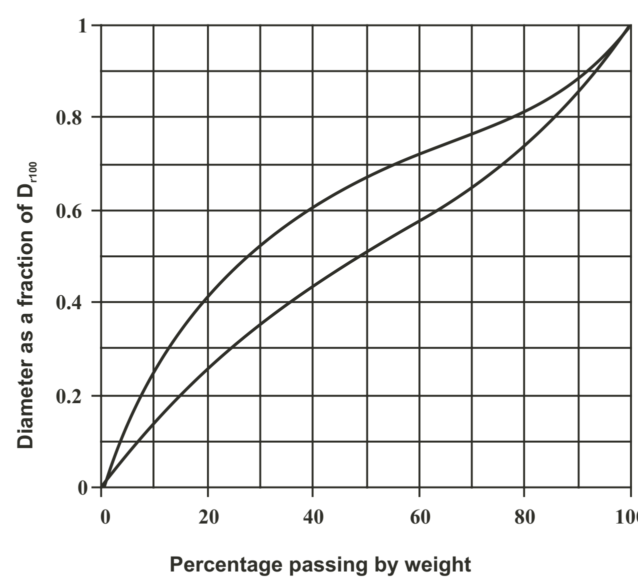

Fig. 11-10. Riprap grading curve envelope (after Gregorius 1985).

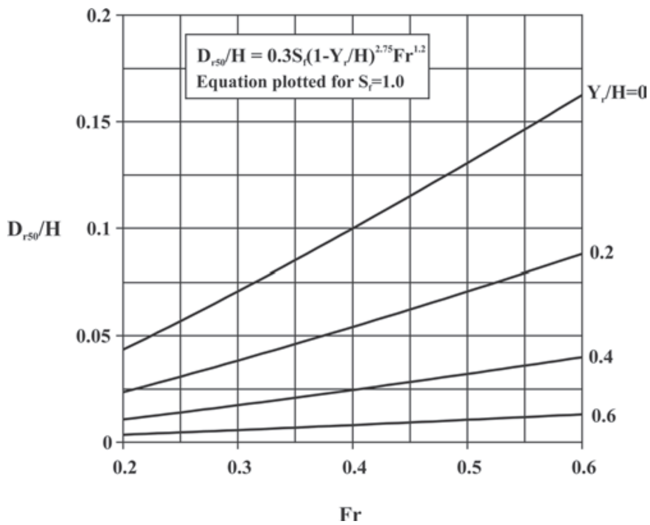

Fig. 11-11. Pier riprap size according to Lauchlan (1999) (modified from Melville and Coleman 2000).

are more resistant to bed-form undermining; see Fig. 11-8. Conversely, Parola (1991; 1993) observed enhanced stability for riprap mounded around the pier under clear-water conditions compared to that for riprap placed in preformed scour holes. Mounded riprap has construction advantages, and the mound may provide a source for replenishment of riprap material in the event of loss of riprap stones during a flood. Richardson and Davis (1995) warn that "it is a disadvantage to bury riprap so that the top of the mat is below the streambed because inspectors have difficulty determining if some or all of the riprap has been removed." Also, Parker et al. (1998) found that riprap dumped over a geotextile placed on an unexcavated bed performed almost as well as riprap placed with prior excavation.

11.4.3.8 Filters To combat winnowing effects at the pier face and to improve general stability of riprap layers, the use of a filter layer beneath the riprap stones has been proposed. Filters can be either granular filters, which make use of the filtering effect of graded sediments, or synthetic filters, commonly known as geotextiles. Filters must prevent the passage of the finer bed sediment, but also have adequate permeability to prevent build-up of water pressure in the underlying sediment. The well-known Terzaghi and Peck filter criteria (see Appendix B) have been proposed to select suitable granular filter media, although Posey (1974) found that a single filter layer was sufficient and Worman (1989) found that a thick single layer of riprap was an adequate alternative to the conventional Swedish practice of using multilayered riprap (incorporating a granular filter layer). The use of granular filters in the highly turbulent flow region at the base of a bridge pier is questionable. For example, Escarameia and May (1992) concluded from an experimental study that sand complying with the Terzaghi-based requirements performs poorly in highly turbulent environments.

An advantage of using geotextiles is fabric flexibility, which allows the geotextile to deform and remain intact, as well as to be reasonably resistant to tension and tearing. Important parameters in geotextile selection are appropriate pore size to retain finer sediments without clogging, adequate permeability to release pore pressures without causing uplift of the fabric under flood conditions, ultraviolet light resistance, puncture resistance, and shear strength. The lateral extent of the synthetic filter should be limited to about 75% of the lateral extent of the riprap. The reduced coverage of the synthetic filter ensures that edge stones in the riprap layer are able to protect the synthetic filter from being rolled up by the flow (Parker et al. 1998). It is important that the geotextile be adequately sealed to the pier face to prevent sediment from leaching at the pier/geotextile interface. Parker et al. (1998) offer suggestions for underwater installation of geotextiles at bridge piers.

On the basis of a detailed, large-scale laboratory study of riprap protection at bridge piers, with and without geotextiles, Parker et al. (1998) found "that under flood conditions in sand-bed streams with developed bed forms, the leaching of sand from the interstices of any armoring countermeasure may ultimately result in failure of the countermeasure. With this in mind, and in light of the positive results of experimental testing, it is suggested that such an armoring countermeasure be underlain by an appropriately selected geotextile." They also suggest that geotextiles not be used for gravel bed streams, due to the abrasive nature of gravel and its low potential for leaching. In addition, geotextiles should not be used at sites where significant degradation is likely, because the scour may leave the geotextile and riprap perched during floods, possibly leading to the loss of both (see below). At bridge piers on the floodplain where clear-water scour conditions typically pertain, this potential disadvantage of the use of geotextiles is less likely to exist.

11.4.3.9 Riprap Tolerance to Degradation Degradation occurs in rivers when the outflow of sediment exceeds the inflow, leading to a net loss of sediment in the reach. Lauchlan (1999) investigated the effect of a degrading bed on riprap protective layers at bridge piers. Laboratory experiments indicated that riprap layers are capable of providing a reasonably high degree of protection for bridge piers for high rates of degradation and high flow rates. However, the results also imply that in a degrading bed situation, riprap protective layers would eventually fail. As the bed surrounding the riprap degrades, the stones subside and can move outward. Long-term degradation causes the majority of the stones to move outward from the pier, which reduces the thickness of the riprap layer and its ability to protect the pier. However, if lack of sediment input is merely a short-term problem, the layer is likely to be able to withstand the attack.

Because subsidence of the riprap layer with the degrading bed is important in maintaining stability of the layer, filters should not be employed where significant degradation is anticipated. A geotextile or granular filter would prevent winnowing from occurring at the pier face, and winnowing is essential if the riprap is to subside. The implication is that the riprap would not subside if coupled with a filter, leading to increased exposure of the stones, disintegration of the riprap, and loss of protection against scour. At sites where degradation is anticipated, it is preferable to increase riprap layer thickness rather than use a filter.

11.4.4 Alternatives to Riprap

Alternatives to rock riprap include other armoring measures, overviewed in Section 11.3.4, and flow-altering measures. Aside from these active countermeasures to prevent scour, bridges can also be structurally modified through underpinning or foundation extension. Examples of flow-altering measures that have been used or suggested to protect piers against local scour include sacrificial piles placed upstream of the pier, Iowa vanes, and flow deflectors attached to the pier such as collars. Field experience of flow-altering devices is limited.

11.4.4.1 Artificial Riprap Each form of artificial armor unit (Section 11.3.4.3) will have unique design criteria that should be available from the unit manufacturer. In terms of common forms of artificial riprap, Parker et al. (1998) consider guidelines for implementation at bridge piers to be complete, the work of Ruff and Fotherby (1995) being noteworthy.

Despite the detailed design criteria available, there are few examples of the use of artificial riprap as a scour countermeasure at bridge piers. Studies to date indicate that artificial riprap does not offer significant advantages over rock riprap for scour protection at piers. Ruff and Fotherby (1995) conclude this in terms of toskanes. Fotherby (1992) and Bertoldi et al. (1996) both suggest that the use of tetrapods at bridge piers offers little advantage over riprap in terms of stability of the armoring units. An additional disadvantage of artificial riprap in comparison to rock riprap is a possible lengthy installation time to achieve the required interlocking nature of the units.

If artificial riprap is to be utilized at a bridge pier, the reader is referred to Fotherby and Ruff (1996) and Parker et al. (1998) for design guidelines and principles, along with comments on construction and maintenance for this scour countermeasure. In addition, Lagasse et al. (2001) provide summaries of design procedures and present design examples for bridge-pier protection using toskanes (Fotherby and Ruff, 1996) and modules of A-Jacks (Armortec Inc., Bowling Green, Kentucky).

11.4.4.2 Cable-Tied Blocks A few examples of the use of cable-tied blocks (Section 11.3.4.3) currently exist at piers in the United States. More such installations may follow; cable-tied blocks have recently been shown (Jones et al. 1995; Bertoldi et al. 1996; University of Minnesota 1996; 1997; Parker et al. 1998) to provide a useful alternative to riprap at bridge piers over a wide range of conditions and over successive flow events. The performance of this countermeasure is aided by the blocks being underlain by an appropriately sized geotextile filter, and also by the geotextile being sealed to the pier. Design guidelines given by Parker et al. (1998) are summarized in Table 11-6.

11.4.4.3 Gabions and Reno Mattresses Gabions (Section 11.3.4.5) have experienced significant use in the field as a countermeasure for bridge scour, although a recent evaluation of their field use in New York State is rather pessimistic, following the failure of many installations. Design guidelines given by Parker et al. (1998) for the use of gabions and Reno mattresses (Section 11.3.4.5) at bridge piers are summarized in Table 11-7. Additional detailed guidelines for the materials and construction of gabions and Reno mattresses are also given in Parker et al. (1998).

11.4.4.4 Grout-Filled Bags or Mats Grout-filled bags (sacks) or mats constitute fabric shells filled with concrete. These measures can be deployed rapidly and provide an economical alternative to rock riprap where this is not readily available. A particular advantage is that shells filled with dry concrete can be placed directly at bridge foundations, with hydration occurring naturally.

With regard to their potential to slide and disperse (Section 11.3.4.3), Parker et al. (1998) recommend avoiding grout-filled bags, concluding that riprap and cable-tied blocks are generally more effective as countermeasures for pier scour. In the event that grout-filled bags are nevertheless to be utilized, design guidelines given by Parker et al. (1998) are summarized in Table 11-8. Installation practices at bridge foundations, critical to the success of grout-filled bag systems, are also discussed in Lagasse et al. (2001).

Fotherby (1992), Jones et al. (1995), and Bertoldi et al. (1996) report studies of the use of concrete-filled mats (Section 11.3.4.4) for pier protection. These studies show that mats need to be bound to and sealed with the pier (although they recognize potential increased pier loadings) and recommend that mats be installed with their top surfaces flush with the bed, this reducing or eliminating the need for any anchoring to prevent uplift failure. Failure likely involves replacement of the entire unit; failure modes include undermining, overturning and rolling up of an unsecured leading edge, and uplift of the inner mat at higher velocities. Guidelines for mattress areal extent, thickness, and anchoring remain to be determined, including a lift criterion to size grout mattresses to prevent failure by rollup.

11.4.4.5 Concrete Apron and Grouted Riprap Concrete pavements and asphalt paving are best suited to applications in ephemeral rivers in arid environments (Parker et al. 1998). In general, bridge designers doubt the durability of in-stream pavements and anticipate turbulence-induced and confined-groundwater uplift stresses generated during flood events to cause failure of the impermeable pavement. Pavement edges are also prone to undermining, possibly leading to destabilization of the pavement.

Table 11-6 Principles and Guidelines for the Design of Cable-Tied Blocks (Fig. 11-2) for Pier-scour Protection

Factor	Design criteria
Feasibility	Suitable for sand-bed and gravel-bed rivers Not suitable for pile bents or complex pier shapes Not suitable for rivers with large cobbles or rocks Not suitable for corrosive water quality, such as saline (including estuarine) or acidic environments Favorable characteristics for ephemeral, flashy, and moderate hydrograph streams, as well as floodplain installations May become cost-effective where rock riprap of a required size and quality is not readily available
Block shape, spacing, and size	Block shape to facilitate mat flexibility Spacing between blocks to facilitate mat flexibility $\varsigma = a_{cb}\left(\frac{\rho_{cb}}{\rho_{cb}-\rho}\right)\rho U^2$ and $\varsigma = \rho_{cb}\, gH_{cb}\,(1-p)$ where: ζ = weight per unit mat area (N/m^2) (required for mat stability) H_{cb} = block height (m) p = volume fraction pore space of the mat ρ = water density = 1000 kg/m^3 ρ_{cb} = density of the block material for the mat (kg/m^3) a_{cb} = 0.20 U = depth-averaged flow velocity (m/s)
Mat installation	Preexcavation of the upstream edge is required, and for gravel-bed streams, all edges must be anchored (requiring preexcavation) General prior excavation is not required unless $4H_{cb}$ > design approach flow depth Mat (centered on the pier) is to be of width ($4D/\cos\beta$) and length in the direction of flow [L+ ($3D/\cos\beta$)] where: β = angle of flow attack ($\beta = 0^{\circ}$ giving the flow aligned with the pier) D = pier diameter for cylindrical pier, and pier width for rectangular pier L = pier length (= D for cylindrical pier)
Cable location and quality	Cables to be located near the center of each block to allow maximum mat flexibility Cables to be sufficiently flexible to allow mat deformation, but sufficiently durable to survive at least 20 years in situ Stainless steel to be used for harsh environments
Geotextile filter	Resists leaching of bed material from between blocks To be fastened firmly to the base of a mat for a sand-bed river Not to be used for a gravel-bed river Not to extend to the mat edges, but (approximately extending 2/3 of the distance from each pier face to the mat edge) to be of width ($3D/\cos\beta$) and length in the direction of flow [$L + (2D/\cos\beta)$] Not to be replaced with a granular filter layer In some cases, local grouting is recommended wherever there is danger of abrasion of the geotextile
Pier seal	Mat (and geotextile filter) to be fastened and sealed to the pier (recognizing potential increased pier loadings), aided by a granular filter zone if required

Table 11-7 Principles and Guidelines for the Design of Gabions and Reno Mattresses for Pier-scour Protection

Factor	Design criteria
Feasibility	Potential abrasion of casing materials by passing sediments of sand-size and larger needs to be recognized and addressed where possible by material selection Gabions are not recommended for gravel-bed streams owing to bed-load abrasion wearing out the casing causing gabion rupture Well suited to ephemeral streams, but potentially difficult to place in deeper channels Not suitable for corrosive water quality, such as saline or acidic environments Potentially difficult to implement for nonuniform riverbed or pier geometries Useful where rock riprap of a large required size is not readily available
Basket size and shape	$V_{\min} = 0.069\left[\frac{U^6 K^6}{(S_{sr}-1)^3 g^3}\right]$, where: $V_{\min}$ = minimum basket volume for individual unconnected baskets (m^3) U = depth-averaged flow velocity (m/s) K = pier-shape factor (round-nosed piers, $K=1.5$; square-nosed piers, $K=1.7$) $S_{sr} = \rho_r/\rho$ = rock specific gravity ρ_r = rock density (kg/m^3) ρ = water density = 1000 kg/m^3 g = gravitational acceleration, g = 9.81 m/s^2 Basket volumes larger than $V_{\min}$ may be appropriate Baskets to be kept relatively low in height to reduce cross-sectional blockage and resist uplift, with basket heights to exceed a minimum of 0.15 m Standard gabions are of nominal heights of 0.3, 0.45, or 0.9 m; nominal lengths of 1.8, 2.7, or 3.6 m; and a nominal width of 0.9 m Standard Reno mattresses are of nominal heights of 0.15 or 0.225 m; nominal lengths of 2.7 or 3.6 m; and a nominal width of 1.8 m
Gabion-field installation	Riverbed to be smoothed and existing scour holes filled with stones before gabions are installed, preexcavation to give the top of the gabion installation flush with the bed being advantageous The gabion-field coverage (centered on the pier) is to be of width ($5D/\cos\beta$) and length in the direction of flow [$L+(4D/\cos\beta)$], where: β = angle of flow attack ($\beta = 0^\circ$ giving the flow aligned with the pier) D = pier diameter for cylindrical pier, and pier width for rectangular pier L = pier length (= D for cylindrical pier) Gabions are readily stacked in stable configurations and mould themselves in response to instabilities Adjacent baskets to be joined using the same wire used to lace the baskets Completed gabions lifted into place, or empty baskets joined to gabions already in position, then stretched and correctly aligned before being filled Hand work helps to minimize the percentage of voids in baskets
Basket materials	Minimum rock size to be at least 25% larger than the minimum basket opening Maximum rock size not to exceed 2/3 of the minimum basket dimension Casing materials must be durable and also facilitate basket flexibility, ideally single-strand galvanized or PVC-coated wiring that resists corrosion (and with the wire recommended to be like a chain-link fence, i.e., formed with a double twist to prevent unraveling) Basket sidewalls to be reinforced with wires of diameter larger than that used for the basket mesh in order to provide sidewall stiffness
Geotextile filter	Resists leaching of bed material from beneath the gabions To be used underneath the gabion field for a sand-bed river Not to extend to the edges of the basket field Can be replaced with a granular filter layer if the geotextile is not available
Pier seal	Geotextile filter to be fastened and sealed to the pier (in recognition of potential increased pier loadings), aided by a granular filter zone if required

Table 11-8 Principles and Guidelines for the Design of Grout-Filled Bags for Pier-scour Protection

Factor	Design criteria
Feasibility	Potentially applicable only to small streams, or where bag width (~1 m) > pier width (bags then being large relative to any local scour hole) Not suitable for gravel-bed streams, or sand-bed streams with developed dunes Can be aesthetically unacceptable Useful where rock riprap of a required size and quality is not readily available
Bag size and shape	Design as for riprap, with D_{r50} = bag height, and with the following amendments: In calculations, use the material density ρ_r pertaining to the grout Increasing the bag size D_{r50} by a factor of 1.2 is recommended to aid bag stability Bags are not to be of sizes or shapes that hinder flexibility of the installed countermeasure Shorter bags heights are desirable An example bag size is $3 \times 0.9 \times 0.3$m
Bag-field installation	The bag-field coverage (centered on the pier) is to be of width ($5D/\cos\beta$) and length in the direction of flow [$L + (4D/\cos\beta)$], where β = angle of flow attack ($\beta = 0^{\circ}$ giving the flow aligned with the pier) D = pier diameter for cylindrical pier, and pier width for rectangular pier L = pier length (= D for cylindrical pier) Upstream bags are to overlap downstream bags to aid stability Fotherby (1992) indicates that properly sized bags are more effective if used to extend a single layer of protection laterally, rather than if they are stacked Imbricated (shingled) stacking can potentially enhance interlocking and aid stability
Bag materials	If possible, the surface of the bag should be rendered angular and rough Grout quality should ensure that the grout does not degrade and break or crumble
Geotextile filter	Resists leaching of bed material from between bags To be used underneath the bag field for a sand-bed river Not to be used for a gravel-bed river Not to extend to the edges of the bag field, but to be of width ($3D/\cos\beta$) and length in the direction of flow [$L + (2D/\cos\beta)$] Can be replaced with a granular filter layer if the geotextile is not available
Pier seal	Geotextile filter to be fastened and sealed to the pier (recognizing potential increased pier loadings), aided by a granular filter zone if required

Parker et al. (1998) indicate concrete-grouted riprap to be relatively cost-effective, although it scores poorly in terms of aesthetics and environmental acceptability, as well as on feasibility for use with finer sediments such as silt. The reduced permeability arising from the grouting is disadvantageous because uplift can cause failure of the riprap layer in entirety. The decreased countermeasure flexibility also negates the natural benefit of riprap being able to deform and armor a developing scour hole.

11.4.4.6 Sacrificial Piles Sacrificial piles are piles placed upstream of a pier to deflect high-velocity flow from impacting the pier, with the pier located in the wake region behind the piles (Fig. 11-12). The piles can be arranged in a variety of plan configurations, with varying pile sizes and numbers, and with the piles submerged or extending over the full depth of flow. A triangular pile configuration, with the apex pointing upstream, has been shown to be one of the better configurations in terms of protecting the pier (Fig. 11-12). Sacrificial piles have the benefit of being relatively quick to implement. They are, however, not suited to riverbeds of bedrock or boulders, and they can be unacceptable on aesthetic grounds. Chang and Karim (1972) and Paice and Hey (1993) report laboratory studies and field experience of the use of sacrificial piles for pier-scour protection. Further laboratory studies are reported by Chabert and Engeldinger (1956); Levi and Luna (1961); Shen et al. (1966); Wang (1994); Singh et al. (1995); and Melville and Hadfield (1999).

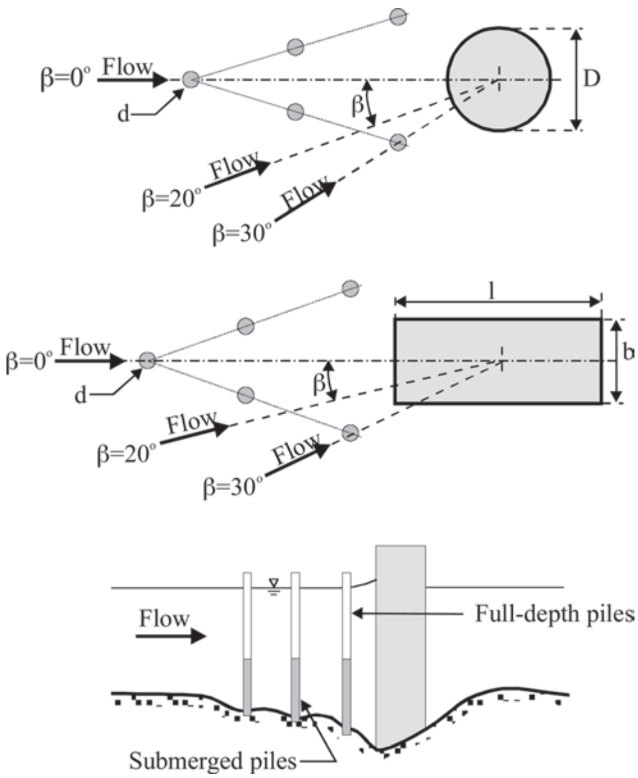

Fig. 11-12. Example configurations of five sacrificial piles.

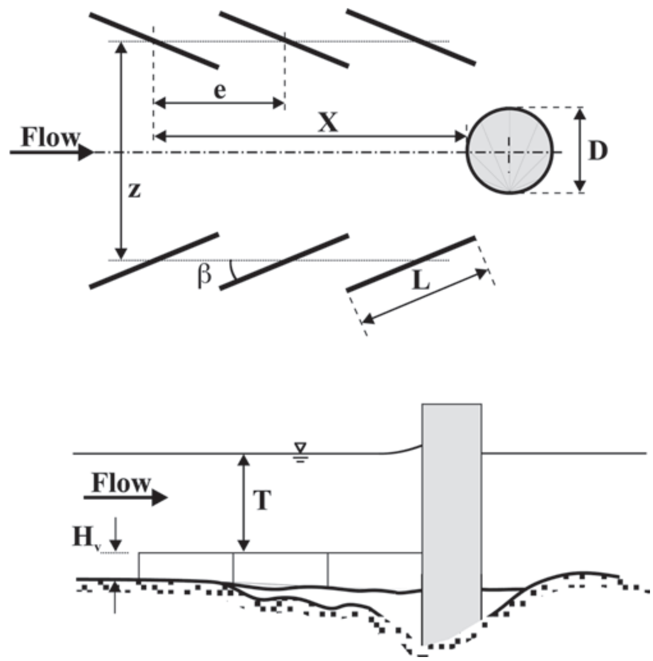

Fig. 11-13. The use of Iowa vanes as a pier-scour countermeasure.

The effectiveness of sacrificial piles for pier-scour protection is found to be particularly dependent on the approach flow angle β (Fig. 11-12) and flow intensity V/V_c, where V is depth-averaged flow velocity, and V_c is this velocity at the threshold condition for sediment movement. For aligned ($\beta = 0^o$) clear-water ($V/V_c < 1$) flows, sacrificial pile configurations can give up to 40 to 50% reduction in scour at the protected pier, with reduced effectiveness under live-bed conditions of $V/V_c > 1$ (Melville and Hadfield 1999) due to the passage of bed forms. Parker et al. (1998) conclude that sacrificial piles are an ineffective way to suppress scour under mobile-bed conditions. A significant consideration is that variation in flow alignment β typically reduces the effectiveness of the pile configuration in protecting the pier. Large flow skewness ($\beta > 20^o$) may result in the piles actually exacerbating scour at the pier.

In general, sacrificial piles are not recommended unless the flow remains aligned ($\beta = 0^o$) and the flow intensity is relatively low. Under such conditions, submerged and full-depth piles give similar reductions in scour. If sacrificial piles are to be utilized for such conditions, model testing to determine the optimum pile configuration is recommended, with the piles themselves needing to be designed against scour undermining. The effects of debris contamination on the performance of the piles also need to be considered, as do any effects of the piles on navigation.

11.4.4.7 Iowa Vanes As a countermeasure for pier scour, Iowa vanes (Section 11.3.4.10) are installed just upstream of the pier and angled inward, looking downstream (Fig. 11-13), with the vane configuration designed both to induce secondary currents that interfere with the horseshoe vortex and also to encourage sediment deposition in the region of local scour at the pier. Potential disadvantages of a field of vanes include the potential to collect debris, the potential for damage by sediment in motion, and possible decreased performance for skewed flows. Parker et al. (1998) conclude, however, that of flow-altering countermeasures, only Iowa vanes show enough promise to warrant further study. Comments on construction and maintenance of Iowa vanes are given in Parker et al. (1998).

By varying vane height H_v (and thereby submergence T for constant flow depth), vane angle of attack β, vane spacings z and e, vane length L, and longitudinal extent of vane field X for two flow velocities (Fig. 11-13), Lauchlan (1999) investigated the performance of vane configurations in terms of countering clear-water and live-bed scour at a cylindrical pier. The results of the tests indicate the angle of attack β and the streamwise spacing e to be principal parameters affecting the performance of vane configurations. The testing, although indicating potential scour reductions through use of the vanes, is not comprehensive, and further tests remain to

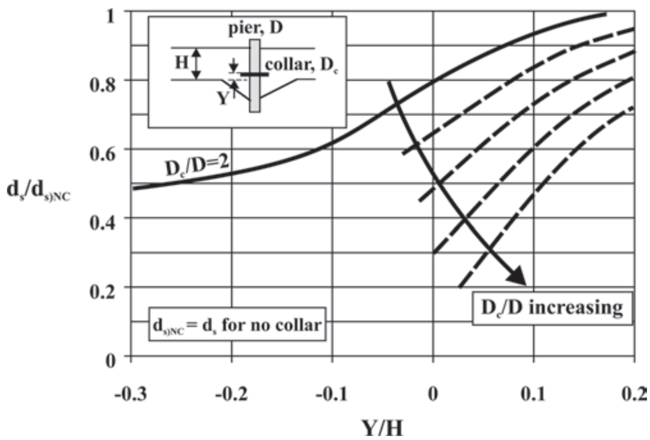

Fig. 11-14. The influence of horizontal-collar location and size on equilibrium scour depth d_s for a cylindrical pier (modified from Melville and Coleman 2000).

determine the degree of usefulness of Iowa vanes for control of pier scour.

11.4.4.8 Horizontal Collars Horizontal collars (Fig. 11-14) are designed to protect piers against scour by shielding the sediment bed from the downflow and horseshoe-vortex flow structures (Fig. 11-1) in the vicinity of the pier. These collars are thin in vertical section in order not to exacerbate scour. They have the potential disadvantage, however, of encouraging debris accumulation. The concept of using collars as scour countermeasures has been investigated by Schneible (1951); Chabert and Engeldinger (1956); Tanaka and Yano (1967); Thomas (1967); Ettema (1980); Dargahi (1990); Chiew (1992); and Fotherby (1992).

To date, collars have not been tested under live-bed conditions, and so they should not be considered for use other than in low sediment-transport conditions, such as may exist on floodplains or in vegetated channels.

For clear-water conditions, Fig. 11-14 summarizes the trends of available data on the influence of collar diameter D_c and location Y above the surrounding bed on scour reduction. The data indicate that a collar can reduce scour depth significantly. For maximum scour protection, the collar should be placed beneath the surrounding bed level. Scour depth for a circular pier can thereby be halved for a collar diameter twice that of the pier. Despite these encouraging results, Parker et al. (1998) do not consider horizontal collars to warrant further study or the development of user guidelines.

11.5 ABUTMENT PROTECTION

11.5.1 Introduction

Protection of bridge abutments from scour includes countermeasures that alter flow and scour patterns and those that armor the bed, bank, floodplain, and embankment slopes. Armor protection frequently includes the coverage of susceptible portions of embankment slopes. Many design guidance documents recommend that an apron be constructed around the toe of the embankment slope. Armor aprons can protect vertical-wall abutments founded on spread footings. Filters have been recommended below the protection to prevent piping of soils through the armor layers. The filters also may be beneficial to prevent winnowing of soils from beneath aprons, especially where the armor layer is used to protect embankments under live-bed conditions. There is evidence that fabric filters may be detrimental to the performance of armor protection where settlement and movement of the armor layer is necessary for the armor layer to conform to general bed degradation or scour hole formation.

Based on extensive field observations of flood-damaged bridges, Parola et al. (1998) suggest that under many circumstances where abutments are founded on piles of sufficient depth, prevention of progressive failure of spillthrough embankments may be detrimental to the protection of the bridge from scour. At many locations, failures of the approach embankments increase flow area substantially, reducing flow velocity and preventing the formation of deep abutment scour holes. This relief mechanism may greatly reduce the depth of scour at piers and the location of the abutment pile bents. This method may be acceptable at locations where such failures would pose no risk to bridge users and would have limited effect on the transportation network.

11.5.2 Failure Mechanisms

Lewis (1972), Macky (1986), Kwan (1988), Croad (1989), Kandasamy (1989), and Eve (1999) ran exploratory experiments on bridge abutments that showed the primary failure mechanism of spillthrough embankments to be the formation of scour holes along the toe of the embankment with subsequent mass failure of the embankment into the scour hole. Progressive failure of the abutment slope into the scour holes eventually leads to failure of the embankment and supported roadway. Observations of flood-damaged bridges confirm the laboratory observations (Parola et al. 1998). Bridge abutments supported on piles frequently are not damaged although sections of approach embankments and portions of roadway may be destroyed.

In a laboratory study of abutment scour protection methods typically used in practice, Macky (1986) found that scour and undermining of the slope protection was initiated at the upstream toe of the embankment. Pagan-Ortiz (1991) observed a critical zone where riprap failure was initiated on the apron at the point of flow separation from the upstream edge for vertical-wall abutments and along a separation line downstream of the spillthrough abutments. Both reports present observations that show a critical point for apron failure and initial undermining of the slope protection at or near the slope toe. Both experiments showed

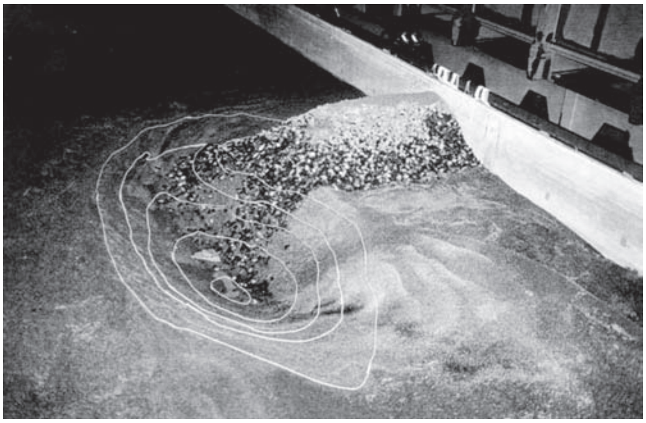

Fig. 11-15. Photograph showing laboratory study of riprap protection at an abutment (after Eve 1999).

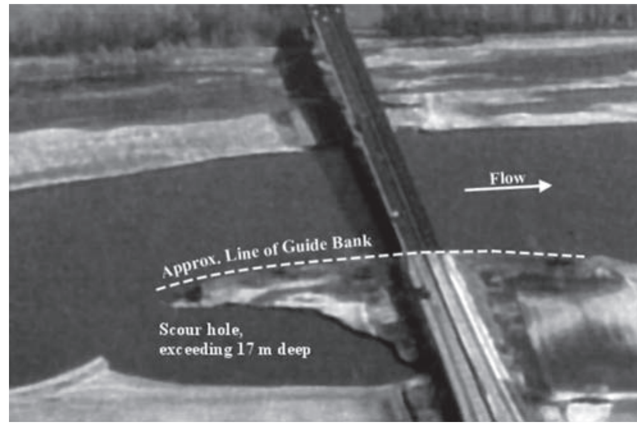

Fig. 11-16. Guide banks at Interstate 70 highway bridge embankment, bridge over Missouri River near Rocheport, Missouri, after 1993 flood.

subsequent failure of the embankment to be governed by mass movements such as translational slides of the granular fill materials used. Eve (1999) conducted clear-water scour experiments and a limited number of live-bed experiments on the failure conditions of slope and toe protection that included filters beneath the riprap. She found that failure under clear-water conditions was progressive, with failure of the protection being initiated at three different locations. These initial failure mechanisms were failure of the edge of the protection where the protection rolled into a scour hole, entrainment of pieces of riprap along the toe of the abutment, and entrainment on the upstream edge of the embankment slope. Undermining of the protection occurred as slope materials mass-failed toward the slope toe. Fig. 11-15 shows a downstream scour hole and the dispersal of riprap within the scour hole. Under live-bed conditions without a filter, winnowing of the particles beneath the riprap caused rapid failure of the protection.

11.5.3 Abutment Shape Design

For abutments that are sited near the edge of a channel, considerable reductions in local scour depth are associated with streamlined abutment shapes. The local scour at spillthrough abutments can be as much as 50% less than that at the same-sized vertical-wall abutment, for example. On the other hand, waterway contraction effects for a given bridge span are greater at a bridge founded on spillthrough abutments.

11.5.4 Guide Banks

Guide banks are curved embankments that extend upstream and, in some cases, downstream from and perpendicular to the abutment end. The use of guide banks was first introduced in 1888 for the construction of a bridge on the Chenab River, Pakistan (CBIP 1989). Guide banks extending upstream from the end of bridge embankments have been used to

- confine flow in braided rivers to the bridge opening;
- improve flow distribution and alignment through bridge openings;
- alter flow in bends that impinge on abutments; and
- transfer the point of highest flow curvature and deepest scour upstream of the bridge away from the abutment.

An illustration of the benefits of guide banks is presented in Fig. 11-16, which shows the Interstate 70 highway bridge embankment after the 1993 Midwestern U.S. flooding of the Missouri River near Rocheport, Missouri. Guide banks upstream and downstream of the Interstate 70 embankment transferred the formation of a deep scour hole upstream of the bridge. Although this scour hole was in excess of 17 m deep and caused the failure of the tip of the guide bank, the embankment slopes, including the toe of the highway embankments, were not damaged. The scour hole extended beneath the structure; however, the maximum scour depth under the structure was less than 8 m.

Guidance for the design of guide banks is presented in Neill (1973); Bradley (1978); Ministry of Works and Development (MWD 1979); CBIP (1989); and Lagasse et al. (1995). Lagasse et al. (1995) present detailed guidance that is based on the laboratory research of Karaki (1959; 1961), procedures developed by Bradley (1978), and experience of many U.S. state highway agencies. The main features of guide banks are their orientation with respect to the abutment face and embankment, plan view shape, upstream and downstream length, cross-section shape, and crest elevation. The recommended shape is a quarter of an ellipse with upstream length (L_s) equal to 2.5 times the offset length; see Fig. 11-17. The alignment of the guide bank should be parallel to the face of the abutment in the

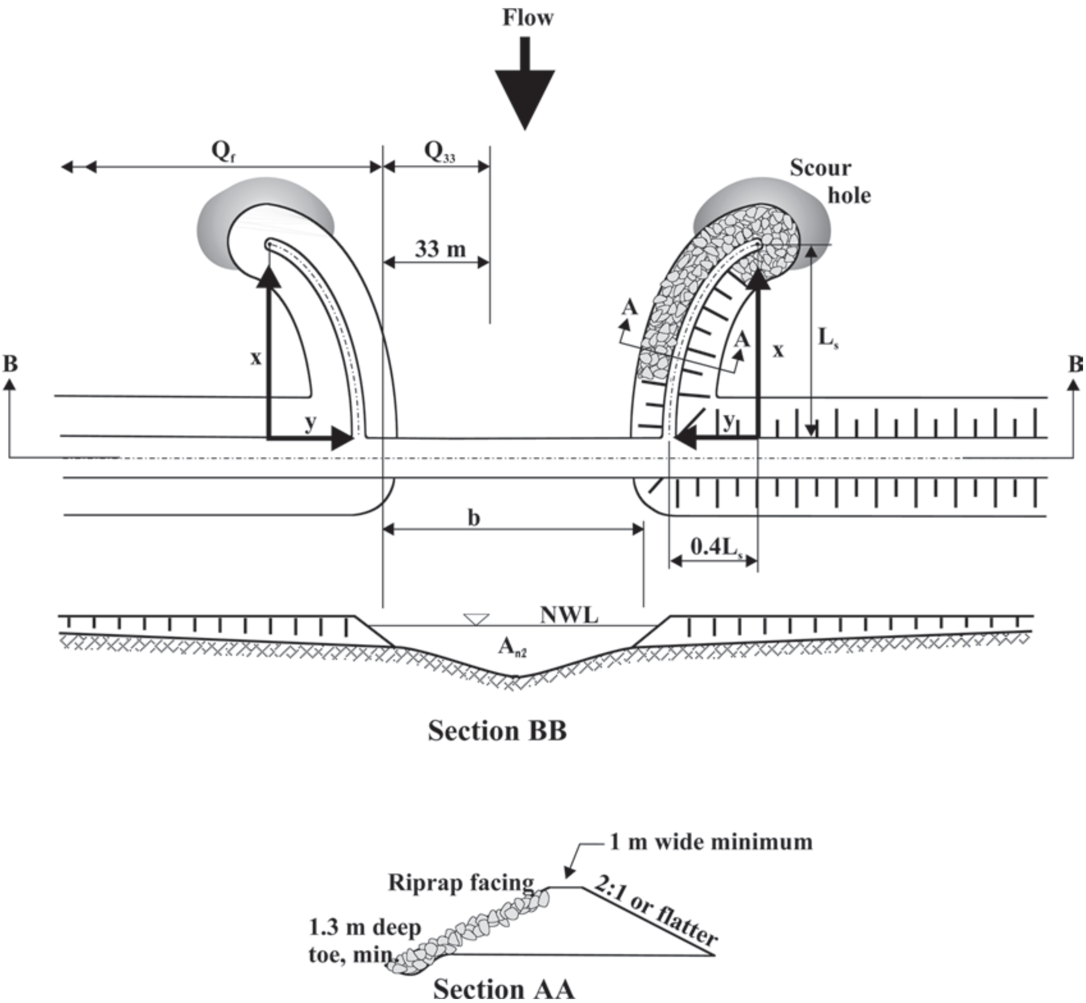

Fig. 11-17. Typical guide bank details (modified from Lagasse et al. 1995).

bridge opening. The plan view coordinates for the crest can be determined from

$$\frac{x^2}{L_s^2}+\frac{y^2}{\left(0.4L_s\right)^2}=1 \qquad (11\text{-}3)$$

The length, L_s, is determined from the nomograph in Fig. 11-18 developed from the studies of Karaki (1959; 1961) and Neeley (unpublished report, U.S. Geological Survey, 1966). In the nomograph, Q is total discharge of the stream; Q_f is lateral or floodplain discharge of either floodplain; Q_{33} is discharge in a 33-m width of stream adjacent to the abutment; A_{n2} is cross-sectional flow area at the bridge opening during normal stage; $V_{n2} = Q / A_{n2}$ is average velocity through the bridge opening (m/s); Q_f / Q_{33} is guide bank discharge ratio; and L_s is projected length of guide bank (m).

The use of the nomograph should be limited to a minimum length of 16 m and a maximum length of 82 m. Lagasse et al. (1995) recommend that guide banks should not be shorter than 16 m or longer than 250 m. Experience indicates that a standard length of 50 m has performed well. Shorter lengths have been used successfully when the upstream end of the guide bank was extended to a tree line where the roughness of the trees reduced velocities at the tip of the guide bank.

The crest of the guide bank should be placed at least 0.6 m above the design flood elevation to prevent flows over the guide banks that may be damaging to the bridge. A downstream guide bank of length 16 m is used to prevent rapid expansion of flow in some U.S. states. Riprap protection is recommended for the channel side of the protection and the upstream tip of the guide bank. Rock protection may not be necessary at locations where vegetation will reliably protect guide banks.

11.5.5 Design Criteria for Riprap Protection

Model studies have shown and field observations have confirmed that failure of spillthrough abutments is progressive, in contrast to piers, which can fail catastrophically. The

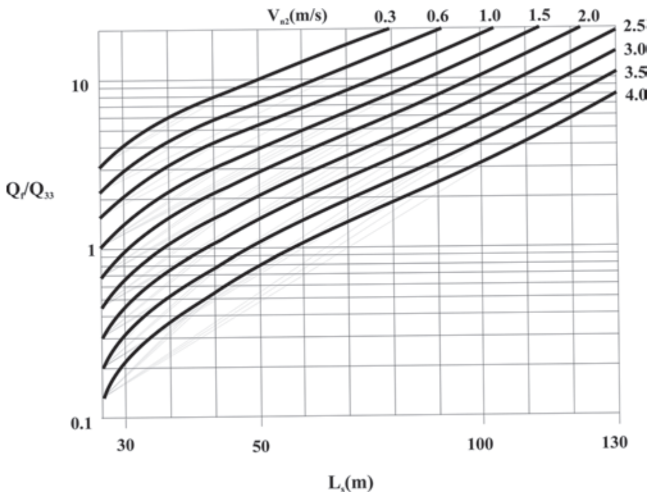

Fig. 11-18. Nomograph for selecting the length of guide banks at bridge crossings (modified from Lagasse et al. 1995).

design of countermeasures may require that partial failure of the armor protection be acceptable. For extreme event design and where abutment foundations are supported on piled foundations, complete erosion and failure of the embankment and supported roadway may be acceptable as long as the piles have sufficient depth to resist failure of the embankment.

Research on the performance of riprap protection on abutment slopes and aprons was conducted by a number of researchers including Simons and Lewis (1971); Lewis (1972); Macky (1986); Croad (1989); Simons et al. (1989); Pagan-Ortiz (1991); and Eve (1999). These laboratory studies were conducted under clear-water conditions. Design guidance is presented by the Ministry of Works and Development (MWD 1979); Gregorius (1985); Harris (1988); Richardson et al. (1988); Brown and Clyde (1989); Central Board of Irrigation and Power (1989); Austroads (1994); Richardson and Davis (1995); and Lagasse et al. (1997).

11.5.5.1 Riprap Size Simons and Lewis (1971), using the research of Lewis (1972), developed a method for predicting the stability of rock based on detailed velocity measurements around and on the slopes of spillthrough abutments. The method requires the use of a two-dimensional numerical model and the determination of the velocity one rock diameter above the streambed. Croad (1989), based on a limited number of small-scale model tests and velocity measurements, developed an equation to predict the critical conditions for initial failure of riprap protection at the toe of the spillthrough abutment. Croad's method requires a depth-averaged velocity measured over the critical failure point at the abutment toe. He recommended that the depth-averaged velocity over the critical failure point be estimated as 1.5 times the average approach flow velocity.

Pagan-Ortiz (1991) conducted fixed-bed small-scale experiments on the stability of riprap on a spillthrough abutment with side slopes of 2H:1V and on vertical-wall abutments. Flow conditions were adjusted until failure conditions were observed near the abutment. Critical failure zones were found on the streambed protection at or near points of flow separation from the upstream end of the rectangular abutments and downstream of the flow separation point on the spillthrough abutment. Prediction equations for the critical conditions were developed based on the average contracted flow velocity measured in the experiments.

A. T. Atayee (Unpublished TRB paper No. 931021, 1993) extended the research of Pagan-Ortiz (1991) to include compound channel geometry. Critical failure zones similar to those described in Pagan-Ortiz (1991) were observed. The data of both Pagan-Ortiz (1991) and Atayee were used to develop two equations that are based on the contracted flow velocity on the floodplain portion of the bridge opening (Atayee et al. 1993). They recommend that a two-dimensional depth-averaged numerical model be used for determining the average contracted flow velocity on the floodplain. The method presented in Atayee et al. (1993) is recommended as design guidance by Richardson and Davis (1995), although ad hoc recommendations were added to allow for prediction of contracted flow velocity on the floodplain based on cross-section averaged contracted flow velocity and the width of the floodplain in the contracted bridge opening.

A list of the riprap sizing equations for abutment protection is provided in Table 11-9. The equations of Simons and Lewis (1971), Croad (1989), and Atayee et al. (1993) for $\mathsf{F} < 0.8$ can be arranged into the form

$$\frac{D_r}{H} = \frac{C}{(S_s - 1)} \mathsf{F}^2 \tag{11-4}$$

where C is a coefficient. For $\mathsf{F} < 0.8$ and flat-bed conditions, the Simons and Lewis (1971) relation at the critical location of failure can be considered identical to that of Atayee et al. (1993) if the local velocity one rock diameter over the bed is 1.15 times the average contracted flow velocity on the floodplain. For the same flow range and conditions, the Croad (1989) equation can be considered identical to that of Atayee et al. (1993) if the depth-averaged velocity at the critical point of failure is 1.48 times the average contracted flow velocity on the floodplain.

Simon and Lewis (1971) and Croad (1989) both recommend that down-slope gravitational force should be considered in determining the size of rock on the abutment slope. They recommend increasing the rock size according to the relation provided by Lane (1955). The studies by Ulrich (1987) and Maynord (1995; 1996), as described in Appendix B, indicate that the theoretical slope adjustment factors used by Lane (1955) may be as much as 35% larger

Table 11-9 Equations for Sizing Riprap at Abutments

Reference	Applicability	Equation		Symbols
Simons and Lewis (1971)	Spillthrough abutments	$\eta = \dfrac{0.4U_r^2}{(S_s - 1)\, gD_r}$	(11-19)	D_r = riprap stone size U_r = velocity at a level of one rock diameter above the bed H = approach flow depth S_s = specific gravity of rock η = stability factor = 0.595, for flow over a horizontal bed
Croad (1989)	Spillthrough abutments	$D_{r50} = 0.025\, U_b^2\, K_{sl}^{-1}$ $K_{sl} = \sqrt{1 - \dfrac{\sin^2\phi}{\sin^2\theta}}$	(11-20)	U_b = velocity at abutment end K_{sl} = embankment slope factor ϕ = slope angle θ = angle of repose F = Froude number of approach flow = $U/(gH)^{0.5}$
Brown and Clyde (1989)		$D_{r50} = \dfrac{0.006\, U^3}{H^{0.5} K_{sl}^{1.5}} \left(\dfrac{S_f}{1.2}\right)^{1.5}$	(11-21)	S_f = stability factor varying from 1.6 to 2.0 for abutment protection
Pagan-Ortiz (1991)	Vertical-wall abutment	$D_{r50} = \left(\dfrac{1.064\, U_2^2\, H_2^{0.23}}{(S_s - 1)\, g}\right)^{0.81}$	(11-22)	U_2 = mean velocity in contracted (bridge) section H_2 = flow depth in contracted section
	Spillthrough abutment	$D_{r50} = \dfrac{0.535\, U_2^2}{(S_s - 1)\, g}$	(11-23)	
Austroads (1994)		$\dfrac{D_{r50}}{H} = \dfrac{1.026}{(S_s - 1)} \mathsf{F}^2$	(11-24)	
Atayee et al. (1993) and Richardson and Davis (1995)	$\mathsf{F}_2 \le 0.8$	$\dfrac{D_{r50}}{H_2} = \dfrac{K_s}{(S_s - 1)} \mathsf{F}_2^2$	(11-25)	K_s = shape factor = 0.89 for spillthrough abutments = 1.02 for vertical-wall abutments
	$\mathsf{F}_2 > 0.8$	$\dfrac{D_{r50}}{H_2} = \dfrac{K_s}{(S_s - 1)} \mathsf{F}_2^{0.14}$	(11-26)	F_2 = Froude number in the contracted section K_s = 0.61 for spillthrough abutments = 0.69 for vertical-wall abutments

than data would indicate. If the rock protection on the slope is sized according to Atayee et al. (1993), then consideration should be given to use of slope correction factors by Ulrich (1987) and Maynord (1995; 1996).

11.5.5.2 Extent of Rock Protection Under clear-water conditions, Pagan-Ortiz (1991) found that an apron that extended along the toe of the abutment from the point of tangency on the upstream side of the abutment to the point of tangency on the downstream side of the abutment and extended a distance equal to two times the flow depth away from the toe of the abutment was adequate. Atayee et al. (1993) recommended that the width of the apron not exceed 7.5 m.

Eve (1999) conducted riprap tests with approach flow conditions at 90% of the approach shear stress required to mobilize the approach sand bed. Based on her observations of progressive failure of the abutment embankments, she developed the following relation for determining the extent of protection,

$$\frac{W}{H}\left(\frac{0.5W + r}{H + r}\right) = \left(0.5 - 1.82\frac{D_{r50}}{H}\right)\left(\frac{B}{B - L}\right)\left(\frac{180}{180 - (\theta + \phi)}\right) \tag{11-5}$$

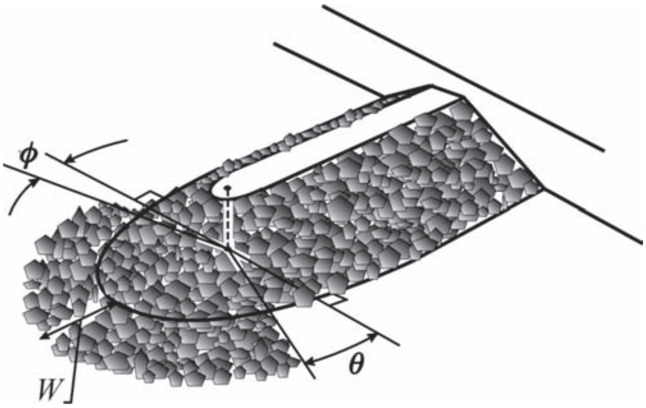

Fig. 11-19. Definition diagram for placement of a riprap launching apron at a spillthrough abutment (after Eve 1999).

where

H = approach flow depth;
B = upstream width of the flume;
L = abutment length;
r is the radius of the spillthrough abutment toe;
and W, θ, and ϕ are defined in Fig. 11-19.

Macky (1986) examined typical New Zealand practice, rather than recommended practices, in small-scale model studies in which the protection on the slope was extended only slightly below the previously existing bed level and no apron was provided. He reported that riprap failed into scour holes that formed around the abutment; however, the slumped riprap armored part of the scour hole. Additionally, the remaining slope angle decreased and was armored by riprap that translated down slope. The tested abutment was substantially undamaged. Macky also found that for aligned flows, the downstream side of the abutment required only nominal protection.

Riparian and floodplain vegetation may provide adequate protection where shading beneath the structure does not prevent its growth. Although insufficient data are available for reliance on vegetation at the critical failure points, vegetation may provide reliable protection on the upper slope areas and at locations upstream and downstream of the bridge. Substantial reduction in the extent of armor protection may be possible. In practice, sufficient protection has been provided to bridges by armoring the area beneath the bridge superstructure. Unfortunately, laboratory studies have not modeled the effects of vegetation under similar conditions. Armor protection should be considered at all critical locations where unraveling of the vegetal cover may be initiated, such as on the toe of the abutment and around piers located within the high-velocity flow of the abutment and any areas where shading may prevent the growth of erosion-resistant vegetal covers. Use of two-dimensional numerical models to determine the extent of armor protection required and the appropriate locations for vegetation should be considered.

11.5.5.3 Thickness of Riprap Although specific tests on the thickness of riprap protection on abutment slopes and aprons have not been conducted, information on the thickness of riprap revetments for stream banks and streambeds is provided by Maynord (1995; 1996) and is given in Appendix B. Where riprap is placed in water, on fine-grained sediment without a filter, or where extensive scour holes are anticipated at the edges of the protection, increased riprap thickness may be warranted. Lagasse et al. (1997) suggest that the thickness should not be less than the larger of either 1.5 D_{r50} or D_{r100} and should be increased by 50% when it is placed underwater to provide for uncertainties associated with underwater placement methods.

11.5.5.4 Filter Requirements An exploratory study by Eve (1999) showed the need for filters at abutments under live-bed conditions. In a very limited number of live-bed experiments, complete failure of approach embankments, initiated by bed-form undermining and winnowing of the bed material beneath the riprap, was observed on the apron as well as on the slope. On the other hand, Macky (1986) and Eve (1999) reported stable riprap configurations in several clear-water experiments in which filters were not used. Other factors such as groundwater flows from such sources as surface runoff may necessitate the use of filters on slopes. Additional research is needed to determine the benefit of placing riprap under the apron and on the slope, especially where riprap is designed to conform to adjacent scour holes or bed-form undermining.

11.5.6 Alternatives for Protection of Abutments

Where riprap of adequate size is unavailable or where environmental or geometric constraints preclude use of riprap, alternatives to riprap are necessary. Lagasse et al. (1997) describe several armoring alternatives for abutment slopes, including articulated concrete block (Section 11.3.4.3), articulated grout-filled mattresses (Section 11.3.4.4), soil cement, wire-enclosed mattresses, interlocking armor units (toskanes, Section 11.3.4.3), and cement-filled bags (Section 11.3.4.3).

As part of the study was completed to evaluate the performance of typical rather than recommended methods of protecting spillthrough abutments, Macky (1986) examined the performance of several alternatives to riprap including: interlocking concrete armor units (akmons), concrete mattresses (Section 11.3.4.4), gabions (Section 11.3.4.5) laid on the embankment slopes, gabions stacked horizontally and staggered up the slope, and boulder-filled wire baskets laid on the bed beneath the stacked gabions. Although a very limited number of tests were conducted, several important aspects of abutment protection were revealed. Although the interlocking armor units behaved similarly to riprap, their interlocking capabilities appeared to hinder dispersal on the slope after toe scour undermined the protection. Tests on riprap showed that dispersal of the rock is a key factor in the ability of the protection to adjust and conform to scour holes and subsequent slope failures. Consequently, large areas of the slope were left unprotected after slope failures occurred. Macky (1986) recommended that noninterlocking shapes be considered. Concrete mattresses, gabions, and wire-filled baskets generally performed poorly, because of their inability to adjust and conform to scour holes and slope failures. As the experience of U.S. state highway agencies has shown, use of rigid concrete pavements (Section 11.3.4.4) on abutment slopes suffers from the same problems. Preexcavation of the scour hole at the toe of the slope and extension of the slope protection to the depth of scour were recommended as a possible way to improve the performance of mattress- and gabion-type countermeasures. An apron using these techniques may also improve the performance of these techniques.

The work on riprap protection on abutments, coupled with the work by Macky (1986), clearly shows that design of spillthrough abutment protection should either extend to a depth near to the scour depth, provide an extensive apron, or conform to progressive scour hole formation and slope failure.

11.6 ENVIRONMENTAL CONSIDERATIONS

Countermeasures for bridges should be constructed so that they enhance aquatic habitat and bridge-crossing aesthetics rather than degrading them. Selection of the size distribution of rock for armor protection should satisfy requirements for stream stability and habitat. Grade control structure drops should be selected to provide for fish migration. Consideration should also be given to countermeasure placement methods and their impact on aquatic habitat.

Although the full spectrum of potential flows at bridges should be considered, the countermeasure should be designed to protect the bridge for design flood events (500, 100, and/or overtopping event), maintain channel stability at bank-full levels, and enhance stream habitat at average annual and lower flow levels. Stream restoration concepts and habitat objectives given in Chapter 9 should be incorporated into the countermeasure designs at the bridge; however, highway right-of-way limits and cost may be apparent barriers to extensive modification of stream channels as part of bridge-scour countermeasures. Use of structures such as spur dikes, barbs, and bendway weirs provide nonuniformity to flow and topography through the bridge opening, which generally improve habitat: conversely, use of uniform rock revetments and other uniform topography and material configurations will tend to degrade habitat. Vegetation, especially riparian trees, should be used to protect streambanks where safety of the bridge is not compromised.

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