RECLAMATION

Managing Water in the West

Bank Stabilization Design Guidelines

Report No. SRH-2015-25
Albuquerque Area Office
Science and Technology
Policy and Administration (Manuals and Standards)
Yuma Area Office

U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center
Denver, Colorado

June 2015

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Technical Service Center, Denver, Colorado
Sedimentation and River Hydraulics Group, 86-68240

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Acknowledgements

This document was prepared using funding from the Albuquerque Area Office, Reclamation's Science and Technology Program, Technical Service Center Manuals and Standards, and the Yuma Area Office. Meg Jonas of the U.S. Army Corps of Engineers, and Jon Fripp of the Natural Resources Conservation Service, provided their input and review of the topics covered and practical aspects of guideline contents. Robert Padilla of the Albuquerque Area Office reviewed drafts of this guide and provided valuable comments. Peter Martin of the Technical Service Center formatted the document and organized figures and tables. These contributions are gratefully acknowledged.

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1 Introduction

This manual was prepared to provide Reclamation personnel with updated guidance on deploying effective bank stabilization methods. Looking back several decades, the two measures of a successful river design were feasibility and sustainability. Feasibility was measured in terms of cost effectiveness and public safety. A feasible structure did not harm the public, and fulfilled its intended function at the lowest possible construction cost. Sustainability was defined as the ability to withstand river changes with minimal or no maintenance for the life of the project. The words “bank stabilization design” were commonly synonymous with sizing riprap.

In recent decades, environmental effectiveness and sustainability have gained recognition as a third fundamental measure of project success. Environmental effectiveness can be evaluated on two levels: (1) The degree to which the design is compatible with river and geomorphic processes; and (2) The amount of disruption of the riparian habitat and natural environment. Environmental and geomorphic sustainability is reduced when the design is lacking in geomorphic compatibility.

A goal of these guidelines is to increase considerations of sustainability and long-term feasibility, and to increase environmental effectiveness in bank stabilization designs. Most of the bank protection methods presented in this manual can be applied in ways that meet all three measures of a successful design: feasibility, sustainability and environmental effectiveness. Although traditional methods of hardening a bank with a full riprap revetment often fail the test of environmental effectiveness, this method is still necessary under specific conditions and is included if other methods cannot be used or do not meet project objectives.

1.1 Challenges of Modern River Design

Beginning in the late 1800’s and early 1900’s, the general approach in river engineering was to control and develop the resource for economic benefit. Peaking in the 1950’s and 1960’s, this type of river engineering relegated geomorphology to the level of a neglected, if not a forgotten, science. Concrete dams reduced flows, concrete drop structures took out energy, and river floodplains could more readily be confined to a fraction of original width by levees. Constricting floodplains opened up additional land to development but today a wealth of experience and the prevailing science indicate that hardening banks and significantly reducing the floodplain can lead to longer term maintenance costs not included in original economic analysis. Flow constrictions and natural channels confined by levees and hardened banks also have significant ecological cost. With many illustrations in our past, today's designers should look first to re-establishing floodplain processes, and to use geomorphic principles and processes to promote ecological improvements and sustainability in bank stabilization projects.

Large temporal and spatial diversity of river conditions, both physical and environmental, make design of river projects a challenging prospect. Methods used in one hydrologic, geologic, or ecological setting may not apply to another location, and different goals may require different designs for the same river system. The traditional use of riprap revetments is no longer sufficient for every location. Guidelines for the design of multiple methods to bank stabilization are presented here to help address the complexities of modern day projects. Illustrations of the complexities of modern river engineering are highlighted in the paragraphs below.

Geomorphic Factors. The success of a river design partially rests on the successful identification of geomorphic and sediment transport factors in the planning stages of the project. Geomorphic factors impact the longevity and function of the project and should be considered both in planning and throughout design. A geomorphologist will consider both historical and current conditions to evaluate the project site. Unstable conditions including planform transitions or sediment erosion and deposition at the site need to be identified before an appropriate method of bank stabilization can be selected. If not addressed in the method selection, the instability in some instances can damage or fail the structure prematurely.

Multi-disciplinary Teams. A second means of improving project outcome is to benefit from a multi-disciplinary team during the development of the design. In addition to consulting a geomorphologist or river engineer during project planning, a plant specialist can provide critical input on planting plans and the handling and installation of "materials" that are part of the structure during project design. Depending on the project site and goals, a review by a stream ecologist, wildlife specialist or fish biologist may also help steer the project clear of some non-intuitive shortcomings. Creative stake-holders and land right-of-way/acquisition specialists can be invaluable in looking at better long-term solutions.

Risk-based Approach to Design. Hydrologic regimes and risk-based assessments are central considerations to water resources design. A structure designed for a 50 yr-flow event can be damaged in the second year by a 100-yr flow event. If the risk level was accurately assigned, this may be an acceptable hazard. A second aspect of risk analysis to be considered is the exposure of the river site during construction. Although ultimately it may be the contractor's decision, consideration of construction sequencing and exposure during design should be considered to improve project success. There is a third aspect to risk assessment when working with plant materials. Plants provide varying levels of bank protection depending on the age of the plant or the plant coverage. The risk of bank failure during the first years of plant establishment can be higher until the plants are well established. This level of risk can be accepted, or the designer can use additional methods to temporarily strengthen the bank while plants establish. Risk analysis is included to improve bank stabilization planning, design and construction.

Planned Monitoring and Maintenance Actions. This approach requires dispelling the negative associations of planned maintenance and monitoring actions. Planned post construction actions are a positive integration of the design into a complex system, not an indication of a poor design. Good designs that meld structural features with the environment may require irrigation or a post construction visit within the first three years of construction, or may require a visit following a high flow event whenever it occurs. Irrigation, monitoring or a maintenance effort after construction can fine tune the project to the high variability in river conditions, and ensure the establishment of living materials. Post-construction actions should be integrated into the project funding mechanism, early in the planning stages of project development.

Non-standard Construction Materials. Successful bank stabilization projects in terms of feasibility, sustainability and environmental effectiveness are often constructed with a mixture of standard and non-standard construction materials. Some standard materials used in fixed river structures (drop structures, dams, diversions, etc) should not be used in river banks and beds where river processes are being preserved. Materials to avoid include geotextile filters (use granular filters only), concrete blocks, and cables used to secure materials. Channel lining such as highway drainage or local urban runoff channels are different design process than strictly riprap, and should include flexible linings covered in FHWA (2005).

Acceptable non-standard construction materials for rivers include live plants, willow cuttings, logs, woody debris, coir fabric, or straw bales. Non-standard materials like plants are complex and may be more challenging for the designer and contractor initially. Plants come as widely varying species; can come as seeds, in tubes, in pots or as cuttings; can be used at different ages and have varying requirements; and the region and climate of the project, in addition to these other factors, will all have an effect on the use and outcome. Yet plants like the willow cutting can be one of the most functional and enduring materials used in the construction of bank restoration projects. Although there may not be standards in place ensuring the quality of these non-standard construction materials, well-written specifications on the acquisition, handling and installation of these materials, and consideration of construction sequencing for planting, can both contribute substantially to the reliability of non-standard materials. Development and incorporation of new design methods and specifications into bank stabilization projects are challenging initially, but can produce benefits in the improved performance of bank stabilization projects.

Ecosystem Services. In the last two decades there is growing awareness for the contributions of nature to human welfare. Ecological services are units to be tracked and measured as part of a nationwide assessment of environmental welfare and performance. This system would be a green version of the Gross Domestic Product (GDP), the market value of all officially recognized final goods and services produced in a country in a year. It is difficult to select an accurate means of measuring and tracking environmental services. Three issues in the development of a green GDP are selection of standardized units for accounting and procurement, value assignment for services not bought and sold in public markets, and prevention of double counting services by distinguishing between intermediate and final goods. Boyd and Banzhaf (2006) remind readers that the current debate over a green GDP is similar to the debate over the definition of goods and services for the conventional GDP, which took place in the last 100 years. Although the measures and accounting for the GDP are now accepted, the current approach is the result of decades of debate within government and the economics profession. Despite debate, there is an increasing drive for more accurate accounting of eco-system services. Subsequently, the condition of floodplain habitat, like wetlands area, may be better monitored in the future and improved accounting of ecological services can be expected to make designs that steepen and harden river banks, to the detriment of the floodplain, increasingly more difficult to justify.

The intent for this Guide is to help ease engineers into the increased challenges of modern bank stabilization design. It is not an all-inclusive Guide, but should help steer the designer towards a well-designed bank stabilization project that is feasible, sustainable and environmentally effective. This goal is aligned with Reclamation’s mission to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

1.2 Bank Stabilization

Traditional bank protection methods have been used to prevent erosion and bank slips, and to reduce the hydraulic load acting on the soil (Hey 1994; Brookes 1988; Escarameia 1998; McCullah and Gray 2005). In this Guide, discussions of bank protection methods are focused on cases where erosion of the bankline and toe is the primary mechanism for bank failure. This includes small bank slope failures or slump block failures. In situations where the bank slope is unstable due to geotechnical processes, other methods would need to be applied in addition to bank stabilization (Escarameia 1998), and additional resources should be consulted in addition to these guidelines.

Traditional riprap protection methods that harden the bank and non-traditional uses of native materials to control erosion are both presented in these guidelines. In addition to these measures, methods that address the geomorphic cause of high erosive energy are included. More than half the solutions in the Guide do not armor the bank, but instead accommodate river processes in ways that alleviate erosive pressure. These methods include preserving the floodplain and extending floodplain connectivity, adding channel elements that relieve pressure and restore channel processes, channel relocations, and the addition of transverse flow elements. All of these methods are designed with a goal for a sustainable and stable channel.

1.3 Project Development

Most projects proceed along the path of planning-design-construction, and this approach is generally assumed though out these guidelines. Two alternative approaches may also be suitable: design-build and adaptive management (a repeating cycle of monitor/evaluate-partial design-partial construction); however, these pathways are not directly addressed in this manual. The suitability of alternative pathways for a project or program should be determined on a project-by-project basis. There are both large benefits and large shortcomings associated with each.

This manual begins at the point where a determination has been made to address a perceived bank erosion concern. The design team should select the appropriate bank stabilization method, but only after they have reviewed project needs, defined project requirements, and assessed the site findings on geomorphic, energy (hydraulic), and scour conditions. Once this information has been assimilated, the team can proceed with the selection of a suitable method. Selection is followed by completion of the analysis and full development of the design, including drawings, sequencing and specifications for construction of the bank stabilization method. Selection of a bank stabilization method may occur in the planning (feasibility) stages of larger projects or, could occur on smaller projects as part of the initial 30% development of the design. Selecting a method and assigning a budget prior to adequately assessing the site may lead to a poor outcome.

A basic assumption in these guidelines is that the design team will be assisted by a river engineer or geomorphologist with a working knowledge of river processes and sediment transport at the project site. The science of river geomorphology is introduced but not detailed in this manual. An understanding of geomorphic properties and the identification of geomorphic factors is an essential contribution to the selection of a bank stabilization method and the development of the design. Similarly, a comprehensive chapter on plant ecology is not included in these guidelines. It is assumed the design team will include a plant specialist on the team during development of designs incorporating plants, development of the construction sequencing, and during the preparation of construction specifications. A third assumption is that the specifications writer will be an integrated team member who can assist and be assisted by other members, including the plant specialist. Finally, consultation with a land acquisition or easement expert, and creative teaming with stakeholders to expand floodplain are desired inclusions to the successful design team, and descriptions of their expertise are not included here.

1.4 Organization of Design Guidelines

Initial chapters of these design guidelines are organized similar to the progression of bank stabilization design procedures. The project design begins with defining the project and developing an understanding of the site factors and conditions as described in Chapters 2 to 5. Selection of a bank stabilization method is described in Chapter 6 with the aid of information and results from the site analysis, developed in previous chapters. In the last half of these guidelines, design procedures for each bank stabilization method are organized as sections within Chapters 7 to 13, and Chapter 14 concludes with a summary of guidelines and a discussion of future directions.

PART I. DEFINING THE PROJECT AND SITE ANALYSIS

Chapter 2. Project Requirements and General Assessment. Essential project information is identified including the determination of project requirements and general site conditions. Design criteria, site hydrology, the risk assessment, habitat and ecological needs, and established budgets are assembled for a complete picture of project requirements. General site conditions are represented through descriptions and maps of terrain, geology, soils, groundwater, vegetation and regional climate.

Chapter 3. Geomorphic Assessment. Required information on the geomorphology at the site, including the river form, and the vertical and lateral assessment of stability are identified.

Chapter 4. Hydraulic and Sediment Transport Assessment. Contains guidance on the hydraulic analysis to determine river energy and shear erosive forces, and to learn more about sediment transport at the site.

Chapter 5. Potential Scour. Presents a prescribed approach to analyzing the scour potential at the site.

Chapter 6. Selecting a Bank Stabilization Method. The information acquired to this point is assembled for the selection of a suitable bank stabilization method. Strengths and weaknesses of each method are described along with other pertinent criteria

PART II. PROJECT DESIGN

Chapter 7. Preserving the Floodplain. This brief chapter describes methods that preserve a floodplain whether it is through reestablishment, reconnection or expanding floodplain area. Preserving the floodplain is the preferred alternative whenever it is possible. Infrastructure relocation or setback is included in this chapter along with conservation easements, vegetation buffer zones, island/bank clearing and destabilization, and means of integrating floodplain preservation with other methods.

Chapter 8. Re-establishing the Floodplain. These methods include habitat enhancements and mitigation measures of longitudinal bank lowering (compound channels), constructed or enhanced side channels, and constructed bankline embayments or backwaters.

Chapter 9. Deformable Banks and Vegetation. Included in this chapter are the concepts of deformable banks and design procedures for live-vegetation protection including plant poles and fabric encapsulated lifts.

Chapter 10. Woody Debris and Boulders. Includes native material revetments, engineered log jams, large woody debris, rootwads, and boulder clusters.

Chapter 11. Relocating Channels. The steps of constructing channel elements or re-aligning a channel are outlined.

Chapter 12. Transverse Features. Transverse features are structures constructed perpendicular or angled to the flow, and include indirect flow deflection structures. These methods can be constructed at a lower cost and have increased environmental benefits, but also have a higher risk of erosion. A new design procedure has been developed for these guidelines. The guidance was developed from Reclamation funded laboratory studies at Colorado State University of a trapezoidal prismatic physical model. Studies addressed the reduction in bankline velocity that results from the installation of vane and spur dike transverse features.

Chapter 13. Bank Hardening. This chapter focusses on the traditional use of riprap as methods that harden the bankline and prevent future channel adjustments. Included are riprap revetments, rock trenches, rock windrow, and longitudinal stone toes.

Chapter 14. Future Directions. Main points of these guidelines are revisited with a discussion of future needs and proposed future directions.

PART I – PRELIMINARY INVESTIGATIONS AND METHOD SELECTION

2 Project Requirements and General Assessment

At the beginning of the project, the initial focus is on determining the project requirements that will define the design, and assembling information for site analysis and drawing development. Identifying project requirements includes developing the design criteria and the project risk assessment. Site hydrology is required for both developing the risk assessment and as a basic element of the site analysis.

2.1 Design Criteria

Project objectives are translated into the design criteria that define the project and guide the design, making development of design criteria a high priority in the design process. Exploring project objectives at startup of the project helps limit the changes and costs that can surface later in the design. A set of design criteria can be developed from project management, a review of project documents and agreements including environmental assessments (EA) or environmental impact statements (EIS), and by actively canvasing stake holders. Agreement should be reached from all parties on design criteria shortly after project startup and no later than 30% into the design to avoid costly changes.

Design criteria should specify the design life of the structure, a factor also needed for the risk assessment developed concurrently. The criteria may contain multiple definitions of flow conveyance including low flows, bankfull flows, peak flows or flow durations. Design criteria should include any habitat or ecological needs that have been agreed to by all parties. Design criteria should be very specific to avoid costly changes later in development of the design. When possible, objectives should be quantified, with statements like *x* acres of riparian bankline restored, near bank velocities reduced below *x* feet per second (fps), bank retreat limited to *x* ft per year, *x* acre-ft of sediment storage, and *x* sq. ft of woody debris added. Known scheduling constraints or other limitations should be identified including construction windows, restricted access, and maximum loss or minimum gain of specific habitat (e.g.. cottonwood-willow, elderberry shrubs, invasive plants of concern) within a defined interval.

A second level of design criteria or requirements can be added in some projects if the project has progressed to the development of typicals and dimensions for channel and structures, or for habitat requirements. The decision to include this information as a basic design requirement should be decided on a project basis. In some cases it can limit the designer if future discoveries and developments make this approach obsolete. In other cases, the inclusion of a typical design helps ensure reaching the agreed-upon goals of the project.

2.2 Ecological Goals

To develop the ecological goals, first determine the desired ecological factors for the project site including aspects of species habitat such as river form, vegetation, hydraulics, water quality, temperature, substrate types and sizes, and sediment transport. Then define what is valuable at the project site and identify the characteristics or features to be protected or enhanced as part of the project design. At a minimum the design should "do no harm". Harm can be recognized as multi-decade efforts to undue the collateral damage from projects designed with a myopic view for design criteria and a focus on short-term construction savings. Enlist the aid of specialists and stakeholders in defining ecological goals and incorporate goals into the project through specific design criteria.

To develop the designs that will meet the ecological goals, understand the current condition and historical form of the river system near the project site and determine the needs of the species of interest. The historical river form is most often the habitat that meets the species needs. One of the biggest challenges in designing for ecological goals is defining the flow stages and the physical characteristics of the species requirements for all pertinent life stages. Species requirements can be sought from both national specialists and local biologists or ecologists who have studied and reported important habitat conditions. Determine the geometric, geomorphic, and ecological characteristics of the existing project site and understand how different bank stabilization methods could alter, for good or bad, the present environment and river form. This effort is not as daunting as it sounds. On larger projects this information is available in the project Environmental Impact Statement (EIS). On smaller projects and for projects in general, these rules-of-thumb can provide some guidance.

- Recover, do not reduce, the floodplain
- Maintain flow disturbances in the form of occasional large peak flows in the flow regime
- Enhance or protect the river form from which the ecology stems. For example, a salmon thrives in a high energy, meandering channel while a whooping crane prefers wide, braided river channels.
- There is less maintenance and more benefits when flow is conveyed in a natural system versus a pipe or ditch
- Riprap habitat can be equivalent to a desert monoculture, so therefore limit riprap use to short transitions and submerged toe protection
- Channel complexity benefits more species
- Riparian, non-invasive, vegetation is beneficial at most locations, except in braided channel where vegetation encroachment is a concern.
- It is cheaper to purchase land now than in the future

- And again, recover floodplain and do not reduce floodplain to address most bank stabilization woes.

2.3 Risk Assessment

The design life of the structure is defined by the level of acceptable risk associated with the project. A longer project design life is required where there are higher consequences to society from a structure failure. A bank stabilization feature should be designed to a lower percent chance of failure if it threatens lives, threatens the survival of a species, if the feature contributes to maintaining critical access (only access road to a dam or hospital), or threatens a main water supply.

Once the level of acceptable risk is understood for the project, the design life of river projects are commonly associated with a peak flow interval unlike non-riverine structures that are designed to survive for a minimum number of years. This can confuse both the designer and the public when planning and evaluating a river structure. A river structure harmed in the year following construction has only “failed” if the flow event that destroyed the structure is less than the flow event specified by the design. If the river design must both resist a flow event and operate successfully for a specified number of years, hydrologic probability needs to be considered further. Project designers and all stakeholders should understand the ambiguity of the term ‘design life’ with respect to river hydrology and reach agreement on the level of investment for return. In addition, in some site specific cases, intermediate floods can be more damaging to bank protection than larger flows. For example, backwater conditions at a 100-yr flood may reduce velocities at a particular location and create less scour than a smaller flow would. Therefore, a check of hydraulic conditions up to and including the design flow may be necessary to determine bank protection performance.

Based on a 20-yr program of bridge scour investigations, the Federal Highway administration (FHWA) is recommending a design flow for a bridge be larger than the life of a structure (FHWA, 2012). A 50-yr flow return interval is not sufficient for a structure with a design life of 50 years. A 50-yr flow event would have a 63.4% chance of exceedance during the design life of the structure, but there is only a 39.5% chance that a high flow will exceed a 100-year storm event during the 50-year period following construction of the bridge (FHWA, 2012). In addition, the consequences from the next level of high flows such as a 200-yr or 500-yr flow event should be considered to verify the correct level of risk has been accepted. Similarly if Reclamation expects a bank stabilization project to have a 60.5% chance of surviving 50 years beyond construction, the river project should be designed to a 100-yr flow event, not a 50-year flow event.

The acceptable level of risk and cost should be considered for each structure, and compared to the consequences from site specific factors. When low risk to society or the environment is required for a project, alternative bank stabilization methods of backing the project out of the flood plain or expanding the flood plain area can be the most feasible project approaches. On all river projects, the Reclamation designers and the stake holders should understand the implications of the term 'design life' and have agreed on an acceptable level of risk. Beyond this point, the involved parties should also understand that a risk analysis only reduces risk but cannot remove it. This discussion is essential since even the best designed bank stabilization project can be unraveled in the first year of operation if a low-probability storm or snow-melt event occurs.

A second use of risk assessment should be considered and addressed by the designer. Incorporating plant materials into the bank design can increase the risk of project failure in the first few years following construction, if the use of these materials is not properly understood. The designer should account for the lower-level of erosive resistance in younger plants, through a two-stage design that includes a plan for temporary erosion protection following construction. These types of consideration are described in Chapter 9, Design of Vegetated Banks.

A third use of risk assessment is incorporated by the construction engineer to consider the vulnerability of the site during all phases of construction. It is usually not possible to coffer off the toe of bank stabilization for design high flow events. A construction engineer is required to assess the needs and risks in the design of the construction approach. He has to anticipate and match the appropriate seasonal flows with the construction techniques employed and the steps of construction.

2.4 Hydrologic Data for Site Analysis

In addition to development of the risk assessment and design criteria, understanding the current and proposed hydrologic regime is integral to any river design. The width, depth, slope, shape and form of the channel are, in part, determined by the magnitude and duration of peak flows and various lower flows. The hydrologic regime at a project downstream of a dam will be impacted by dam operating conditions in addition to climate conditions. Water release magnitudes and durations influence the channel morphology. The hydrologic regime at the site impacts habitat and ecological needs, river geomorphology, selection of a bank stabilization method, success of riparian plantings, the structure design, and project construction including methods, staging, and scheduling. Knowledge of the range of flow conditions and flow timing from both water operations and uncontrolled tributaries are required and the hydrologic flow regime is a function of both the river location and the period of record. Listed below are indicators of the hydrologic flow regime that aid development of the river design.

Mean Annual Flow. Various flow conditions and channel dimensions are used to define the hydrologic regime. Discussed here are the mean annual flow, the annual mean low flow, and annual mean high flow. Calculate the mean annual flow for the applicable period of record using the nearest gage data, adjusted for tributary inflow and flow diversions between the project site and gage. The mean annual flow is the numerical average of the mean daily flow.

Annual Mean Low Flow. Also determine the low-flow periods of discharge. For many systems in the Western United States this is a large part of the year, excluding months when spring runoff or summer thunderstorms typically occur. In the Western Cascades, the winter months when high flows typically occur would be excluded from the computation of the mean daily flow hydrograph. The annual low flow is the average of the mean daily flows for the low or base flow period of the hydrograph.

Annual Mean High Flow. Annual mean high flow is the average of the annual maximum mean daily flow (one flow value per year).

Period of Record. The mean annual flow, annual mean low flow and annual mean high flow are computed over a period of years to account for annual variability. Using the longest period of record possible increases the statistical validity of the flow estimates. When there have been major hydrologic changes such as the construction of dams and/or water withdrawals, it is best to use the period of record after these events. Ideally, the period of record will include both drought periods and periods of high flows.

Hydrographs. Two additional tools for representing the hydrologic regime are a cumulative flow hydrograph and a hydrograph of daily flows. Both will provide additional information about the hydrologic regime. Although bulky and site specific, a hydrograph of daily flows can provide detailed information on seasonal timing of flow events. This is especially useful for evaluating environmental factors similar to fisheries and avian habitat studies, or vegetation establishment and growth studies.

Discharge Specified by Return-Intervals. Probability is used to match a discharge value to the return period or frequency for that flow. For example a 50-year flow (Q_{50}) is assumed to occur an average of once every 50 years. A suite of return-interval discharge values can be used in the development of a river design, with selection depending on the element of the design. Design criteria and risk analysis described in the previous sections may specify a 50-year design life for the structure and a 100-year return interval for the scour analysis of the structure. Channel lining may be designed for a 2-year flow event and a riprap toe at the same location could be designed for a 25-year event depending upon rock availability, constructability and economic project life. A 2-year return period peak flow (Q_2) is sometimes used as an approximation of the bankfull flow or referred to as the mean annual peak flow. For this guide, Q_2 is estimated by using peak flow regression techniques such as Log Pearson type III, and the mean annual peak flow is calculated as given above. Q_2 would be best to use as the mean flow when there are adjacent high value lands or infrastructure.

Bankfull Flow. It is possible that the highest near bank velocities occur at intermediate flows, not the largest flow. Bankfull flow is the discharge required to fill a channel to bank height before flows spill onto the floodplain. Bank height for undisturbed conditions is a natural balance between soil and floodplain conditions, and the hydrologic regime. It is a geomorphic concept that aids interpretations of river conditions and is often used in river designs and habitat improvement methods.

Method 1. Bankfull flow can be determined from a cross section, or preferably, multiple channel cross sections assuming normal flow depth in the sections. A better estimate could be developed from a field survey, but for a less accurate value, the cross sections for the study reach could also be sliced from a good terrain surface. Care is required when selecting the cross section location for both techniques due to the high variability of conditions along the channel and to the common occurrence of human and other impacts on flow and physical conditions since the original formation of the channel and main bank heights. There is more certainty in determining bankfull flow from cross section measurements if the following circumstances are met (McCullah and Gray, 2005):

- the stream bed and banks are alluvial;
- there have been no recent extreme floods, droughts, earthquakes, forest fires or other catastrophic events;
- the watershed is free from anthropomorphic impacts including channelization, floodplain alterations (commonly constrictions), levees, or land use changes (grazing, mining, road and dam building);
- the channel is not incised and is relatively consistent for the reach represented by the study section.

There are few instances where the study reach meets these criteria and field indicators of bankfull flow, such as permanent vegetation or terraces can be misleading (McCullah and Gray, 2005) when the channel is degrading, aggrading or rapidly migrating. Despite the challenges in determining a best value for the study reach from the geometry of the channel, bankfull flow is an important and useful indicator of hydrologic regime for bank stabilization designs.

Method 2. Bankfull flow is also frequently estimated as a 2- to 5-yr flow event. This value can range up to a 10-yr flow event for ephemeral streams in the arid southwest (Pemberton and Lara, 1971). The estimated value and cross section value should be compared when a cross section value can be measured.

Method 3. A third method, also based on cross sections, is to assess river bends in a hydraulic model such as HEC-RAS or SRH-2D (Lai, 2008). The water surface elevations at the site for mean annual flow, mean annual low flow, and mean annual high flow can be used in the design and dimensions of bank stabilization structures including transverse flow structures.

Channel Top Width. This is usually the width of the approach channel at either the mean annual peak flow, or Q_2 determined using a 1-D hydraulic model such as HEC-RAS. For applications with less precision required, the top width may be the distance between detectible bank lines in an aerial photo.

Mean flow depth. The mean flow depth is also described as the hydraulic depth and computed by dividing the flow cross sectional area by the channel top width.

Green Line. When vegetation is used in the project, determine the discharge below which vegetation does not grow on the banks. Most plants are not tolerant of consistently wet roots and herbaceous and willow plants will often establish above the consistently wet elevation. Ignoring cattail and other wetlands plants, visual identify the green line along the river banks and survey points along this elevation. Even in concrete lined channels, a green line can sometimes be identified from plants established in the cracks between concrete slabs. The green line elevations can vary somewhat between years, in response to wet and dry climate cycles. The slope of the green line should approximate the slope of the longitudinal water surface elevation. Find the best match between the elevation points of the green line survey and the water surface in a one-dimensional hydraulic model, by varying the discharge in the model. A return interval can also be estimated for this discharge from the site hydrology for a better understanding of the hydraulic and riparian dynamics. The return interval, discharge, and green line elevation points provide quantitative values for site planning, structure design and planting design.

Active Channel Width. The width of the active channel is the width between the permanent vegetation lining each bank, i.e. the horizontal distance between the green line on each bank. Depending on site conditions, active channel width can be less than or more than the channel width measured between banks. This parameter can sometimes be easier than top width to detect from aerial photos. The width of the active channel can be reviewed to determine temporal and spatial changes to the channel, and the rate of change. For cases where channel width indicated by vegetation has a high rate of change, and/or is significantly different than the width calculated using either the mean annual peak flow or $Q_{2,}$ the choice of width to use in a design should be based upon the application, site information, risk analysis, design criteria and other pertinent factors.

Channel Maintenance Flows. Sufficiently high flows at frequent intervals (annually or every 2 to 3 years) define the river shape (cross section) and river form (plan view). These periodic flow events move stored sediment, both sustain and control vegetation, and help to maintain channel width and channel capacity.

2.5 Other Site Data

The geomorphic assessment and sediment transport, like characterization of the hydrologic regime, are fundamental aspects of a successful river design and a sustainable project. Integration of geomorphic site conditions and sediment transport conditions with design of bank stabilization measures is introduced in the next chapter. In addition to the hydrologic regime, geomorphic conditions, and sediment transport conditions, other elements of the site analysis and investigation include acquisition of terrain data, groundwater and geology information, soils observations and mapping, vegetation observations, understanding of the eco-system inhabitants and endangered species, and a general climate assessment.

Good terrain data is essential for design and accurate 1 ft contours are often necessary to represent the surface to develop the design and prepare construction drawings. Inaccurate surfaces result in costly changes during construction. Riparian groundwater conditions at the site are mostly impacted by the river but point source flows like springs, flow loss diversions, bogs, significant groundwater extraction from a nearby location, or other unique conditions are items to identify. Detailed information on geologic features of rock in the bed both at the site and downstream of the site are beneficial to the development of the design. Unusual soil types or coverage can influence sediment transport, vegetation growth, and flow loss, and subsequently are significant to the sustainability of the project. River sediment analysis is often included in the geomorphic assessment, but observations photos and samples of channel bed material and bed features during a field site inspection are valuable. Channel bed material samples and bedform observations help establish river form and morphology, sediment transport conditions, and erosive conditions. Identifying the types and location of riparian vegetation currently on site, including surveying the green line in the field, can provide information on the hydrologic regime, hydraulics, groundwater, soils and geomorphic conditions.

2.6 Permitting

The permitting system is used to regulate activities that take place in or along streams. These programs generally intend to prevent the creation of flood hazards, protect against damages to aquatic life, and protect the rights of neighboring landowners. Regulated activities may include streambank stabilization, road improvements that encroach on streams, bridge construction or repair, dam removals and utility crossings under streambeds. Permits may be required from federal, state and local entities. Commonly, one department or group within an agency or company will handle the permitting for all projects, since familiarity with local and state laws and requirements can reduce time invested in the permitting process. Timing for beginning the permitting process is generally “as soon as possible” and the schedule can vary with each region. However, permitting agencies may want to review a well-defined project and refuse anything less than a 30% design. Contacting the permitting agencies for general information and questions on common permitting concerns prior to submittal, may help reduce required changes and help streamline the process.

Most common Federal permits are briefly described in Table 2–1.

Table 2–1. Federal Permits

Sacramento River Watershed Program, http://www.sacriver.org/aboutwatershed/permitguide/permittype, accessed 7/25/2014

Permit	Permit Name	Description
U.S. Army Corps of Engineers (USACE) Clean Water Act	Section 404	Issued for the placement of dredged or fill materials into waters of the United States, including wetlands, below the Ordinary High Water Mark (OHWM).
USACE Rivers and Harbors Act Section 10 Authorization	Section 10 for structures in a navigable waterbody	Issued for the placement of structures, or work (including discharge of dredged or fill materials and excavation) in, above, or below navigable waters that could obstruct navigability in such waters.
U.S. Fish and Wildlife Service (USFWS)/National Marine Fisheries Service (NFMS) Endangered Species Act Take Authorization	Section 7 “Biological Opinion” or Section 10[a] “take permit”	Issued for adverse effects to federally listed plant and wildlife species.
RWQCB National Pollutant Discharge Elimination System (NPDES) permit	NPDES	Issued for the discharge of waste and pollutants into surface waters. Issued to maintain the quality of surface waters and ensure that project actions do not reduce the quality of the water.

3 The Role of Geomorphology in River Projects

Fluvial geomorphology is the study of land and river forms which are created and evolve by the action of flowing water and sediment. Rivers are spatially and temporarily dynamic, and vertical and lateral position of the river can change over time in response to changes in hydrology and sediment supply, and in response to human influences. River position can also change in the absence of changing conditions, simply due to natural river meandering. By definition, even a river described as stable is not in a static condition, but instead is described as a state of dynamic equilibrium. The river can adjust laterally through bank erosion and bar building, and can adjust vertically around a central position in response to seasonal or temporary climatic variations. Although there are cases where there is no sediment balance under natural conditions, rivers in dynamic equilibrium generally have an average supply of sediment that matches the average sediment transport capacity. This definition of dynamic equilibrium is assumed throughout this guide for stable rivers.

It is easier to construct a sustainable (low maintenance) project in a stable river reach than in an unstable reach. Bank stability projects are sometimes constructed at stable locations where there is no tolerance for lateral bank adjustments under dynamic equilibrium; or projects are constructed at locations where there is instability causing excessive bank erosion. Due to the natural condition of dynamic equilibrium, rigid lateral constraints on river banks can make a goal for a stable but sustainable river bank very difficult to obtain. Hence there is a conflict in seeking to ‘stabilize’ the bank and expecting the river to be ‘stable’ and unchanged. A project may involve only a small portion of the river system but has the potential to trigger morphological response in other parts of the system, both upstream and downstream (Watson et al., 2005). In addition, the project site can be impacted by changes instigated at other river locations. Less traditional methods to bank stabilization can be more feasible and improve the sediment continuity, helping to improve sustainability and reduce the need for future stabilization requirements (Thorne et al., 1997). A geomorphic process-based approach is dependent on an understanding of present and future river processes in the selection of methods to stabilize the bank, preserve a sustainable river, and maintain or enhance ecological diversity.

The potential river response to a project, and the projects impact on other locations should be understood and evaluated during the development of the project to accomplish NEPA and ESA compliance, but also should be understood for an accurate evaluation of project benefits, effects and risks, and project sustainability. As part of the bank stabilization effort, river processes currently acting at the site should be understood, and potential future interactions between river processes and the constructed bank should be considered during development of the design.

3.1 Geomorphic Assessment

A geomorphic assessment is an early task in the design. The assessment is an integration of information from geology, climate, topography, soils, channel sediment, vegetation, channel morphology, the chronology of disturbances both natural and anthropogenic, aerial photographs, and field reconnaissance. If the designer does not have a strong background in geomorphology, the assumption in this guide is that a geomorphologist or river engineer will be a member of the design team and will carry out the geomorphic investigation. As described by Randle et al., (2006):

> Geomorphology provides the context to help understand the river channel planform, historical channel paths and rates of migration, interactions with flood plains and terraces, and sediment sources and sinks. The analysis helps to identify upstream and downstream influences, geologic controls along the study reach, and human actions that have affected the natural processes. The analysis assists in identifying the cause(s) and magnitude of the disturbance.

Processes and channel changes can be investigated at the local, reach, and watershed scale (Thorne, 2002). The aim of a geomorphic assessment is to provide baseline information necessary to characterize process-form interactions in the river, to identify river control points and reach dynamics, and to support division of the channel into distinct sub-reaches that have a common morphology. This includes an assessment of channel stability on a reach-by-reach basis and an evaluation of potential future channel changes if no action is taken.

The assessment should match its scope and content with project goals, authority, stream and watershed characteristics, and available resources. The geomorphic assessment alone is not sufficient for project sustainability but rather is a valuable and necessary component of an integrated design process (Watson et al., 2005) that includes evaluation of potential river response using sediment transport and hydraulics models such as SRH-1D (Huang and Greimann, 2013) and SRH-2D (Lai, 2008). Many bank stabilization projects have failed, not as a result of deficient hydraulic and structural design, but because of a failure to consider the significance of geomorphology to the project. The geomorphic assessment provides a system context and framework within which the designer can:

- Select applicable hydrodynamic and sediment transport equations based on the channel conditions
- Match, if possible, stable stream dimensions and plan form to project goals

- Use computer models to incorporate geologic and human-induced controls, to predict channel response to proposed project features
- Integrate environmental features into the engineering aspects of the project
- Anticipate maintenance requirements and optimize design for sustainability
- Develop the scope of post-project monitoring and adaptive management.

Geomorphology coupled with sediment transport and hydraulic models opens the door to cost-effective, sustainable bank stabilization solutions, including alternatives to bank stabilization.

3.2 Procedural Steps

The outline below identifies general steps in a geomorphic analysis:

1. Assess the physical characteristics of the land and stream, such as topography, soils, streambed and banks and channel geometry.
2. Evaluate the current sediment sources of the stream.
3. Identify existing and future erosion and deposition, including the creation of a sediment budget at the watershed, reach, or site scale. Identify how vegetation and runoff affects sediment supply.
4. Evaluate channel stability in terms of aggradation and erosion.
5. Evaluate habitat. Evaluate man-made influences including levees, roads, bridges, channelization and bank stabilization, culverts, etc.
6. Determine discharge, including flow frequency, depths, duration, and floodplain inundation. This would include low flows, mean annual flow peak, flood flows, groundwater recharge and discharge, flood events (such as rain on snow), and how discharge is affected by vegetation, floodplain connectivity, etc.
7. Assemble existing geomorphic data and information on the site including aerial photos, sediment samples, and reported information from reach or basin-wide reports.
8. Determine the current channel form and sediment transport features.
9. Analyze and interpret historical and current geomorphological information.
10. Describe the stream type and assess reach stability.
11. Estimate future potential channel response to the proposed project.

For more information on geomorphic principles, users of this guideline are referred to existing publications such as Knighton (1998), Schumm (2005), Kondolf and Piegay (2003), and Randle et al. (2006), among many others.

3.3 Geomorphic Considerations to Promote a Stable Channel

Floodplain establishment and connectivity should be incorporated to bring sediment transport in balance with supply. Otherwise, there will be increased potential for channel degradation and lowering of the water table. Bed stability is an essential consideration in any bank stabilization scheme. When there is system-wide channel degradation; a more comprehensive treatment plan is necessary (Biedenharn et al., 1997). If the channel is incising, the toe of any bank protection could be undermined and fail. Countermeasures include additional toe protection, grade stabilization (Watson et al., 2005). Alternatively, measures to facilitate or encourage a new dynamic equilibrium condition (Shields et al., 1999) may be required.

In all cases, maximizing sustainability and reducing future requirements are a consideration. Sustainability is increased when the methods treat the cause of bank erosion rather than the symptoms. For example, this may involve increasing sediment supply, reducing sediment transport capacity with longitudinal bank lowering, removing or relocating lateral channel constraints, or bed stabilization. Changing the sediment supply and bed stabilization are two areas of channel design not included in this document.

4 Hydraulic Assessment of Energy, River Form, and Shear Forces

Equally important to the geomorphic assessment described in the previous chapter, is the hydraulic assessment which determines the level of erosive energy acting at the project site.

Vegetation is an ideal bank stabilization technique as it provides habitat, slows erosion and can continuously adjust to the naturally evolving changes in river form and alignment. However vegetation or natural materials like woody debris are not always sufficient for the project site if the site has been altered from a natural form or the channel or floodplain width has been reduced. Planting can still be successful at altered sites, if the stream is low energy, but tree roots, rock and engineered log jams may be required to resist erosion in rivers with a medium level of energy. Rivers with high energy are challenging and could require both erosion resistant materials and a change in channel form that reduces the energy level. Identifying the erosive energy at a site through a hydraulic assessment, before moving forward with the selection of materials and bank stabilization methods helps to increase project success and reduce future maintenance.

This section contains descriptions of flow energy and width/depth ratio, energy and river form, erosive forces, and sediment analysis. These are presented as considerations and as general information recognizing that not all rivers have these same characteristics. More detailed geomorphic evaluation and hydraulic analysis may show some deviation from these general concepts, due to the complexities of fluvial systems, geology, vegetation patterns, and human modification.

4.1 Energy and Width/Depth Ratio

A trapezoidal shape with a small width/depth ratio (narrow and deep) minimizes energy loss and is an efficient flow conveyance shape for an irrigation system. Agricultural areas typically do not have high gradients and a channel that minimizes energy loss is desirable. In contrast, trapezoidal shapes with a small width to depth ratio are not desirable for rivers or bank stabilization projects because the low friction loss translates to larger erosive forces acting on the bed and banks of the channel.

In nature, channels often evolve to a larger width/depth ratio than found in a constructed trapezoidal channel. Larger width/depth ratios can help balance the available energy in the system by increasing friction loss and losses from secondary flow currents and turbulence. When erosive energy is balanced by the resistance in the channel, the river tends to be stable. Shown in Figure 4–1 is the relationship of width/depth ratio to the weighted mean percentage of silt-clay in the channel boundary as presented by Schumm (1960, 1971). Wider and shallower channels are associated with a higher percentage of coarse sediment transport, while deeper channels with steeper banks can develop in soils containing higher clay content. A channel with a greater width is more efficient at transporting bedload material downstream through saltation, i.e. bouncing particles along the bed.

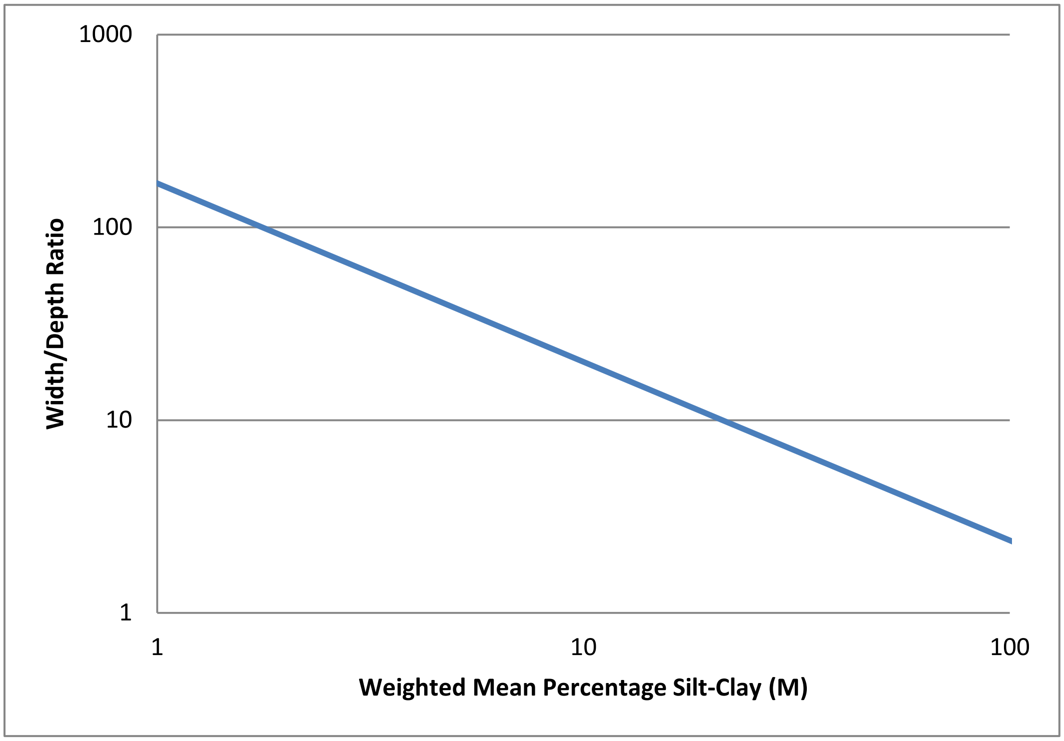


Figure 4–1. Relationship of width/depth ratio to weighted mean percentage of silt-clay by Schumm (1960).

The width/depth ratio of a river is also influenced by the type and density of vegetation along the banks. Charlton et al. (1978) found that in comparison to a general width-discharge relation, grassed banks were 30 percent wider, and tree lined banks were 30 percent more narrow. Dense vegetation on the banks produces a more narrow and deep channel.

4.2 Energy and Complex Channels

In a natural system the channel may evolve so an energy balance can be maintained at different flow levels. Shown in Figure 4–2 is a typical natural channel with a complex shape to balance energy at three levels. There is a low flow channel within the main channel, and the main channel is located within a large floodplain. Flow energy is balanced with resistance of the low flow channel during daily flows. At peak flows up to a 2-year to 5-year return interval (bankfull flow), energy is balanced by the form of the main channel. At higher flows, energy is balanced by flow spilling onto the flood plain for downriver conveyance, in addition to flow in the channel. Flow depth on the flood plain is relatively shallow even for rare recurrence intervals of flow (i.e. 50-yr and 100-yr events) and erosive flow energy can be balanced during high flows by the resistivity of vegetation in the overbank area. These complex river cross sections develop naturally to match the flow and sediment regime of a river.

Side channels are an additional feature of a complex channel. During high flows, flow may spill or avulse to a second channel at a low point in the bank line. This is an intermediary step before uniform conveyance across the floodplain, but the side channel also serves to reduce erosive energy in the main channel by reducing main channel flow and flow depth. The shape of the side channel can evolve to a stable form that balances flow and slope energy through channel resistivity, but may also form or be destroyed frequently to match the flow regime.

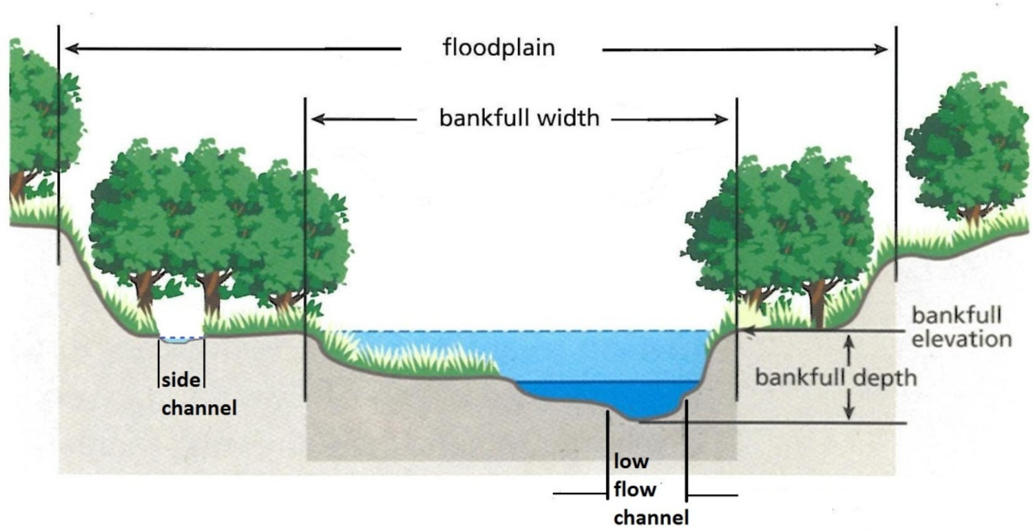


Figure 4–2. Channel cross section with low flow channel, bankfull channel, floodplain, and high flow side channel (NRCS 2007a).

4.3 Energy and River Form

River form including the plan form of a river as viewed from above, the cross-section shape, and the longitudinal slope, provides clues about the energy level of a river. As presented in the previous chapter, river form is constantly adjusting to changes in flow and sediment regime. The energy of the flow is balanced by the energy reduction imposed by the river form and sediment availability and therefore, the form of the river reflects current and previous energy levels.

River slope and flow rate define the level of stream power available with steeper slopes and larger flow rates imparting more energy (Lane, 1957; Leopold and Wolman, 1957). The potential energy from elevation, i.e. a steep slope, is converted to motion or dissipated, and can also do the work of transporting sediment (Roberson and Crowe, 1993; Yang, 1996). A balance between incoming energy from the stream gradient (potential energy from slope) and kinetic energy of the incoming flow is balanced against the outgoing energy including kinetic energy and energy losses. Energy losses can occur as shear forces along the channel boundaries, turbulence, and the transport of sediment. Different methods of energy conversion are presented in Table 4-1 in a relative ranking demonstrating the methods that are most effective at energy conversion.

Table 4–1. Energy Reduction and River Form

Relative Ranking of Energy Loss*	Method	Common River Form and Requirements
High	Large hydraulic Jump	Concrete lined channels and rock-wall canyons
High	Large vertical drops in the channel bed	Cascades, Step Pools, and lower energy meander pool-riffle systems with appropriate geology
Medium	Small water surface drops over cobbles and boulders	Cascades, Step Pools, Pool-Riffle Systems
Medium	Small hydraulic jumps	Cascades, Step Pools, Pool-Riffle Systems
Medium- dependent on supply	Sediment Transport-Boulders, cobbles, gravels, sands	Cascades, Step Pools, Braided, Multichannel with bars, Meander channels with bars
Medium- for braided channels	Friction losses from bed and banks	Braided and complex channel forms, dependent on a larger width to depth ratio
Medium to Low	Turbulence from secondary circulation patterns, mainly bends	Meander Channels, pool-riffle, requires some depth to develop flow pattern
Medium to Low	Channel contractions or expansions	All forms, dependent on flow and local conditions
Medium to Low	Sharp channel bends	All forms, dependent on flow and bend
Low- for deep single channels	Friction losses from bed and banks	Single channels, dependent on a smaller width to depth ratio
Low	Sediment Transport-suspended fine sediments	All forms, dependent on supply
Low	Changes in flow velocity	All forms, floodplains, dependent on local conditions, terrain, geology

* Energy “loss” is the dissipation of kinetic and potential energy.

Plan form and cross section of a river are a function of available energy, local conditions (valley terrain and soil), and flow rate; and the final form reflects the energy balance that establishes at each location. Size and volume of sediment transported by the river is dependent on river flows, overland material runoff, and the sediment in the bed and banks of the channel. Higher energy is required to move coarser particles and erode particles from the bank and bed. Likewise river form is dependent on the materials in the valley. Easily eroded materials in the flood plain contribute to the formation of anastomosed or braided rivers with wide floodplains, or complex meander channels with bars and multi- channels.

A study of river form provides information on the erosive energy at the project site. Wide, shallow channels develop when there is an abundant supply of sediment and medium to high levels of slope/potential energy. This form can move large volumes of bedload sediment in a straight braided river form to balance potential energy. While there are exceptions, many channels with less available sediment may balance potential energy with smaller width to depth ratios. A deep channel and small width to depth ratio can create energy losses in addition to shear force losses along the boundary, through the larger rollers of erosive secondary flow patterns. Meandering rivers have less excess energy. If there is no significant sediment supply, energy is balanced by the channel evolving to a greater stream length and an increase in the number of meander bends. A longer channel increases shear force losses and more frequent or tighter meander bends increases energy losses from the turbulence of secondary flows. Steep streams, including mountain streams, are balancing a large value of potential energy. These systems may transport large bedload particles (cobbles and boulders) and or large volumes of smaller material from cut banks and steep slopes during high flows, and also can convert a large amount of excess energy to turbulence and heat loss in large drops and drop pools, or simply in a number of small step drops similar to the height of the bed particles.

A high flow or series of high flows can produce the changes in river form that balance erosive energy and channel resistance including meander bend erosion, channel lengthening or drop developments. Continuous daily flows can also produce less extensive changes within the main channel. Changes in channel form that developed during low flows to balance energy, are often erased during higher flows to balance bigger differentials between potential energy and energy losses. Erosive energy during bankfull flow conditions, however, can sometimes be more erosive than overbank peak flood conditions so a range of flow conditions should be considered in an evaluation of energy and bank erosion.

Concepts on the links between river and floodplain characteristics, and the resulting balance between energy and river form are related in a myriad of channel forms initially presented as the channel continuum (Lane, 1957; Leopold and Wolman, 1957). Due to the complexities of river response, illustrations of these concepts often focus on a few main factors. Lane (1957) and Leopold and Wolman (1957) selected slope and discharge as major defining factors of a straight, meandering and braided channel form. Brice (1975) looked at classifications of river form by degrees and character of sinuosity, braiding and anabranching. Schumm (1960, 1977) illustrated the relation between sediment load, slope and river form characteristics including stability, width to depth ratios, slope, bar forms and sinuosity. Figures developed by Chang (1985) and Van den

Berg (1995), provide more detailed information on the relation between grain size, slope and discharge to help define the transition between a meandering or braided river form. Nanson and Knighton (1996) describe the causes of river anabranching, and an illustration by Montgomery and Buffington (1997) relate slope to steeper stream forms including cascades and step-pools. These are only a fraction of the papers relating river form and factors. A design should not be based on a streamform categorization without consideration of acting forces, but this material does help define current conditions, ecological habitat, and help predict the potential of changes to the project site with different bank stabilization methods. Four categories of energy and river form are presented in successive sections to help summarize the dependence of river form on energy and the valley and channel conditions.

4.3.1 River Form with Low Energy Systems

Low energy rivers do not have much stream power, i.e. the product of slope and discharge is small. Incised channels can often be grouped within the low energy category when the channel has a shortage of sediment supply and the banks are strengthened by vegetation coverage, geologic features or bank stabilization measures. Energy can often be balanced by shear forces acting along the channel bed and banks with little significant bank erosion, sediment transport, or turbulent secondary flow patterns. Erosion of the bed continues until a balance is achieved by reducing the bed slope i.e. reducing the potential energy and incoming velocity of the system to match the sum of the energy losses and outgoing velocity. Although the system had higher energy initially, the reduction in bed slope evolves the incised channel to a low energy system.

River alignment and form is not distinct in low energy systems with limited sediment. The channel plan form is defined by the terrain, the geology and by anthropogenic features since there is insufficient energy to erode banks, alter the alignment or change the slope through bed erosion and deposition. River alignment may be straight or meandering, but the current alignment and cross section form could have been defined by historical large flows that no longer occur, or by development that forces the current channel form. These channels may have limited fish habitat with no secondary flows to form deep pools at bends, no scour holes at woody debris, and cannot develop pronounced pool-riffle systems. A flat and incised channel provides minimal environmental benefit and is a common condition in over-stabilized rivers with long or frequent lengths of riprap revetment and/or levees. At these locations there may be limited floodplain connection and limited meander bend or bank adjustments. Low energy systems may also have been constructed as part of water delivery or flood control projects with an excess number or height of drop structures, which have removed most potential energy from the system.

A flat and incised channel can also develop after invasive, erosion-resistant plants establish monocultures along river banks. The initial constriction caused by the confining banks of vegetation creates channel incision and traps sediment building natural levees along the banks.

Streams with reduced flows downstream of a dam can be another example of a low energy system. Channel that formed prior to the dam may have a coarse armor layer and well vegetated or stabilized banks that can easily resist the erosion of post-dam, lower-energy flows. In this case the bed may not be flat. Bed forms, bars and pools that develop under previous high flow conditions, remain since there is no sediment supply from upstream or sufficiently high erosive conditions to alter the previous channel form.

Springs and locations with minimal variation in the hydrologic cycle can be low energy systems. Another characteristic may be low gradient terrain with a wide unrestricted floodplain. Often rivers near sea level fit this category. In wide floodplains, flows can divide into small individual pathways that further reduce channel flow energy. Deltas and depositional areas are examples of low energy systems with sediment supply and transport into a reach at high flows, but insufficient energy to move sediment or erode the bed and banks out of the reach until a depositional buildup restores a steeper gradient. Unstable, low energy deltas and depositional areas should not be confused with medium to high-energy braided river systems that are commonly stable.

4.3.2 River Form with Low to Medium Energy Systems

An anastomosed river form has multiple flow paths divided by vegetated bars or islands and may be a low or medium energy system. An anastomosed river can evolve from a previously braided reach of river with high energy flow events and a plentiful sediment supply, but these conditions are rare in the present state. The main mechanism of energy balance for anastomosed rivers is split flows. With split flow conditions, bed and bank resistance can be sufficient to balance the potential energy of the system with also possibly some secondary flow patterns. Multiple channels continue to convey flows unless there is a sufficient supply of sediment to plug side channels and concentrate flows to one or two main channels. There may also be limited floodplain connection if the multiple channels persist.

4.3.3 Medium Energy and River Form

The width/depth ratio of the channel, number of meanders and radius of meanders, and the stability of the meander channel is all a reflection of the energy balance for the reach. Channel form is more narrow and deep (a small width to depth ratio) when there is not an excess sediment supply and there is minimal energy dissipation from the process of sediment transport. Instead, the channel can balance potential energy with secondary flows in addition to shear. A deeper channel can develop larger helicoidal flow patterns in bends in contrast to shallow channels with an armored bend that limit the size of the roller. Shallow channels subsequently have lower erosive force acting on the bend and less energy loss from this process. Meandering rivers with active cut banks (meanders still developing) are indicators of excess energy during higher flows. Erosive energy can result from the potential energy of the stream gradient (slope) or the kinetic energy from a large discharge. Changes to the bends can lengthen the river and serve to increase channel resistance, and the erosion and transport of sediment also helps to balance excess potential energy. Continued expansion can shift the channel towards a more stable configuration or be a repeating occurrence of meander migration with this process serving to balance energy during higher flows.

At the highest flows, water should overtop the banks of the channel. Overtopping flow limits the erosive force placed on the channel banks. The potential energy of flows moving across a wide floodplain can often be dissipated by the resistance of vegetation on the floodplain. Shallow flows spilling across the floodplain are less erosive than deep flows retained within a channel. When the channel is restricted by bank stabilization features that prevent channel adjustment or channel overtopping, more erosive conditions develop possibly shifting the channel to an incising condition as described in the previous section.

4.3.4 Medium to High Energy and River Form

Energy can come from steep slopes, large flows or some combination of these streampower factors. A river will braid when there is medium to high energy, an abundant sediment supply matching the energy of the system (sand for medium levels of energy and gravels and cobbles for higher levels), and some width for floodplain development. Braided rivers have a shallow wide channel with large width to depth ratios and more than two side by side flow paths within the main channel cross section. Applying his extremal hypothesis, Chang (1979b) showed analytically the number of braids in a braided channel will increase with an increase in stream power. The slope of a braided channel, established by the deposition of excess sediment, is consistent and sufficiently steep to move the available sediment load along the bed of the channel. The transport of sediment dissipates energy in addition to the bed shear forces. The wide, shallow cross section with multiple flow paths is often a reflection of a higher but balanced energy conditions, and a stable river form. In contrast, deltas or depositional areas are unstable systems with low energy until the buildup of sediment deposition steepens the delta and increases the potential energy. The combination of flow rate and bed slope creates the higher energy of the system, but is balanced by energy-dissipation from sediment movement, bed resistance, and some turbulence. Helicoidal secondary flow patterns are less prevalent.

An anabranched or multi-channel meander river with some side channels is often a medium-energy system. Mechanisms of energy balance include the division of flow between multiple channels and shear forces on the bed and banks. Other mechanisms for energy reduction may be sediment transport as indicated by the presence of bars and bedforms, or deeper bend pools with secondary flows that generate some energy losses through turbulence. Blocking flow to a side channel concentrates flow in the main channel and may also reduce floodplain access.

Under this action, some increase in erosive activity can be expected in the now single channel reach, and may occur as increased bank erosion or some level of bed erosion (channel incision).

4.3.5 High Energy Systems

In addition to braided gravel-bed rivers, mountain streams with cascades or steps of cobbles and boulders, woody debris or rock outcrops are another form of a high-energy river. In these cases, high energy is generated from a steep slope but the flow rate is not large often because the reach is high in the watershed. Energy dissipation or resistance is primarily through turbulence at the bed and banks, or from rollers in pools at drops. Some material is transported downstream and also contributes to energy dissipation. Steps can form in uniform spacing from interlocking cobbles and boulders, and drops with pools develop at interlocked boulders and rock outcrops. With the exception of pools, large helicoidal flow cannot develop and stream banks are not susceptible to the development of secondary flow patterns that erode banks. Impact, turbulence and secondary flow in the pools at the base of the drops, dissipate the energy of an elevation drop. Bed resistance acts between drops but excessive energy builds until expended in small hydraulic jumps or in the next drop. During shallow flows, energy is also balanced in rocky stream beds through form resistance as flows move by and over rock cobbles and boulders.

High energy systems often have resistant and confining banks that prevent the channel from evolving to a medium-level system. Gravels, cobbles and vegetated banks may be sufficient to confine small-discharge streams or streams with a large bedload, while canyon walls can confine larger streams.

4.3.6 Summary of Energy and River Form

Each reach of river is stable in the current condition when the potential and kinetic energy entering the reach is balanced by conversion of energy within the reach. Potential and kinetic energy entering the reach can be converted to heat loss within the reach or converted to kinetic movement of flow and sediment out of the reach through the means listed in Table 4-1. Channel form can also evolve due to the ongoing energy balance. Using recognition of both channel form and natural means of energy reduction and balance, an engineer can identify the level of erosive concerns at a site prior to computations. This knowledge can also aid the engineer in the selection of suitable bank stabilization approaches. For example it may be possible to stabilize a bank with vegetation if it is identified as a low energy site, while a site identified as being a medium or higher energy level may require means of shifting or constructing the channel to a larger width to depth ratio, in addition to improving vegetation on the banks. Altering the river form to balance the excess energy and reduce the erosive pressure on the banks preserves more river function and habitat, in comparison to more traditional bank hardening techniques. In addition to increasing the width/depth ratio of the channel to a stable condition, examples of energy reducing bank stabilization measures include adding a secondary channel or side channel to split flow and lower erosive energy; more overbank flow i.e. reconnection with the floodplain; or, in special circumstances, sediment augmentation to instigate a change in width/depth ratio could be used. Conversely, where the goal is to maintain a medium to high level of energy to preserve braided river habitat, it may be necessary to prevent side channels from diverting flow and energy away from the braided main channel in a wide valley (Fotherby, 2008). Consideration of the channel form is a qualitative means of assessing energy in the channel, and subsequently aids the selection of appropriate bank stabilization methods for the conditions at the study site.

4.4 Computing Erosive Force and Assessing Material and Methods Suitability

After assessing energy levels through a study of river form, the next step in an energy driven design is the computation of erosive forces acting at the site. Values from the computation are compared against the resistance values for a range of materials and bank stabilization approaches. The analysis of a reach with relatively simple conditions can be readily carried out using one-dimensional flow models. Reclamation has a flow or a flow and sediment model, SRH-1D (Huang and Greiman, 2012), and also a flow, sediment and vegetation growth model, SRH-1DV (Fotherby, 2012), for better representation of habitat and vegetation factors. The HEC-RAS (USACE, 2010) model can also be used for flow and sediment computations. At more complex sites where the flow pattern includes cross-channel flows or flow diversions, a two-dimensional model like Reclamation's SRH-2D (Lai, 2008) may be more appropriate for the computations of erosive force.

Indicators of erosive forces can be average velocity, unit discharge as volume divided by flow width, shear force as force per unit area, energy as amount needed to raise one unit of water one degree of temperature, or work, the rate at which work is produced from energy. Average velocity is not an accurate indicator of the erosive forces in localized flow conditions but it is easy to acquire from 1D flow models and is useful as a general indicator of conditions. Shear force is also an easy indicator to acquire from a HEC-RAS or SRH-1D model. Shear force and/or velocity are used for selecting materials that will be able to resist the erosive flow forces.

Materials sufficient to resist erosive forces may range from grass to riprap and each material is associated with a different range of resistivity values. Resistances to flow for different materials that are placed on a river bank as a revetment are shown in Table 4-2. After computing velocity and shear values using a 1D or 2D flow model, or getting an estimate from a section analysis, compare these values to the resistance of common materials below. It is not unusual to find the erosive forces exceeding the resistance of the materials, or require excessively large riprap sizes that are difficult to obtain. This is an indication that an alternative method to bank stabilization, such as a change in channel form or a reconnection of the floodplain should be explored. If the riprap design calls for a rock size that is not obtainable, the design is neither feasible nor sustainable in addition to failing the third goal of environmental effectiveness.

Table 4–2. Permissible Shear and Velocity Resistance Values for Selected Lining Materials (Fischenich 2001)[1]

Boundary Category	Bank Material Type	Permissible Shear Stress (lb/sq ft)	Permissible Velocity (ft/sec)	Citation(s) ([2])
Soils	Fine colloidal sand	0.02 - 0.03	1.5	A
	Sandy loam (noncolloidal)	0.03 - 0.04	1.75	A
	Alluvial silt (noncolloidal)	0.045 - 0.05	2	A
	Silty loam (noncolloidal)	0.045 - 0.05	1.75 – 2.25	A
	Firm loam	0.075	2.5	A
	Fine gravels	0.075	2.5	A
	Stiff clay	0.26	3 – 4.5	A, F
	Alluvial silt (colloidal)	0.26	3.75	A
	Graded loam to cobbles	0.38	3.75	A
	Graded silts to cobbles	0.43	4	A
	Shales and hardpan	0.67	6	A
Gravel/Cobble	1-in.	0.33	2.5 – 5	A
	2-in.	0.67	3 – 6	A
	6-in.	2.0	4 – 7.5	A
	12-in.	4.0	5.5 – 12	A
Vegetation	Class A turf	3.7	6 – 8	E, N
	Class B turf	2.1	4 – 7	E, N
	Class C turf	1.0	3.5	E, N
	Long native grasses	1.2 – 1.7	4 – 6	G, H, L, N
	Short native and bunch grass	0.7 - 0.95	3 – 4	G, H, L, N
	Reed plantings	0.1-0.6	N/A	E, N
	Hardwood tree plantings	0.41-2.5	N/A	E, N
Temporary Degradable RECPs	Jute net	0.45	1 – 2.5	E, H, M
	Straw with net	1.5 – 1.65	1 – 3	E, H, M
	Coconut fiber with net	2.25	3 – 4	E, M
	Fiberglass roving	2.00	2.5 – 7	E, H, M
Non-Degradable RECPs	Unvegetated	3.00	5 – 7	E, G, M
	Partially established	4.0-6.0	7.5 – 15	E, G, M
	Fully vegetated	8.00	8 – 21	F, L, M

Boundary Category	Bank Material Type	Permissible Shear Stress (lb/sq ft)	Permissible Velocity (ft/sec)	Citation(s) ([2])
Riprap	6 – in. d50	2.5	5 – 10	H
	9 – in. d50	3.8	7 – 11	H
	12 – in. d50	5.1	10 – 13	H
	18 – in. d50	7.6	12 – 16	H
	24 – in. d50	10.1	14 – 18	E
Soil Bioengineering	Wattles	0.2 – 1.0	3	C, I, J, N
	Reed fascine	0.6-1.25	5	E
	Coir roll	3 – 5	8	E, M, N
	Vegetated coir mat	4 – 8	9.5	E, M, N
	Live brush mattress (initial)	0.4 – 4.1	4	B, E, I
	Live brush mattress (grown)	3.90-8.2	12	B, C, E, I, N
	Brush layering (initial/grown)	0.4 – 6.25	12	E, I, N
	Live fascine	1.25-3.10	6 – 8	C, E, I, J
	Live willow stakes	2.10-3.10	3 – 10	E, N, O
Hard Surfacing	Gabions	10	14 – 19	D
	Concrete	12.5	>18	H

[1] Ranges of values generally reflect multiple sources of data or different testing conditions.

[2] Citations:

A. Chang, H.H. (1988).
B. Florineth. (1982)
C. Gerstgraser, C. (1998)
D. Goff, K. (1999).
E. Gray, D.H., and Sotir, R.B. (1996)
F. Julien, P.Y. (1995).
G. Kouwen, N.; Li, R.M.; and Simons, D.B., (1980)
H. Norman, J. N. (1975)
I. Schiechtl, H.M. and R. Stern. (1996).
J. Schoklitsch, A. (1937)
K. Sprague, C.J. (1999).
L. Temple, D.M. (1980).
M. TXDOT (1999)
N. Data from Author (2001)
O. USACE (1997).

Table 4-3 provides typical examples of permissible shear stress for selected lining types. Vegetative and RECP lining performance relates to how well they protect the underlying soil from shear stresses, so the permissible shear stress values are dependent on soil type.

4.5 Traditional Riprap Revetments

Traditional riprap coverage has historically been a standard and popular approach to bank stabilization. At many geographic locations, large volumes of small-sized rock are easily acquired at low cost. The size of rock can be matched to the erosive forces of low energy to high energy streams. A characteristic that makes riprap effective is that individual rocks can shift position and evolve to partially self-heal when flow patterns change, provided sufficient material is available. A thickened toe section can also be constructed to shift downward to protect against toe scour. In contrast, concrete walls or large blocks cannot adjust to flow patterns and can be undermined by erosion or scoured to failure at the fringes of the monolith. Riprap can be quickly placed during high flow events making it ideal for emergency response sites, while a more considered design that includes a good gradation, a deep toe and a filter can greatly improve the life of the installation.

Table 4–3. Typical Permissible Shear Stresses for Bare Soil and Stone Linings (FHWA, 2005)

Lining Category	Lining Type	Permissible Shear Stress	
		N/m^2	lb/ft^2
Bare Soil Cohesive (PI=10)	Clayey sands	1.8–4.5	0.037–0.095
	Inorganic silts	1.1–4.0	0.027–0.11
	Silty sands	1.1–3.4	0.024–0.072
Bare Soil Cohesive (PI≥20)	Clayey sands	4.5	0.094
	Inorganic silts	4.0	0.083
	Silty sands	3.5	0.072
	Inorganic clays	6.6	0.14
Bare Soil Non-cohesive (PI<10)	Finer than coarse sand D_{75}<1.3 mm (0.05 in)	1.0	0.02
	Fine gravel D_{75}=7.5 mm (0.3 in)	5.6	0.12
	Gravel D_{75}=15 mm (0.6 in)	11	0.024
Gravel Mulch	Coarse gravel D_{50}=25 mm (1 in)	19	0.4
	Very coarse gravel D_{50}=50 mm (2 in)	38	0.8
Rock Riprap	D_{50}=0.15 m (0.5 ft)	113	2.4
	D_{50}=0.30 m (1.0 ft)	227	4.8

Despite these advantages, riprap installations have become less-desired solutions as our understanding of the associated impacts has increased. Riprap has notable environmental and feasibility shortcomings:

- A bank protected with riprap promotes bank steepening due to increased velocities.
- Rock that has displaced vegetation has a lower hydraulic resistance that increases erosive forces.
- Steepened channel banks promote higher velocity, and thus more erosive flows.
- Bank area and habitat is reduced when the bank steepens, and the remaining bank area is transformed from the high diversity of a riparian environment to a relatively sterile condition of steep banks and rock coverage.

- As velocities increase and more flow is conveyed in the channel, the main channel grows disconnected from the floodplain increasing an erosive trend and deteriorating riparian conditions on the floodplain.
- Riprap increases lateral constraints on the channel, and increased lateral constraint and decreased overbank flow connection deteriorate riparian habitat.
- Despite a good gradation, large riprap can have large voids that drain quickly. Even in a channel with a good sediment supply, it may still take decades for sufficient sediment to be caught in the voids before moisture retention supports the establishment of vegetation. This scenario presumes the bank is not too steep to support growth.
- It is difficult to repair revetment at a later date without disturbing the vegetation that has established around the revetment, which may strengthen the structure.
- It is difficult to install or repair the toe of a revetment without dewatering the stream.
- Repairing a revetment by dumping material from the top causes rock separation and requires excess material that can be lost in the channel. Material lost in the channel can also cause channel constrictions under the wrong conditions.
- As the number of sites with revetment increases, finer materials in a riprap gradation mix can cause a coarsening of the reach for sand bed or gravel bed rivers. A coarsening of bed material instigates general river narrowing and shifts in channel form, i.e. a reduction in complex channel form or braiding and a decrease in width/depth ratio. And again this causes increased velocity and erosion.
- Riprap fixes the channel bank preventing bar formation and meander migration, a process that revitalizes and generates riparian habitat (Griggs, 2009).

Designers prefer riprap materials since the design of riprap installations is almost universally taught and readily learned by engineering students, yet there is little instruction on what to do when a site exceeds the limitations of a riprap design. Common pitfalls of riprap design include:

- A riprap installation can increase erosive forces due to steepened banks or lower bank roughness after vegetation is replaced by rock. Riprap designs should be based on future conditions not present conditions.
- Rock sizes should not be extrapolated beyond the limits of design charts or methods. Design charts are limited by the range of the laboratory or field study data source.
- Large rock sizes specified by the design may not be available in the

project area and possible sources should be investigated during the design process.

- All rock is not equivalent and a good quality of large rock that does not degrade or abrade can also be difficult to acquire.
- Reducing the peak flow selected for design should only occur if a corresponding increase in maintenance funds is set aside for more frequent repairs.
- Loss of riparian habitat and floodplain, including any reduction in overbank flows (loss of floodplain) due to increased flow velocities, should be considered and addressed in the standard riprap design procedure.

In low to medium energy systems, native materials may offer more benefits as bank protection, and in high energy systems, alternative methods including methods that lay back slopes and expand or reconnect the floodplain may be safer and more sustainable alternatives.

4.6 Sediment Analysis and Modeling

During the geomorphic assessment, the investigator can search in the field, in existing literature, and review aerial photos for indicators of vertical instabilities (deposition or aggradation) at the project site. Information on head cuts or downstream controls also helps define the level of uncertainty associated with the site. This qualitative information can help in the selection of bank stabilization methods. A sediment impact analysis provides quantitative information on the site and can provide more complex information on vertical stability. Many rivers are influenced by water operations especially where there are a number of facilities that control and manage the system. If scenarios are considered with nuances in how flow is added or withdrawn, a quantitative analysis can help detect the difference in sediment conditions between the proposed flow regimes. Modeling sediment conditions is always valuable to assess the long-term stability of restoration and channel maintenance actions. With more options for sediment modeling, and the increased availability of both good terrain mapping and sediment sampling data, sediment modeling is becoming more cost-effective in the design of many of today’s projects.

A calculated sediment budget approach can be used for relatively simple projects and the SIAM model (USACE 2010) can be used to calculate a sediment budget for multiple reaches and stream networks in watersheds. If more detailed information is required for the design, including predictions of potential aggradation or degradation under altered flow management scenarios, a numerical model can be used.

The same numerical models used in the computation of erosive energy (section 4.4 Computing Erosive Force), can be the basis of the sediment transport models, and both analysis can be carried out together. Numerical models include Reclamation’s series of flow and sediment transport models. Sedimentation and

River Hydraulics One-Dimensional Sediment Transport Dynamics Model (SRH-1D) can be used as a flow model or a flow and sediment transport model and incorporates the solution of sediment and water continuity to determine average conditions in a longitudinal direction. The Hydraulic Engineering Center- River Analysis System (HEC-RAS), the Corps of Engineers 1D flow model, also has sediment modeling capabilities. An advantage of 1D models is that there smaller data demands. Cross sections can be used to represent the dry and wet terrain and the terrain model can be constructed relatively quickly. For complex or detailed information on the site, 2D sediment models like SRH-2D (Lai, 2008) are recommended. Two-dimensional models address variations in sediment conditions across the channel, in addition to conditions in the longitudinal direction. Sediment information that is provided in the output is more detailed and provides point velocities that are more accurate than the average cross section velocities computed in a 1D model. One-dimensional sediment information can also only provide average longitudinal changes in depth, while the 2D model can present changes resulting from lateral features like bars.

5 Scour Assessment

Scour occurs in multiple forms, but in all cases can be defined as the removal of sediment from the bed and banks of a channel by the flow of water. Most forms of scour are a result of secondary flow patterns including horizontal or vertical rollers, or, flow turbulence. Secondary flow patterns are localized and move faster than the main current, creating more erosion on the bed and banks. Meander bend erosion, bank scalloping, and jet scour, are all processes that erode the bank. Scalloping is common to the ends of hard revetments when the revetment does not transition well to the softer surface of the bank. Bank scour is not included in the present discussion.

Vertical scour can threaten the foundations of a bank stabilization feature, but can also offer the benefits of improved habitat or flow conveyance at specific sites. A bend pool in the meander of a stream is habitat for some species, and is both formed and maintained by bend scour processes. Some features of design are a function of maximum scour depth, including the depth of revetment toe, foundations of a transverse structure, or the downstream extent of a riprap revetment. Depth and location of scour can be a determining factor in selecting one bank stabilization method over another. Forms of vertical scour, scour countermeasures and an approach to analyzing scour depth are introduced here. Equations for computing scour depth are presented in Appendix A, Scour Computation Methods.

5.1 Forms of Vertical Scour

General forms are bend scour, bedform scour, degradation, low flow channel incision and, in braided rivers, confluence scour. Bend scour and degradation should be considered at all sites. At locations adjacent to structures or obstructions, local scour and contraction scour are considered upstream of the structure, and culvert scour or contraction scour may be considered downstream. Scour associated with unique structures including transverse structures is also evaluated upstream or downstream of structures. Forms of scour are listed in table 5–1.

5.2 Descriptions

Features, structures or obstructions that can contribute to scour include stream bends or confluences, piers and abutments of bridges, culverts, box culverts, levees, drop structures, hard bank stabilization, and features that constrict flow or flow can impinge against. Natural hard features and constrictions that generate scour are canyons, large boulders or outcrops, root wads, woody debris piles, vegetated bars or islands, and recurring locations of ice jams.

Table 5–1. Forms of Scour

GENERAL FORMS OF SCOUR
Bend scour (not used with low-flow incision)
Bedform scour (sand and gravel streams)
Low-flow channel incision (thalweg shifts, not used with bend scour)
Confluence scour (multi-channel and braided systems)
Long-term degradation (geomorphic assessment, and stable slope and bed armoring computations)
NEAR-STRUCTURE SCOUR
Local scour- Pier scour and abutment scour (upstream of bridges or obstructions in flow) [1]
Contraction Scour (upstream and downstream of bridges and other constraining features) [1]
Culvert Scour (downstream of culverts) [2]
Scour at Unique Structures
Scour at boulders, trees, root wads, or concrete piles
TOTAL SCOUR
Summation of applicable General and Near-Structure forms of scour

[1] *Resource is FHWA HEC-18 (FHWA, 2012)*
[2] *Resource is FHWA HEC-14 (FHWA, 2006)*

5.2.1 Bend Scour

Flow patterns causing meander bend formation and migration, also produce bend scour. Sediment is eroded by a transverse roller of flow at the outside bank in a bend. The roller is a “secondary” current and scours sediment from the bed and outside bend, and deposits sediment at the inside of the bend and downstream. Because bends can migrate, bend scour is computed at all locations, regardless of presence or absence of bends under existing conditions. Four bend scour equations are available: Zeller, Maynord, Thorne, and USACE. See table 5–2 for source information.

5.2.2 Bedform Scour

Bedform scour occurs as part of the formation and movement of dunes in sand-bed or gravel-bed rivers. Dunes and troughs migrate longitudinally and troughs develop between dunes causing erosion of the channel bed. Bedform scour is often less than bend scour but migrates faster than a meander bend. Bedform scour may be a fraction of the dune height or depth of mean flow, and also a function of grain size. Deeper scour can occur with a uniform grain size (Raudkivi, 1990). It can be computed from methods described by Simons, Li and

Associates, and the value is compared to results from a dune scour equation described by Maricopa County. See Table 5–2 for source information in section 5.4.

Dune bedforms can migrate through a bend so bedform and bend scour values are summed. The larger value for bedform scour is used.

5.2.3 Long-Term Degradation

Degradation differs from most other forms of scour, since erosion is not the result of secondary flow patterns. A shortage of sediment supply can instigate degradation or a change in the base elevation of the channel can drive the upstream migration of a headcut. Figure 5–1 is a tributary of the Arkansas River that has incised 8 ft, presumably from changes in water management in the base level of the Arkansas River. The energy dissipation box acts as a bed control and blocks further upstream progression of degradation. There will be no incision upstream until the box is removed or lowered. See section 5-5.3 for more information on estimating degradation depth.

Figure 5–1. Example of channel degradation, looking upstream. Headcut migration upstream is blocked by the concrete energy dissipator that is now hung above the channel.

5.2.4 Low-flow Channel Incision

A low-flow channel is the small inset channel on the bed of the stream. It can form when the width-to-depth ratio of the main channel is large and the daily flows are significantly less than bankfull flows. The channel shifts laterally across the bed of the channel forming an independent meander pattern. When the low flow channel is not at the site being assessed, channel incision scour is added to account for a future lateral shift. An incision depth is selected based on local or regional conditions, or can be assigned 0.5–1 ft.

5.2.5 Confluence Scour

Confluence scour is common to braided sand-bed or gravel-bed rivers, but can be found wherever two branches of a river converge and create secondary erosive currents. An increase in total discharge, confluence angle (between 30 degrees and 90 degrees) or grain size can increase scour depth (Ashmore and Parker, 1983). In very large and fully braided rivers, field measurements of four to six times the mean depth of approach flow have been reported (Fahnestock and Bradley, 1973; Mosely, 1982; Ashmore and Parker, 1983; Klassen and Vermeer, 1988) when measured from the water surface. If two channels converge at 60 degrees, the Ashmore and Parker equation (1983) for sand to gravel grain size, estimates confluence scour as 3.1 times the average approach depth of the channels. See Appendix A for the equation.

5.2.6 Contraction Scour

Contraction scour is caused by flow confined and accelerated between erosion-resistant walls and can be readily detected at sites with a bridge deck producing pressurized flow. If the bed is not erosion resistant, for example at a bridge or a bottomless culvert, constriction scour erodes material across the full width of the channel bed. The deepest scour is within the structure but the scour pattern extends both upstream and downstream (Figure 5-2). Culverts can also cause contraction scour upstream and downstream, but within the structure the deepest scour is prevented by the hard bed of the culvert. Maximum contraction scour forms at the design flow or at bankfull flow depending on the configuration of the structure and terrain.

An example of contraction scour, and probably also abutment scour, is seen in Figure 5-3. The site was previously constricted by the abutments/embankment of a railroad bridge. The bridge deck is now missing but piles remain. A scour hole 4 ft deep is seen in the bed between the abutments, and the scour hole extends upstream and downstream for a total longitudinal distance of about 45 ft.

The footprint of contraction scour is difficult to predict without the use of a 2-dimensional model. Downstream distance is dependent on site geometry channel width and severity of constriction, Contraction scour is initially assumed to be potentially significant within about ½ channel width 50 ft upstream of the structure and extends a longer distance to about 1 to 1.25 channel widths downstream of the bridge. These distances are approximate, local experience, if available, should also be used. Use the Modified Laursen’s live-bed or clear-water equations for computing contraction scour (FHWA, 2012). See Appendix A for the live-bed equation.

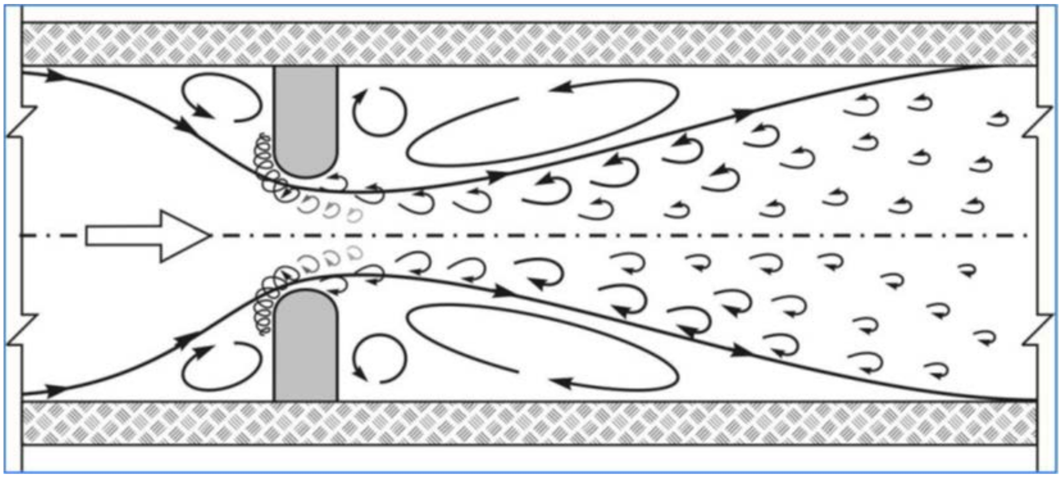


Figure 5–2. Flow structure including macro-turbulence generated by flow around abutments in a narrow main channel (NCHRP, 2011b), causing contraction scour.

Figure 5–3. Looking downstream past scour at old railroad embankment and former bridge crossing, and towards a more recently constructed railroad embankment and timber bridge. Scour may be the result of contraction and abutment scour.

5.2.7 Local Scour at Piers and Abutments

Local scour occurs when approach flow is impeded by a hard, vertical surface. Flow is diverted into a roller that spins towards the bed and removes material through accelerated flow (Figure 5-4). At a vertical obstruction like a pier, the roller spins off to both the left and right creating the signature horseshoe scour pattern in the bed of the channel. At abutments the flow can spin inwards and downstream after striking the structure. The deepest local scour occurs at the upstream face. Bed surface area where erosion occurs can be defined by a radius extending horizontally for a distance 2x the scour depth from the obstruction (FHWA, 2012).

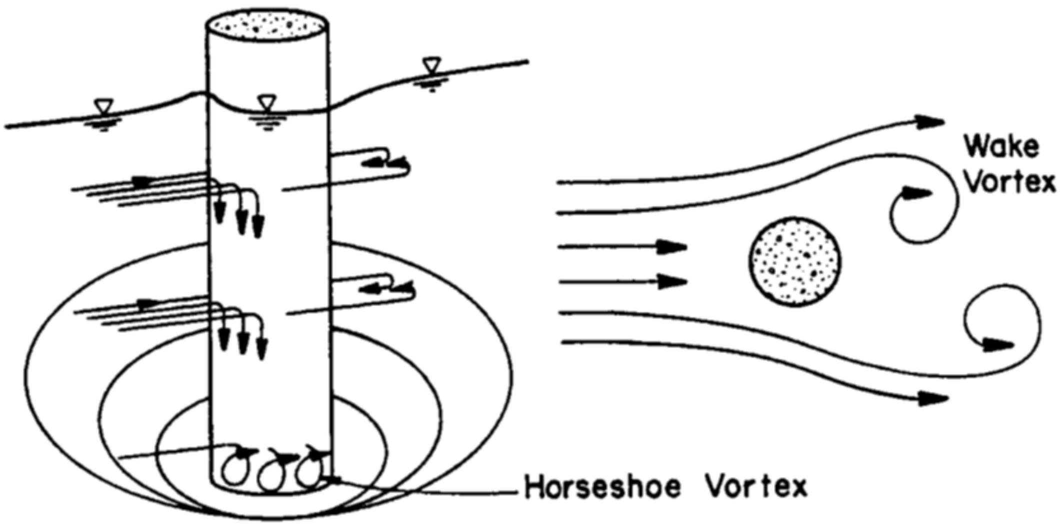


Figure 5–4. Horseshoe scour pattern in a laboratory flume at a circular pier (FHWA, 2012).

5.2.8 Wake Scour

Wake scour is caused downstream by flow vortices shedding off a pier or other obstruction in the flow. Wake vortices also cause erosion but this depth is shallow compared to the scour hole upstream of the structure (figure 5-4). Location extends downstream beyond the scour hole radius (> 2 x scour depth).

5.2.9 Other Forms of Structure Scour

Near-structure features that may generate unique scour forms include in-channel transverse features: bendway weirs, vanes, spur dikes or J-hooks. Always consider bend scour, bedform scour and degradation, but also compute near-structure scour using specific recommendations for these features. Consult available guidelines on a variety of pier and abutment shapes for a shape most similar to the project feature. There is specific guidance on scour downstream of some of these structures in Chapter 12.

5.2.10 Pipeline Scour

Pipeline scour is not a unique form of scour but is an assessment of scour depth at a pipeline crossing of a stream. Different forms of scour may impact the crossing and the maximum scour depth is computed to define the minimum safe burial depth for the pipeline.

5.3 Countermeasures

Five common approaches to protecting structures from scour are:

- move the design structure outside the range of scour, when another feature is causing the scour;
- construct the design structures, including pipelines, bridge pier foundations and the toe of bank revetment, below the maximum depth of scour;
- harden the surface using riprap or other structural protection measures, when the structure is not founded or buried sufficiently deep;
- install countermeasures or modify structure form to alter the scouring flow pattern, including specially formed bridge pier scour caps (Figure 5-5) or more pervious wood log jams; or
- move the structure out of high flow, for example suspending a pipeline over the channel.

The first two bullets are non-structural solutions and are the preferred countermeasures. Assessing scour depth early in the selection process of bank stabilization features may help determine when scour can be avoided, when foundations can be set deeper, or test if the design is suitable for the site.

Shortcomings of structural countermeasures include design and construction costs, the need for periodic inspection and possibly maintenance costs, and environmental degradation with bank or bed hardening solutions. Riprap material may need maintenance several times during the life of the design, due to the use of smaller and more readily available stone. Methods for designing and placing structural countermeasures at bridge foundations (methods can be adapted to other features) can be found in FHWA HEC-23 (2009).

5.4 Scour Assessment Method

Steps listed below outline the evaluation of potential scour depth. Some steps contain a reference to additional information in a later section.

1. **Identify all sites** for the scour evaluation, and the types of scour that need to be evaluated at each site. When trying to determine the depth to set the toe of a riprap revetment, evaluate bend scour depth and degradation scour depth. Also check if other forms of scour could affect the site. See section 5.5.1 for additional information.
2. **Calculate the design discharge** and bankfull discharge for each site, based on the risk assessment in section 2.3. The return interval of a design discharge is often a larger number of years than the design life of the

structure, i.e. a 100-yr design discharge may be selected for a 50-yr design life.

3. **Acquire hydraulic data** for scour equations at each site. Hydraulic data from a one-dimensional or two-dimensional flow model, or from a flow and sediment transport model, may be needed for more complex sites. Survey a cross section and bed profile or cut these sections from mapping for a less accurate estimate of hydraulics, assuming normal depth (no backwater computation).

4. **Acquire sample of the** gradation and cohesion that represent the soils under the bed of the channel. Remove most of the armor layer before shoveling a bulk sample since this concentration of larger particles can skew the sample. Soil gradation and cohesiveness are assumed to be consistent in the bed of the channel for the complete depth of scour. See section 5.5.2 for additional information.

Figure 5–5. Depth of pier scour hole is partially limited by the pile cap/footing that interrupts the downward diving flow of the secondary flow pattern. This protection is lost if degradation erodes the bed and exposes the larger diameter of the pile cap to approaching flow.

5. **Select general forms of scour** to be evaluated. Degradation and bend scour are evaluated at all sites. Other forms to consider are bedform scour, low-flow incision, and confluence scour.

6. **Degradation depth**. Evaluating degradation depth is more arbitrary than other scour depth computations but can be improved by sediment transport modeling, a “depth to armor” assessment, a “slope stability” assessment (Pemberton and Lara, 1984), field identification of downstream bed controls including culverts, and/or a geomorphic assessment. Integrate available information with a risk assessment for this project to assign a degradation depth at each site. Any long-term aggradation that might reduce scour is generally not included in the total scour calculation. For more information see section 5.5.3.
7. **Bend scour** in a meandering channel is the result of transverse or “secondary” currents that scour sediment from the outside of a bend. Bend scour can be computed from four bend scour equations: Zeller, 1985; COE, 1984; Thorne, 1995; and Maynord, 1996 in the PBS&J spreadsheet. One method may more closely match the project conditions or an average of two to four equations can be used. Select bend severity for the project site that represents more severe upstream and downstream bend angles, although no bend may be present. It is assumed a bend can migrate to the crossing location during the life of the project.
8. **Bedform scour** is an allowance for anti-dune or dune trough depths that occur in sand and gravel channels. It is computed from Simons, Li and Associates (1985) methods, and the value is compared to results from a draft dune scour equation from Maricopa County (2003) in the PBS&J spreadsheet. The larger result from the two methods can be selected.
9. **Low flow incision** Include the low flow channel depth (can use 0.5 to 1 foot) if there is a low flow channel in the stream but not at the evaluation site. When there is a low flow channel in the cross section, there is no need to add this depth for evaluating a pipeline crossing. Low flow incision could also be added when there is insufficient detail in the cross section to identify a low flow channel. See section 5.2.4 for additional information.
10. **Confluence scour** is the result of two converging flow paths in braided streams that have multiple flow lines in a cross section. Estimates of scour depth are based on very large rivers and fully braided rivers with unlimited sediment loads. See section 5.2.5 for additional information.
11. **Evaluate near structure scour, upstream of structures.** Compute local scour at piers and abutments, or at the headwalls of some culverts**,** if the site is near these structures (see section 5.2.7). The range of scour effect can be estimated as a distance of twice the maximum depth of scour for the area (sometimes 60 ft upstream of the structure). Evaluate contraction scour (see section 5.2.6) and pier scour, and evaluate abutment scour with contraction scour to determine the location that has the deepest scour. HEC-RAS can be used to compute most scour values and deepest location. See section 5.5.4.
12. **Evaluate near structure scour, downstream of structures.** Evaluate contraction scour downstream of bridges (see Section 5.2.6), and culvert scour downstream of culverts. Confining bridges or bridges with wide decks

can have flow patterns similar to a culvert, while a box culvert or multiple box culverts may, at some locations, be better evaluated as contraction scour at a bridge. For evaluating culvert scour, use the software or equations to evaluate the depth and length of the scour hole. Also calculate the location of the deepest point in the scour hole.

13. **Adjust near-structure scour depths for distance** between the location of maximum scour and the site of the proposed bank stabilization feature (or pipeline crossing). A slope of 1:2 might be used as a scour depth reduction rate for local scour features, but contraction scour and culvert scour may extend farther and have a more gradual reduction in scour depth. One countermeasure is to relocate the bank stabilization feature or design feature (pipeline crossing) beyond the extent of the near-structure scour hole. See section 5.5.5.

14. **Sum all overlapping scour forms at the site.** There may be multiple forms of scour reshaping the channel at the design site, depending on terrain, features, soils, stream form and flow conditions. A total value is computed by evaluating potential depth for each form of scour, estimating potential depth for degradation, considering location for each type of scour and assembling the worst-case scenario with overlapping scour forms that erode at the bank protection site. Before summing, convert all forms of scour to depths measured from the bed of the channel. See section 5.5.6

15. **Re-evaluate the total scour depth.** Check results by comparing total scour value to available data for similar river conditions, or look for consistency based on field observations and previous findings. One source for comparison is a Pemberton and Lara (1984) method referenced as the “USBR envelop curve” (ASCE, 2005). An envelope curve was developed from field measurements at several USGS gage sites during high flow events and verified with data from five additional New Mexico streams. Stream form for the study sites was described as wide, sandbed, and ephemeral. Reassess factors in the analysis if total scour depth is unreasonably high or low. See section 5.5.7.

16. **Re-evaluate the design**. Consider the proposed bank stabilization design with respect to total scour and with respect to countermeasures. Can the design be shifted to reduce the impacts of scour and/or can the structure or structure foundation be buried sufficiently deep to avoid failure? Also consider construction concerns. Excavation at a wet site can be challenging and a deep stone toe is difficult to construct well if the site cannot be de-watered.

Equations for general forms of scour are referenced in table 5–2 and equations for scour near structures are referenced in table 5–3. Some of the scour equations are also presented in Appendix A. A scour spreadsheet, version 1.2 (May 28, 2008), developed by the firm PBS&J, automates degradation computational methods from Neil and Pemberton and Lara, and bedform and bend scour equations referenced in table 5–2. "General" scour computations in the spreadsheet are an earlier version of bend scour equations and are not used in this approach.

5.5 Scour Computation Topics

Sections below expand on steps in the scour assessment process.

5.5.1 Site Identification (Step 1)

Scour should be evaluated at all sites when considering different bank stabilization options. In pipeline scour studies, scour is evaluated at all drainage crossings that have potential to exceed the minimum pipeline burial depth. Although large discharge rates can cause deep scour, sites with low discharge should be considered if: there is a constricted floodplain, a steep bedslope, a location immediately upstream of a bridge or downstream of a culvert, or if the site is susceptible to degradation from headcuts.

Table 5–2. Equations for General Forms of Scour

Scour Form	Equation	Source
Bend Scour	Zeller Bend Scour	Simons Li & Associates (1985); pp. 5, 105 106
	Maynord Bend Scour	Maynord (1996)
	Thorne Bend Scour	Thorne et al. (1995)
	USACE Bend Scour Design Curves – sand	EM 1110-2-1601, Plate B41, in USACE (1994a)
Bedform Scour	Simons Li & Associates	Simons Li & Associates (1985)
	Dune Scour Equation	Flood Control District of Maricopa County (2003), as presented in the PBS&J Scour Spreadsheet (PBS&J, 2008)
Confluence Scour	Confluence in sands & gravels	Ashmore and Parker (1983), as presented in Melville and Coleman (2000)
Long-Term Bed Degradation:	Schoklitsh Method	Schoklitsh (1932) as adapted by Pemberton & Lara (1984)
	Meyer-Peter, Muller Method	Meyer-Peter and Muller (1948) as adapted by Pemberton & Lara (1984)
Stable Slope Equations	Lane's Tractive Force Method	Lane (1955) as adapted by Pemberton & Lara (1984)
	Shield's Diagram	Shield (1936), as adapted by Pemberton & Lara (1984)
Bed Armor Equation	Shield's Incipient Motion	Shield (1936) as adapted by Pemberton & Lara (1984
Scour Comparison	Reclamation Envelope Curve	Pemberton & Lara (1984)

Table 5–3. Scour Equations for Sites Adjacent to Structures

Contraction Scour Equations (in constriction, at ends, downstream) Laursen Live Bed Equation (Laursen, 1960) Laursen Clear Water Equation (Laursen, 1963)
Equation for Local Scour at Piers (upstream of bridge) Richardson, et al. Equation (Richardson et al., 1990)
Equations for Local Scour at Abutments (upstream of bridge) Froehlich Equation (Froehlich, 1989) – in HEC-RAS HIRE Equation (Richardson, 1990) – in HEC-RAS NCHRP 24-20 Equation (FHWA HEC-18, 2012)**
At Unique Structures: bendway weirs, vanes, spur dikes, J-hooks, or constructed log jams Use scour equations specific to structure
Large boulders, trees, root wads, or concrete piles (upstream) Local scour guidance (FHWA, 2012) including Richardson, et al. Equation for piers (Richardson, et al. 1990)

** Not available in HEC-RAS.

As an example approximately 335 pipeline crossings of drainages were initially identified as potential scour concerns in a detailed stream hazard study for design of a proposed pipeline (Sixta et al., 2015). The alignment was located in a semi-arid region east of Pueblo, Colorado. Based on aerial photos, brief field reviews, and preliminary measurements of drainage areas, 71 % of small drainages and irrigation ditches were dismissed as having no potential for scour to exceed a 5 ft burial depth. There remained 57 sites with minimum potential and 41 sites with high potential and all were evaluated for scour. Scour depth exceeded burial depth at 14% of sites with low potential to scour, and scour depth exceeded burial depth at 12.5% of the original 335 sites. Results are shown in table 5–4.

Table 5–4. Summary of Sites Evaluated for Scour at Pipeline Stream Crossings for Example Project (Sixta et. al., 2015)

Estimated Potential to Exceed Burial Depth	Number of Sites	Number of Sites Exceeding 5' Minimum Burial Depth	Percent Exceeding Burial Depth
High	41	34	83
Low	57	8	14
Unlikely	237	Not Calculated	-
Total	335	42	12.5

5.5.2 Effects of Sediment Gradation on Scour (Step 4)

There is a clear-water and a live-bed equation for estimating contraction scour depths to reflect sediment supply from the bed. There are also two culvert scour equations, one for non-cohesive soils (granular) and the second for cohesive soils with a plasticity index of 5 to 16. Briauds (2011) model in Figure 5-6 suggests there will be deeper scour at pipeline crossing sites with fine to medium sandy soils and less scour at sites with a higher percentage of clay.

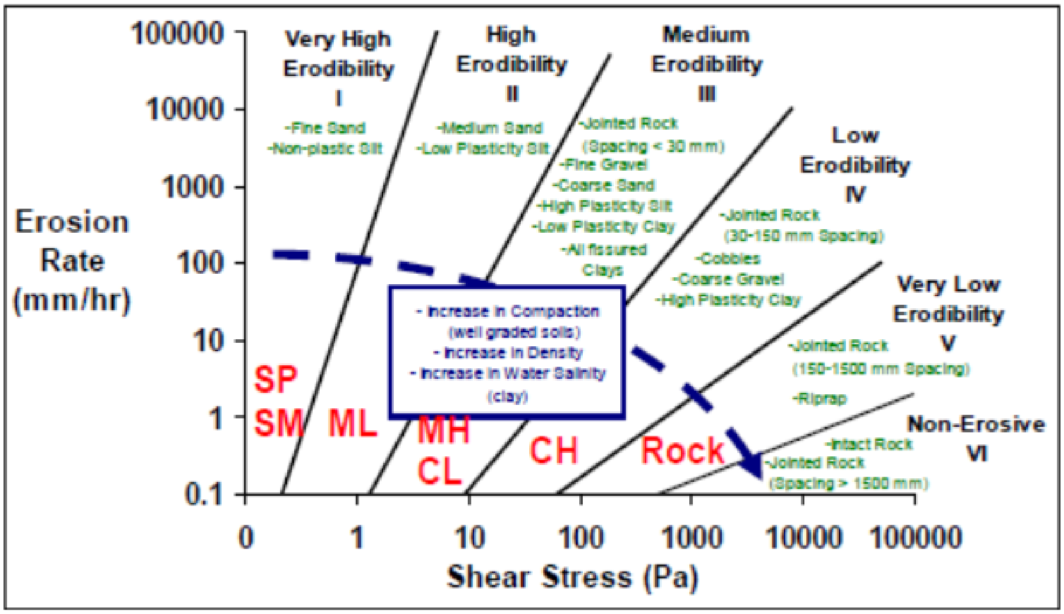


Figure 5–6. Generalized relationships for scour in cohesive materials (Briaud et al. 2011). Figure 6-11 from HEC18, 2012.

5.5.3 Computing Degradation and Considering Risk (Step 6)

There is not a definitive means of predicting future degradation at a site, and it is highly dependent on both current and future conditions in the downstream drainage. Degradation can occur as a shortage in upstream sediment supply or as headcuts (knickpoints) migrating upstream in the drainage as a result of changes in downstream bed level elevation. Methods of estimating degradation include numerical models, slope stability analysis, armoring analysis, downstream bed controls and a geomorphic assessment.

Method 1: Numerical Model. A 1D flow and sediment transport model is one means of computing degradation resulting from a shortage in sediment supply if sediment boundary conditions are specified. However, the progression of head cuts can be difficult to predict. When the modeled estimate of degradation does not account for head cut potential from base level lowering, this value can be added from a geomorphic assessment of the area. The sum is a preliminary degradation depth.

Slope Stability. A slope stability assessment and a depth to armor assessment proposed by Pemberton and Lara (1984) can be used to estimate degradation depth (Table 5–2). Pemberton and Lara use four methods to compute the equilibrium slope at the site: Schoklitsh, Meyer-Peter, Muller, Lane's Tractive Force, and Shield's Diagram (Table 5–2). Equilibrium slope is a function of bankfull depth and sediment grain size and is the stable slope the channel will evolve to given sufficient flow and time. A downstream distance is multiplied by the difference between the computed average stable slope and the existing slope at the pipeline crossing. Distance to the closest downstream control, often a culvert, is used when known, or, a standard distance can be selected. A stable slope computation can produce very large depths of erosion in steep terrain.

Armor Layer. The degradation depth required to develop an armor layer and prevent further bed degradation, is estimated from Shields Incipient Motion Equation (1936) as adapted by Pemberton and Lara (1984, table 5–2). A gradation for the study site or area, developed from sediment sampling, is used in the depth to armor assessment. There can be locations where $d_{critical}$ is larger than the largest size in the gradation, implying insufficient coarse sediment to halt channel incision through armoring. When the estimated depth is smaller than calculated with the stable slope method, the bed of the channel should harden before it can incise down to a stable slope. Equations for both assessments are presented in Appendix A, and values can be computed using the PBS&J spreadsheet (2006).

Method 2: Slope Stability, Armor Later and Bed Control. Degradation depth can also be computed from a combination of a slope stability analysis, an armoring analysis and/or from observations of downstream bed controls including bedrock or culverts. Fine soils and higher terrain relief cause larger depth estimates of degradation in the slope stability analysis. Degradation depth from the bed armoring computation may be less if there are coarse grains in the bed material that are larger than the value for incipient motion. These grains would have to be correctly represented in the sediment sample. Known downstream bed controls, combined with a slope stability analysis also improve the estimate of degradation. Select the dominant condition, often the smallest value as a preliminary degradation depth.

How Risk Can Effect Degradation Depth. Compare preliminary degradation depths and rating of vertical stability from the geomorphic field assessment, and integrate values based on the risk assessment from Chapter 2. The Federal Highway Administration recommends summing total degradation depth and total scour depth at a bridge site (FHWA, 2012) since there is a high risk of loss of life during a bridge failure. When level of risk associated with the structure failure does not match bridge failure concerns, a fraction of the total estimated degradation depth might be used. Total scour depth however should approach a total degradation depth that is consistent with field observations.

For example, consider a site with low, risk-to-life at failure, and a low-to-moderate geomorphic rating of stability. Scour components total 6 ft: 5 ft of bend scour and 1 ft of bedform scour and the site is not in a bend. The site has a computed degradation depth of 10 ft, and field observations of degradation ranging from 2 ft to 8 ft At a high risk site, scour components (6 ft) could be summed with the degradation computation (10 ft) specifying a conservative foundation depth or burial depth of 16 ft An alternative approach for the low risk site is to adjust the computed degradation rating for the risk and stability of the site. Adding 2 ft of degradation depth to 6 ft of scour components provides 8 ft total scour depth, a value protecting against the maximum field observations of degradation, protecting against all potential scour and some degradation depth, or protecting against the occurrence of some combination of the two factors.

5.5.4 Modeling Flow and Bridge Scour (Step 11)

One-dimensional flow models including SRH-1D (Huang and Greimann, 2013) or HEC-RAS will provide a water surface elevation and hydraulic information including flow area, depths, average flow velocities and shear forces, for calculating scour. Two-dimensional modeling that provides flow direction for points in the flow field is often justified at sites with complex flow patterns or medium and large streams to identify likely scour sites. HEC-RAS has scour computing functions for contraction scour and local scour at bridge piers and abutments. Options for contraction scour are Laursen's clear water equation (1963) or Laursen's live bed equation (1960). The model options for pier scour are Richardson, et al (1990) or the Froelich (1988) equation. Woody debris accumulations on the piles or piers (Figure 5–7) can be represented by increases in pier width. Contraction scour and pier scour are added for total depth of scour at the pier.

HEC-RAS computes abutment scour using either the Froehlich (1989) or the HIRE (Richardson, 1990) equation. The selection can be different for each abutment and is based on a ratio of abutment length to approach flow depth, L/y. The NCHRP 24-20 equation was added to the HEC-18 manual in 2012 as a third method of computing abutment scour. The NCHRP 24-20 method appears least conservative and is recommended by FHWA, but is not available in HEC-RAS. Contraction flow patterns are the basis of the NCHRP 24-20 abutment equation and the computed value for abutment scour also incorporates contraction scour in the single value.

Figure 5–7. Debris caught on pile-piers of the Old Bridge from spring flows at the Little Colorado River, April 2013.

5.5.5 Extent of Near-Structure Scour (Step 13)

When the study site is not close to the structure creating scour, actual scour at the study site is estimated based on descriptions of the location of near structure scour.

Pier and Abutment Scour. Measurements from laboratory flume studies reported by the Federal Highway Administration (FHWA) indicate the radius of a scour hole that develops at a structure (local scour), can be 2 times the depth of the scour hole in a sandy bed (FHWA, 2012). The area affected by local scour extends in a horizontal radius upstream of the structure (pier or abutment) for a distance of twice the local scour depth (FHWA, 2012).

Contraction Scour. Contraction scour at a bridge may extend further downstream than local scour effects and Devadason (2007) reports the distance is dependent on the flow velocity with scour extending further downstream under higher Froude numbers. He also describes a uniform scour depth for the extent of the scoured area with a depth which is more than at least 50% of the depth of the maximum scour hole. The uniform scour hole extends well beyond the maximum scour hole and has a fairly even bed elevation.

Culvert Scour. Culverts can have distinct forms of scour and FHWA HEC14 (2006) guidelines should be used to estimate culvert scour in place of contraction scour computations. This is especially true when there is a drop at the outlet of culverts. Guidance is also provided on the extent of the scour hole. When pipelines or bank revetments are located proximal to culverts, the location should also be evaluated for bend scour, bedform scour and degradation. If the culvert has a bed, and the structure is located upstream of the culvert, degradation from headcuts does not need to be considered since the culvert acts as a control.

Degradation Headcuts. If there is an option on the location of the design feature, locate the pipeline or bank stabilization feature outside the limits of the local scour and contraction scour concerns, and locate upstream of control features including culverts or rock outcrops. Degradation from headcuts does not need to be considered for these locations. Degradation that migrates upstream as knickpoints or head cuts can be stopped by the bed control, and the control prevents degradation from undercutting the foundation of the upstream bank stabilization feature.

5.5.6 Identifying Maximum Local and Contraction Scour (Step 14)

A HEC-RAS model will automatically sum the overlapping forms of scour and compare scour depths across the profile to determine the location of deepest scour. Scour depth is presented visually at the bridge cross section as shown in figure 5-8. Note the depth of scour at the abutment is measured down from the abutment cross section point. Calculated scour depth values for abutment scour can appear very large until the bank height is subtracted, and values are adjusted to the bed of the channel. Pier scour may need to be similarly adjusted if a pier is located on the channel side slope (spill-through slope). Without the aid of the HEC-RAS figure, adjust all scour depths to a similar reference point, often the bed of the channel, before determining maximum depth. As described above, contraction scour and pier scour are added, and contraction scour and abutment scour are added, with the exception of abutment scour computed using the NCHRP 24-20 abutment equation. Summed values are compared to determine maximum near-structure scour depth.

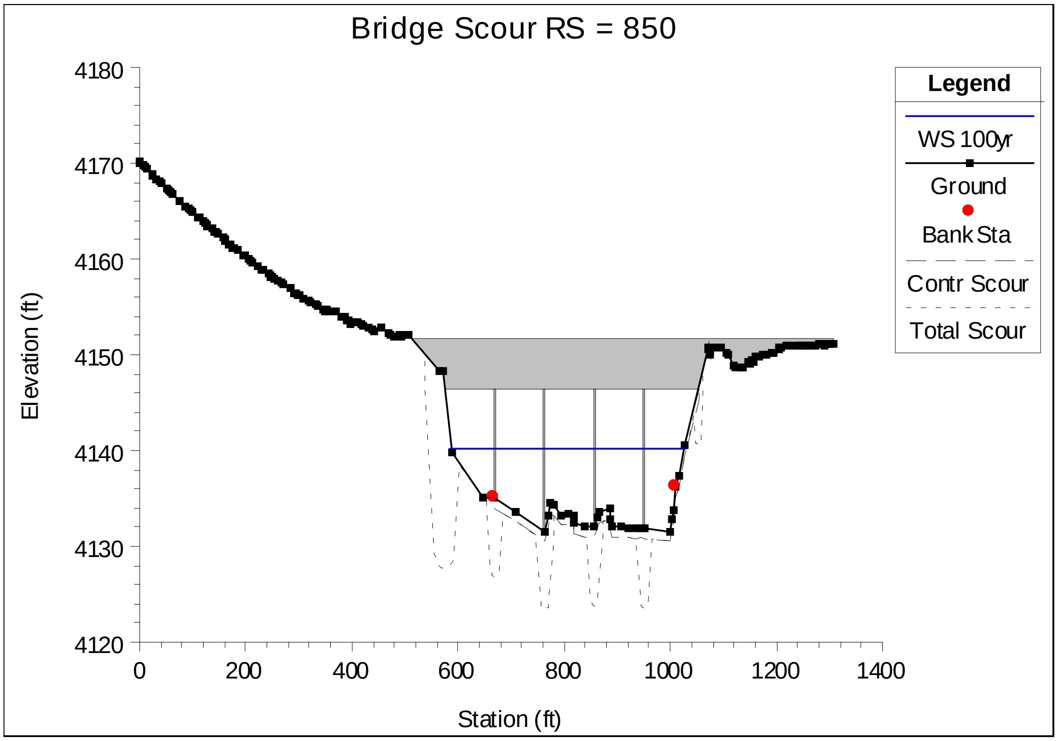


Figure 5–8. Arkansas River at Rocky Ford bridge crossing. Largest scour value occurs at the left abutment but pier scour penetrates deeper under the channel bed. Depth of scour at this bridge is pier scour added to contraction scour as shown.

5.5.7 Safety Factors (Step 15)

A factor of safety was applied to scour depth in early scour approaches including Neil (1973), and Pemberton and Lara (1984). A factor of safety is not included in the FHWA, 2012 recommendations for high or low-risk bridges. Less frequent use of a safety factor in recent approaches may be due, in part, to three decades of laboratory flume studies that have improved definition of forms of scour, and improved the estimates of depth. Safety factors have also been integrated within some computations of specific scour forms, and general conservative steps have been included in the assessment. One conservative step is including scour depth for a moderate or severe bend at all sites. Bends are not always present, cannot always migrate to the site, and are not always severe. Other conservative assumptions in more recent approaches is summing all scour components that have even a small likelihood to occur coincidentally, and the number of scour forms considered at each site has increased. If there is sufficient uncertainty and risk associated with the scour analysis to justify the increased cost of deep burial, a factor of safety of 1 to 1.5 might be used (ASCE, 2005), but it is not proposed for every assessment.

5.6 Basis of Scour Guidelines

Practical approaches founded on classic studies of river stability, sediment transport, and initial scour investigations dominated the first period of scour evaluation up to the mid-1980's. Thirty five years of scour laboratory flume studies followed, often funded by the FHWA bridge scour program. Field studies and risk analyses also contributed to this topic. Large advances were made in the understanding and quantification of bedform scour, bend scour, culvert scour, local scour (pier and abutments) and confluence scour, and in the realm of hydrologic risk and risk assessment. These are major elements in both bank stabilization and pipeline scour studies. Main sources for the scour assessment method presented in the previous section are:

- general methods, bend scour and degradation methods presented in the text by Neil (1973);
- general methods, bend scour and degradation from the original Reclamation standard by Pemberton and Lara (1984);
- bedform scour studies by Simons, Li, and Associates (1985);
- bend scour studies by Zeller [Simons, Li & Associates, 1985], USACE, 1994a, Thorne et al., 1995, and Maynord, 1996;
- HEC-RAS version 4.1 (2010);
- summary text of bridge scour research by Melville and Coleman (2000);
- ASCE pipeline scour short-course by Williams (ASCE, 2005);
- culvert scour studies summarized in HEC14 (FHWA, 2006);
- PBS&J's Scour Analysis Spreadsheet v. 1.2 (c 2006, 2008, PBS&J); and
- FHWA bridge scour research summarized in HEC18 (FHWA, 2012).

Neil (1973) used the term general scour for contraction scour and bend scour at a bridge. General scour was calculated from a contraction scour equation, multiplied by a factor of 2.6. Pemberton and Lara (1984) also used the term general scour, but subtracted the contraction scour multiple (2.6 – 1) leaving the bend scour multiple (1.6). A general scour estimate was applied at sites distant from a bridge. In the period of approximately 1990 to 2005 both bend scour and general scour computations were included for a total estimate of scour, and in addition, a factor of safety could also be applied. Because summing both general scour and bend scour double-counts the same erosion process, general scour has been removed from FHWA guidance (FHWA, 2012) and is not included in this approach. This is one of multiple advances that have emerged in the last ten years of synthesis of scour studies. Listed below are advances that have been integrated into the approach presented here for evaluating scour:

- Removal of general scour, a double-counting of bend scour, from the list of scour components.
- Removal of the use of a generic factor of safety, since the computation of all forms of scour has improved.
- Improvements in use of abutment scour equations and clarification of contraction scour use.
- Better use of location, for each scour type to determine where scour impacts overlap and should be summed.
- Integration of risk assessment into scour depth assessments
- Improved integration of soil descriptions in the computation of scour
- Re-assessment of total scour value, and
- Re-assessment of countermeasures and structure selection

After evaluating the scour depth and location at the study site, consider if scour depth can be reduced through the preferred countermeasures. Can shifting the proposed structure away from other features reduce scour depth or can the structure foundations be constructed to a depth below the scour? Also, which bank stabilization functions are suitable for the scour conditions at the site? In the next chapter, the energy in the flow is also considered to help determine a suitable bank stabilization feature for the site.

6 Selecting a Bank Stabilization Method

At this stage in the design, the designer has identified the:

- Design Criteria including Environmental Requirements,
- Risk Assessment,
- Site Hydrology, and
- General Site Data

The designer has also performed or acquired the following assessments on the project site:

- Geomorphic Site Assessment,
- Hydraulic Assessment of Energy, Shear Forces, Sediment Transport and Plan Form, and a
- Scour Assessment

With this information in hand, the designer can select a suitable bank stabilization method. Brief descriptions of bank stabilization methods are presented in the next sections, and included are two tables summarizing the advantages and disadvantages of each method.

The designer selects the most appropriate method by matching project requirements and river site conditions, to a method with compatible features and acceptable drawbacks. Results from the hydraulic, geomorphic and scour assessments help to best select a suitable method. The energy/shear forces estimates for the site are likely the most telling. Not all methods are appropriate for highly erosive sites but the alternative to hardened banks can also be re-establishing floodplain area to reduce the erosive pressures occurring at the site. A design may also integrate a combination of options to arrive at a successful river engineering solution. Ultimately the design should be feasible, sustainable, and environmentally effective.

6.1 A Comprehensive Selection Process

The selection of an appropriate method takes into account the broadest context possible for the site and for the design life of the project. The evaluation and selection of methods involves:

- Forecasting the most likely future conditions for each alternative.

- Comparing each alternative condition with the estimated future condition without the project.
- Summarizing the effects of each alternative in terms of magnitude, location, timing, and duration.
- Determining alternatives that meet a minimum standard of project objectives.

The suitability and effectiveness of a given method are a function of the method properties and the physical characteristics of the worksite. There is no single method that applies to all situations and appropriate actions can be determined by evaluating method characteristics, ecological benefits and effects, geomorphic response, the range of applicable river conditions, and both initial and long-term costs. Method selection can be coordinated with other agencies to ensure compatibility with basin wide goals and long-term sustainability and effectiveness. Involving stakeholders and regulatory agencies can inject a stronger multidisciplinary vision that leads to a more robust solution.

Methods that work best with geomorphologic process and within existing constraints should be favored over methods that impose a local change to the geomorphic process to meet project goals. Sustainability is increased when the methods treat the cause of bank erosion rather than the symptoms. For example, this may involve increasing sediment supply, reducing sediment transport capacity with longitudinal bank lowering, removing or relocating lateral channel constraints, or bed stabilization. In all cases, maximizing sustainability and reducing future requirements should be a consideration.

Value weighting is a part of the methods selection for river projects. High value resources and requirements (public safety, habitat for endangered species, uninterrupted water supply or access) and a longer period of sustainability justify more dollars spent. Relocating infrastructure outside the floodplain often brings greater sustainability and environmental effectiveness at a lower long-term cost. Similarly a relocation of a channel can reduce long-term maintenance costs and increase quality and sustainability of riparian habitat.

Eliminating alternatives due to high initial costs for design and construction/implementation can be a false economy if the selected method includes unacceptable long term costs. These costs can manifest as issues concerning infrastructure maintenance, river sustainability, public safety, and detrimental impacts to environmental habitat and water quality. A comprehensive review of both short-term and long-term costs is necessary for an accurate assessment of project costs and successes under the trifecta of a feasible, sustainable and environmentally effective solution.

6.2 Methods

Six categories of bank stabilization are introduced in the sections that follow.

- Preserve the Floodplain
- Construct Channel Elements
- Construct Banks with Native Materials
- Construct a Natural Channel
- Add Transverse Elements
- Harden the Banks

Bank stabilization categories are generally ordered from the most preferred to the least preferred although there is no clear leader in the middle grouping of methods. Most categories include more than one method of bank stabilization. Each method is introduced briefly below, and a detailed description of the method advantages and disadvantages can be found in Appendix A. Tables summarizing the advantages and disadvantages of the methods in each category are presented in the last section.

Methods in the preserving the floodplain category are the most desirable in addition to methods in any category that reconnect or expand the floodplain, while also maintaining sediment continuity. The category ranked second is constructing channel elements that restore channel function or increase floodplain area. Hardening the banks is the least desirable category of methods and the category listed last. The remaining three categories are preferred over hardening the banks but the order of preference may be interchangeable depending on site conditions and project design criteria. Relocating a natural channel may be the preferred option if it expands floodplain access, or, could be the least desirable option if it is poorly designed without consideration of incorporating river function (flow conveyance ditch).

Selecting a preferred option may consist of working down through the methods in each category and selecting one found in the highest category that best matches site conditions, design criteria, and the engineering trifecta goal for a sustainable, feasible and environmentally effective solution.

6.3 Preserving the Floodplain

The preferred approach for bank stabilization is, in most cases, the one that preserves the greatest extent of the floodplain. An integrated and unrestricted floodplain allows the natural stabilizing processes of the river to evolve unimpeded. Natural processes tend to adjust channel form to the lowest erosive energy on the river banks for the conditions, and to maximize environmental benefits. Methods that help preserve the floodplain can include:

- Infrastructure relocation or setback,
- Conservation easements and vegetation buffer zones,
- Longitudinal bank lowering,
- Adding side channels (see constructing channel elements), and
- Channel relocations (see construct a natural channel).

Each method can evoke a unique channel response and meet different requirements for each project. All methods for a sustainable, stable channel are dependent on understanding the erosive forces at work in a stream or river system.

6.3.1 Infrastructure Relocation or Setback

After assessments of historical river stability, erosive energy at the site, available ecological resources, habitat requirements for the area, and realistic estimates of construction, long-term maintenance, and environmental costs, the preferred solution may be to move the infrastructure, not force stabilization on the river. This option may be more attractive in rural areas when assessed over a short-term period but can also be practical in developed areas when considering the longer project life span.

Disadvantages are temporal demands to accomplish land acquisition planning and implementation, and the costs of design and construction of new infrastructure. One advantage may be the opportunity to expand floodplain access locally, possibly reducing erosive pressures at this site and at adjacent sites. Cost savings may also be realized from a reduction in long-term maintenance, prevention of lengthy environmental conflicts, and prevention of environmental losses and costs.

Relocating riverside infrastructure may provide the best opportunity for geomorphic processes to occur unencumbered by local lateral infrastructure constraints. This method can encourage geomorphic processes to continue and may provide for long-term channel dynamic equilibrium (Newson et al., 1997; Brookes et al., 1996). These processes include lateral migration, which maintains the health of the riparian zone through erosion of banks and sediment deposition. Bank erosion can remove older growth riparian areas, while deposition can create new flood plain and riparian areas, thus maintaining a riparian zone with a mosaic of different age classes of native plant communities (Brookes, 1996).

Levee relocation can provide the potential for river flows to access historical flood plain areas (Bauer et al., 2004; Brookes, 1996; and Petts, 1996). The magnitude and frequency of access depend on local topography and availability of flows that go overbank into adjoining flood plain riparian zones. Levee relocation can provide opportunity for the river to relocate into historical channels and oxbows in flood plain areas cut off by levee installations (Bauer et al., 2004; Brookes, 1996), depending on local topography and channel changes since levee installation.

For cases in which lateral bank erosion is threatening the integrity of a riverside facility, relocation can allow continued lateral migration. When riverside infrastructure is placed outside the meander belt width or braid plain, future bank protection is generally not needed.

For the case of incised channels, lateral migration may provide an opportunity to establish a new inset flood plain and riparian zone surfaces. This is especially important when incision has led to the main channel being disconnected from its historic flood plain. Riparian zones that are narrower than historical widths have been considered successful rehabilitation projects (Brookes et al. 1996; Kondolf et al. 2007).

Levee relocation, in many reaches of the Middle Rio Grande, involves moving the existing levee and the riverside drain. When a riverside drain is reconstructed, the drain excavation may provide material for the relocated levee, thus leaving the existing levee to provide some small amount of sediment enrichment if the project is located in a reach that is supply-limited.

6.3.2 Conservation Easements and Vegetation Buffer Zones

Conservation easements and vegetation buffer zones preserve and/or enhance the riverine corridor. Conservation easements are land agreements that would limit or prevent development from occurring and allow fluvial processes to continue. Vegetation buffer zones between the active river channel and riverside infrastructure can reduce the need for future bank stabilization.

Conservation easements also preserve the riparian zone in its current state and future states as determined by fluvial processes and flood plain connectivity. Vegetation buffer zones between the active river channel and riverside infrastructure can reduce the need for future bank stabilization, and are promoted for conservation to provide corridors of riparian forests and ecosystems (Karr et al. 2000). Conservation easements may, or may not involve infrastructure relocation or setback. Similar to infrastructure relocation or setback, it may be possible to use conservation easements as an opportunity for the river to access historical flood plain areas but this should be assessed based on current and future flow patterns, not based on historical flow patterns.

Expanding the river migration corridors through land acquisitions or easements can reduce or eliminate the need for other bank stabilization or river maintenance works to protect riverside infrastructure. This can reduce infrastructure protection requirements, preserve riparian areas and allow more natural river movement (i.e., erosion and deposition). There are no direct effects upon the main channel river characteristics, which can be an advantage or disadvantage depending on the situation. Conservation easements may allow the river the greatest flexibility to adjust its hydraulic geometry and to migrate laterally as determined by river processes. Riparian lands provide ecological benefit and promote and sustain valuable areas of riparian habitat.

Disadvantages include the challenges of land acquisition and site selection, critical to the success of the project. The site may not be suitable for riparian establishment or development. The site may have a revised hydrology that limits channel formation and other river dynamics. Altering the current flood plain may introduce a temporary instability in sediment continuity until the system stabilizes under the new configuration. Habitat effects would be similar to infrastructure relocation or setback.

Conservations easements are appropriate for all rivers and are being used on many rivers in the United States, Canada, and Europe to preserve lands for habitat purposes, river migration corridors, flood plains, and a meander belt (Karr et al., 2000; Brookes, 1988). Conservation easements should promote the protection of naturally functioning native fish and aquatic communities and ecosystems (Karr et al., 2000).

6.4 Hardening Banks

The river banks of highly erosive rivers can be hardened with riprap, concrete, sheet pile, or other inert materials. Bank hardening methods can also be described as longitudinal bankline or direct methods. Hardening is applied directly to the bankline and provides protective armoring against erosion. Armoring with riprap has been a universal and commonly applied solution prior to a general understanding of geomorphic sustainability and environmental consequences. Within the category of bank hardening, riprap is recommended over other materials due to the articulating nature of the individual particles that shift to adjust to the changing form of a river bank or bed. Riprap is placed in a layer that is multiple particles thick, and can adjust to local displacement of materials scour, settlement, and surface irregularities without complete failure of the installation. This is often referred to as “self-healing” and can sustain minor damage and still continue to function without further damage (McCullah and Gray, 2005). But there are limitations on the availability and cost of large suitable rock. In cases of very high erosive forces, concrete or other hard surfaces may be required.

An investment in design time for a suitable filter, rock gradation and toe can largely increase the life of the riprap revetment and this method is very successful at presenting bank erosion. Bank hardening methods (except the deformable bankline method) can induce local channel deepening and local bank-toe scour (Brown, 1985; Niezgoda and Johnson, 2006), and can cause a short term increase in bed material load as the channel deepens (Stern and Stern, 1980). Channel deepening can cause decreased width-to-depth ratio (Niezgoda and Johnson, 2006) and generally maintains existing high-erosive energy conditions. These changes can be beneficial or detrimental depending on individual site conditions and project goals.

A negative aspect of bank hardening is the associated loss in riparian habitat. A narrow band of habitat is lost on the banks where inert materials are placed, but a wider swath of habitat is lost when the floodplain is disconnected by a deep, narrow channel that conveys flow at high velocities and does not spill out-of-bank. Prevention of overbank flow halts flood plain development and natural stream adjustments that occur in response to continuously changing conditions that impact river morphology. River processes maintain and rejuvenate a wide band of species and enrich riparian habitat in the flood plain. For channels migrating slowly this effect is generally more localized over a typical engineering design life. It is difficult to show overall benefit resulting from an extent of riprapped bank, unless environmental effectiveness is ignored and long-term maintenance costs can be substantiated.

6.5 Method Combination

Each method has different features, geomorphic response, and benefits or effects upon the channel morphology. Projects often have multiple objectives—such as stabilizing a bank, which is eroding towards riverside infrastructure, creating variable depth and velocity habitat, and expanding flood plain connectivity to reduce the energy of high flows and benefit habitat of riparian and aquatic species. Some methods provide for increased habitat value while others do not. For a net positive benefit, methods can be combined. A large number of method combinations are available for use (depending upon project needs, local habitat needs, and local site conditions). These combinations can provide multiple benefits. In general, small scale projects may involve a lower number of method combinations due to having fewer effects, while larger scale may need more methods to account for larger environmental effects. Proximity to other projects is also a factor. In reaches where multiple projects are planned, these projects should be integrated to evaluate reach scale effects and benefits to determine the method combinations which are most appropriate. Single projects may have lower reach scale effects, reducing the number of method combinations needed.

Two examples of possible methods combinations are provided to illustrate of increased benefits from such combinations. Both examples are complex and large scale, demonstrating how methods can be combined for a variety of environmental effects and habitat needs. Small scale projects with lower effects could have a lower number of methods involved.

6.5.1 Example 1: Lateral Migration

The first example is a site where the riverbed elevation has lowered, and the river has changed from a wide, low-flow, braided sand bed channel to a single thread gravel-dominated bed, which is slightly sinuous. This change has been caused largely by the combination of reduced flow peaks and upstream sediment supply. The slightly sinuous channel is migrating laterally, and the river likely will erode riverside infrastructure within a few years. In this reach, fish habitat has been degraded as the channel bed has lowered—leading to the channel becoming disconnected from the historical flood plain. Bank line habitats such as backwaters, shallow overbank flows adjacent to the main channel, cover, and variable depth and velocity flow conditions have largely disappeared. The channel bed lowering has eliminated periodic overbank flooding so that the riparian forest plant community is becoming decadent (mature trees, which are not being replenished by younger trees). After evaluating alternatives based upon geomorphic response, cost, environmental effects and benefits, social effects and acceptability, and engineering, a preferred alternative was selected. The preferred alternative consists of these features:

- Relocated river channel into an alignment away from the levee while maintaining some channel curvature (channel relocation using pilot channels or pilot cuts).
- Lowered bank line area created by placing the fill from the relocated channel excavation at a lower elevation than the historical flood plain to re-establish flood plain connectivity. Fill also could be placed with a lateral slope so that there are variable inundation levels for different river flow rates (longitudinal bank lowering).
- Bendway weirs along the outside bank of the relocated channel bend to prevent bank erosion.
- Large woody debris placed at various locations throughout the project area for fish cover. Most native riparian tree species have low durability; and, thus, the large woody debris structures constructed from these native tree species have a short project life.
- Several high-flow side channels re-established along the inside of the bend, formed by the relocated channel.
- Native riparian woody and shrub species plantings in the newly created flood plain areas.

In this example, seven methods are used for a single project to protect the riverside infrastructure, provide for flood plain connectivity, create variable velocity habitat types, and initiate establishment of a new riparian zone in the lowered bank line area.

6.5.2 Example 2: Lower Bed Elevation

The second example is a site where the riverbed elevation has lowered, and the channel has changed from a wide, braided sand bed channel to a multithread gravel dominated bed, which is slightly sinuous with flow around an island. The island is vegetated with mature woody species and is a distinct, longer term feature of the channel. The channel bed elevation lowering and channel width reduction are caused largely by the combination of reduced flow peaks and upstream sediment supply. The channel flowing around the right side (looking downstream) of the island is a slightly sinuous, laterally migrating channel. The migrating right channel likely will cause erosion of riverside infrastructures within a few years. In this reach, fish habitat has degraded as the channel has evolved to a narrow channel, which is not connected to the historical flood plain. Bank line habitats such as backwaters, shallow overbank flows adjacent to the main channel, cover, variable depth, and velocity flow conditions have largely disappeared. The channel bed lowering has eliminated periodic overbanking, so that the riparian forest plant community is becoming decadent. After evaluating alternatives based upon geomorphic response, cost, environmental effects and benefits, social effects and acceptability, and engineering, a preferred alternative was selected consisting of these features:

- Along the outside (eroding) bank line of the right channel around the island, the bank line is lowered to create a flood plain. The decreased depth and flow velocity on the outside of the bend in the lowered bank area would slow lateral and vertical erosion. Sediment excavation would be minimized by balancing cut and fill.
- Placed small-sized riprap and fabric-encapsulated soil lifts with dense willow plantings along the eroding bank of the right channel (deformable stone toe with bioengineering and bank lowering).
- Lowered bank line along the non-eroding bank of the island, to increase flood plain connectivity. Sediment excavation would be minimized by balancing cut and fill (longitudinal bank lowering).
- Placed large woody debris at several locations throughout the project area to provide fish cover and variable depth and velocity habitat.
- Planted native tree and shrub species in the lowered bank line on the left side of the channel (riparian vegetation establishment).

Example two includes five methods for a single project to protect the riverside infrastructure, provide for flood plain connectivity, create variable velocity habitat types, allow for continuation of current geomorphic processes at a slower rate (deformable bank line), and initiate establishment of a new riparian zone in the lowered bank line area.

6.6 Methods Selection

Two tables are provided to aid in the selection of an appropriate method. Table 6–1 contains a confidence rating in the method performance, advantages, disadvantages, and general range of applicability for each method of bank stabilization. Table 6–2 lists assessments of Geomorphic Response, Engineering Effectiveness and Habitat Characteristics for each method. Confidence rankings (see below) in Table 6–1 are based upon performance, reliability of design criteria, the general amount of case studies, and the expected geomorphic response. Rankings are classified into three levels using the following criteria:

- Level 1. Well established, widely used, well documented performance, reliable design criteria, numerous case studies, and a well-known local geomorphic response that is well documented for a variety of river conditions.

- Level 2. Often used but lacks the level of detail, quality of information and reliability that characterizes Level 1, little or no long-term monitoring, limited design criteria, limited knowledge about the local geomorphic response, and/or limited documentation.
- Level 3. Emerging promising technique that does not have a track record, field or lab data, or design or test data, has few literature citations, has sparse documentation and little is known about local geomorphic response, etc.

Many of the Level 2 and Level 3 methods have promise for successful implementation, but do not have a history of publicly-shared design guidelines based upon hydraulic and engineering performance.

Table 6–2 assessments contain information on potential geomorphic responses for the application of each method, an assessment of the level of documentation, and a determination of how well the geomorphic response is understood. If a geomorphic response has not been documented or the documentation could not be located, the assessment is a judgment call on the conceptual geomorphic response. Method requirements may be provided when those requirements have an impact upon geomorphic response. In general, there is little information on the upstream and downstream effects of these methods, based upon literature reviewed, beyond a relatively short distance such as one-fourth to one-half of a meander wavelength (Fischenich, 2000). Most of the available information would be best characterized as a “local response.” Determination of the geomorphic response is more difficult for rivers that are either degrading (sediment transport capacity is greater than supply and the bed is lowering) or aggrading (sediment transport capacity is less than supply and the bed is rising) than for rivers that are closer to dynamic equilibrium.

A general response for biological/ecological riparian zone and aquatic resources has been determined for each method in Table 6–2 based upon its characteristics and potential geomorphic response. Biological/ecological effects are summarized from the amount of potential lateral or vertical movement of the channel, potential ground water elevation changes, amount of low-depth and low-velocity habitat, variability of depth and velocity habitat for different river flows, flood-plain connectivity, backwaters and side channels, sediment transport, and sediment deposition areas. The biological response will be based on similar river conditions and habitat use in these situations. There are often microhabitat features that form with variable biological use. When information is available, general riparian and aquatic habitat responses to the maintenance methods are included.

The prediction of biological and geomorphic response is difficult, and is specific to the study site and river system where bank stabilization is needed. Thus, the geomorphic and biological response information provided in this document is general. As presented in earlier chapters, the watershed-, reach, and/or site-scale geomorphic evaluation is suggested previous to this step of method selection.

Table 6–1. Method Categories and Summary of Performance Confidence Rating, Advantages and Disadvantages, and General Range of Applicability

Method	Performance Confidence Rating	Advantages	Disadvantages	General Range of Applicability
Preserve the Floodplain				
Infrastructure Relocation or Setback	Level 1 (infrastructure) and Level 2 (limited post project field studies-river response).	Greater area for lateral migration. Infrastructure is protected by relocation.	Can be higher cost than other methods; lateral migration may continue to new infrastructure location with the same erosion issue as before.	Applicable to all ranges of river conditions.
Conservation Easements and Vegetation Buffer Zones	Level 2.	Provides land for river migration corridor reducing or eliminating the need for other bank stabilization work. Preserves riparian areas and allows for at least some natural lateral migration of the river channel. Provides great flexibility for the river to adjust hydraulic geometry, and lateral channel position. Can be associated with infrastructure relocation.	Few landowners may be interested in potentially changing their land use, and site selection is critical to have sites with river dynamics.	Used on many rivers in the United States, Europe, and Canada. Applicable to all rivers.
Reestablish Floodplain Areas				
Island/Bank Clearing and Destabilization	Level 3.	Promotes a wider shallower river. Provides increased flood plain connectivity; provides pockets of low depth and velocity habitat, sediment balance, and increased sediment supply.	Sediment balance may be temporary; clearing and destabilization may need to be done several times.	Platte River, Middle Rio Grande Project at Santa Ana Pueblo, Switzerland, Austria, other European Rivers where annual scouring flows can occur.
Longitudinal Bank Lowering (Compound Channel)	Level 2 (design methods available) and Level 1 (limited post project field studies).	Main channel shear stress is reduced during peak flows. There is a lower tendency for channel incision, reduces peak flow water surface elevation, can provide a small amount of downstream sediment enrichment, establishes flood plain connectivity, and promotes new riparian vegetation growth.	Future vegetation will restrict flood flows in the overbank over time, and the lowered terrace or bank may experience sediment deposition.	Used on many rivers in Europe, on incised channels in the United States, and in the State of Washington. Most applicable for channels with low suspended sediment loads.
Side Channels (perennial, High Flow, Oxbows)	Level 2 (design methods available) and Level 1 (limited post project field studies).	Inexpensive method to reconnect abandoned flood plain areas. Method decreases main channel sediment transport capacity, which could reduce channel incision, raise ground water table, and provide surface flows for developing riparian vegetation.	High-flow side channels tend to fill with sediment at entrance and exit locations. Too much flow in the side channels can lead to excessive sediment deposition in the main channel.	Applicable to a wide range of rivers where there is opportunity to reconnect flood plain areas. Most applicable for channels with low suspended sediment loads.
Bankline Embayments and Backwaters	Level 3.	Provides habitat suitable for retaining semi-buoyant eggs, and slack water areas for rearing habitat. Relatively low cost and low level of effort, which can be easily reconstructed and maintained	Can create sediment depositional zones, which reduces the effectiveness of egg and larval retention for certain fish species.	Most applicable where there is a lack of backwater habitat or channel features which produce complex eddy currents and that generate near-zero flow velocity. Best suited for channels with low sediment loads.
Design of Vegetated Banks				
Riparian Vegetation Establishment on Banks	Level 2.	Restores flood plain riparian areas.	Plantings can have a large mortality rate unless planted at the specific elevation to receive water but not too shallow to be excessively inundated.	All but the driest Southwest ephemeral rivers would benefit from riparian vegetation establishment.
Bank Line Bio-Engineering-Vegetation Only	Level 3.	Uses natural materials, assists in stabilizing banks by trapping sediment and adding root strength to the bank line. Creates additional boundary shear resistance.	Vegetation has lowest erosion resistance of all available methods. Application is limited to bank elevations above the base level flow. Does not protect against toe erosion, and most applications include toe protection in the form of rock or logs. Does not work well in sandy soil banks.	This method is not recommended as a stand-alone treatment for most banks where toe erosion is the primary mechanism of bank erosion.

Table 6–1. Method Categories and Summary of Performance Confidence Rating, Advantages and Disadvantages, and General Range of Applicability—Continued

Method	Performance Confidence Rating	Advantages	Disadvantages	General Range of Applicability
Deformable Bankline	Level 2 (riprap sizing) and Level 1 (lack of design guidelines and post project studies).	Began in 1990s. Limited field applications and documentation. The riprap design is well established. Allows bank line deformation after vegetation is established. Increases flood plain connectivity and can bring sediment transport capacity more in balance with sediment supply.	There is a risk that the stone toe design event will be exceeded before vegetation is established. The purpose of the bio-degradable fabric is to prevent erosion until vegetation is established. The method should not be used where high value infrastructure is near the eroding bank because the method depends upon a lateral migration area. The lateral migration area required is not well established.	The method has been applied to small streams with little or no reports of use on large rivers. Has been used on the Middle Rio Grande Project at the Santa Ana site. Coir fabric may have limited longevity on rivers with high bed material load.
Longitudinal Stone Toe with Bio-Engineering on Banks	Level 1 (riprap design, scour, and longitudinal extent of placement are well known) and Level 2 (elevation of the top of the stone toe and bioengineering in arid climates is less known).	Thoroughly tested and used in a wide range of conditions. Vegetation provides aesthetic benefits, shading, and reduces bank line velocity during high flows.	Decreased channel width and increased depth. Creates a local static bank line. In some cases, longitudinal stone toe can lead to accelerated bank erosion of downstream bends. In arid climates, Koir fabric or bio-D blocks are needed to provide suitable conditions for vegetation to grow, and vegetation may need to be replanted to provide the desired benefits.	Well suited to protect against toe erosion where mid and upper banks are fairly stable due to vegetation and cohesion. All types of channels throughout the U.S.
Design of Channel and Boulder Elements				
Native Material and Rootwad Revetments	Level 3	Increases bank roughness and turbulence which moves the location of high velocity flows away from the bankline. Can trap and retain sediment	Banks need to have at least 15% silt or clay otherwise bank erosion will occur around rootwads	All types of channels except with sandy banks, usually used in rivers less than 65 ft wide with slopes less than 2%.
Large Woody Debris and Rootwads	Level 2.	Can create in stream cover, pool formation, deflect flows, retain gravels, and create complex hydraulics. LWD is a natural material.	Length of benefit is usually between 5 and 15 years depending upon the durability of the available tree species.	LWD is used in many areas of the world but is not used much in the arid Southwest where tree species do not last more than about 5 years..
Engineered Log Jams	Level 2	Locally increase hydraulic roughness and divert flows away from the eroding bankline, and can trap sediment within the structure	Accumulation of additional debris and increased hydraulic roughness can cause backwater effects. Limits future channel migration.	Many channels throughout the United States especially in the Pacific Northwest, and South East where hardwoods exist. Best results are realized on rivers where large woody debris is commonly found.
Boulder Clusters	Level 2.	Adds local roughness elements, local areas of variable depth and velocity, and is simple and natural looking in many contexts.	Can often become mobile and lose the shape of the cluster. Do not provide benefits in depositional zones.	This method is used throughout North America, but bed material should be coarser than medium gravel or about 50 millimeters.
Channel Relocation				
Channel Relocation	Level 2 (construction and hydraulics) and Level 3 (limited post project field studies).	River can be relocated away from infrastructure; excavating narrow channel reduces cost and provides a small amount of sediment augmentation. Greater flood plain connectivity; meandering alignments can be in dynamic equilibrium and aesthetically pleasing.	Excavated sediments may take years to be eroded by river flows; must be enough land available for re-meandering alignment and future lateral migration; difficult to predict response with precision.	Applicable to a wide range of rivers where a meandering planform is sustainable.

Table 6–1. Method Categories and Summary of Performance Confidence Rating, Advantages and Disadvantages, and General Range of Applicability—Continued

Method	Performance Confidence Rating	Advantages	Disadvantages	General Range of Applicability
Transverse or Indirect Methods				
Transverse Features in General	Level 2 recently developed design method for vanes or barbs and spur dikes	Little or no bank preparation is needed for construction. Existing channel alignment and geometry can be modified. Geotechnical bank stability can be increased by sediment deposition between structures. Methods are widely used but are less well understood than longitudinal bankline methods. Provides variable depth and velocity habitat, can be used in combination with longitudinal stone toe.	These methods change flow alignment, channel geometry, and roughness; thus, attention must be given to morphological response. These methods can be a safety hazard to recreation because flow is redirected and part of the structure may be submerged, depending upon the method. These structures are subject to severe hydraulic conditions because flow accelerates as it passes over and around the tips of transverse features.	Transverse features have been used extensively throughout the United States in all types of rivers as noted below. One caution is that, when these structures are used in sand bed channels, scour often undermines the riprap, leading to failure. They have been used to add habitat to longitudinal stone toe.
Bendway Weirs	Level 2 (limited design guidelines available) and Level 3 (lack of quantitative design guidelines and post-project studies).	Flows are redirected throughout the flow field. The outer bank toe can become a zone of low velocity and a zone of sediment deposition. Aquatic habitat is improved because bendway weirs create variable depth and velocity habitat.	Weir fields must have sufficient spacing to protect the banks and weir roots so that if bank scalloping occurs, the weirs riverside infrastructure remains protected. Regular monitoring and maintenance are required. Velocity over the weirs along the bank accelerates which can cause a bank shelf to develop in addition to scalloping	Large rivers, such as the Mississippi, to small streams have documented use of bendway weirs to deepen and widen the thalweg for shipping. Bendway weirs also are applied to protect highway bridge crossings on braided or meandering rivers in many States in the United States. Most suitable where the flow entrance angle is expected to remain fairly stable.
Vanes or Barbs	Level 2 recently developed design criteria.	Reduces streambank erosion, modifies flow direction, creates local scour, and gains environmental benefits. Vegetation can grow on sediment deposits between vanes where sufficient supply exists for sediment to deposit between vanes. Vanes generally require less rock than other structures for a similar length of bank line.	The low volume of rock near the tip of the vane often launches into the scour hole, requiring regular maintenance. Bank scalloping between vanes is common and can lead to vane failure. Long-term bank protection is usually only achieved when sediment deposition occurs between vanes.	Suggest for use in channels that have a width-to-depth ratio of 12 or greater. Vanes have been used extensively throughout the United States. Can be positioned in the channel to initiate meander development or migration for habitat purposes.
Spur Dikes	Level 2, recently developed design criteria.	Spur dikes modify channel alignment and provide erosion protection for riverside structures. Provides variable velocity and depth habitat. Can induce sediment deposition.	The bank line between spur dikes can erode when the spur dike spacing is too large. Over time, the channel deepens, increasing flow capacity. Local channel narrowing can occur. The extent of channel deepening and narrowing cannot be predicted with great reliability. The bank line is fixed, thus interrupting fluvial processes.	Most commonly used in shallow, wide streams with moderate to high suspended sediment load. Spur dikes are used widely for protecting highway bridge crossings in the United States.
J-Hooks	Level 2 recently developed design criteria and Level 3 (J-Hook, does not have a documentable track record.)	Same as vanes with a “J” hook added. The “J” tip creates a scour pool in the channel bed, which increases the amount of pool habitat. The rest of the vane provides variable depth and velocity habitat.	“J” hook at the center of the channel is subject to scour erosion. This structure requires more riprap and more in channel construction than vanes. The “J” tip can fill with sediment in sand and fine gravel bedded channels. The remainder of the disadvantages is the same as for vanes.	Same as vanes.

Table 6–1. Method Categories and Summary of Performance Confidence Rating, Advantages and Disadvantages, and General Range of Applicability—Continued

Method	Performance Confidence Rating	Advantages	Disadvantages	General Range of Applicability
Hardened Banks				
Riprap Banks in General	Level 1	In general, these methods are widely tested and used, while deformable bank lines are less well understood.	Can cause the channel width to decrease, creates a static bank line, and can in some, but not all, cases lead to acceleration of bank erosion in downstream bends.	Generally applicable to all types of channels.
Longitudinal Peak Stone Toe (LPST)	Level 1	Bank grading is usually not needed limiting disturbance to existing bank, relatively easy to construct, and can be combined with bio-engineering methods to provide habitat.	Does not protect mid and upper bank where some erosion would likely occur during high flows. Not suitable for reaches with rapid bed degradation or where scour depths can be higher than the height of the LPST. The height of the LPST is not well established.	Tend to be used where the channel is overly wide such as when incision has occurred to the point where the banks have collapsed and the channel widened. Used in many channel in Mississippi, Las Vegas Wash, and rivers in California
Riprap Revetment	Level 1.	Thoroughly tested and used for a wide range of conditions and can be designed with a high degree of precision and confidence. Provides maximum protection for riverside infrastructure.	Decreased channel width and increased depth. Creates a local static bank line. In some cases, riprap revetments can lead to accelerated bank erosion of downstream bends.	Well suited for toe and fluvial bank erosion. Not well suited to address soil mechanics bank failure. Used on virtually all types of rivers found in the North America and Europe.
Riprap Windrow and Trench Filled Riprap	Level 2.	Allows stabilization along a predetermined alignment. Generally effective for controlling lateral channel instability. Work is away from the bankline until supplemental riprap is provided, reducing in water work.	Requires large areas of right-of-way. Self-launching riprap does not distribute evenly along the bank line. Requires supplemental riprap placement to ensure even distribution and revetment stability. Windrow requires more supplemental riprap than trench filled riprap, because the launch distance is greater. More susceptible to continued bank erosion due to uneven launching than trench filled. Creates a static bank line.	Used on Lower Colorado, Arkansas, Red, Missouri, and Mississippi Rivers and is most suitable for noncohesive banks and where emergency sites exist.

Table 6–2. Summary of Geomorphic Response, Engineering Effectiveness, and Habitat Characteristics

Method	Geomorphic Response	Engineering Effectiveness	Habitat Characteristics
Preserve the Floodplain			
Infrastructure Relocation or Setback	Can encourage current geomorphic processes to continue, such as lateral migration and the creation of new flood plain and riparian areas. Opportunity to connect to historical channels and oxbows. For incised channels, may provide an opportunity to establish new inset flood plain and riparian zone.	Effectively protects riverside infrastructure by moving it from the erosion zone. Level of confidence is medium to high.	Lateral river movement creates broader flood plain and more favorable riparian zone habitat. Lateral bank movement should result in deposition of sediment downstream. The river will establish bars and low surfaces, where vegetation can become established. Longer meander bends may establish greater pool depth and eroding banks with vegetation falling into the channel, providing fish cover and habitat complexity.
Conservation Ease-ments and Vegetation Buffer Zones	Allows space for existing fluvial processes to continue, which can preserve flood plain connectivity.	Level of confidence is high and depends on the amount of setback. Often, this is done in conjunction with riverside facilities and structures.	Allows more natural river movement and promotes greater area of undisturbed habitat.
Re-Establish Floodplain Areas			
Island/Bank Clearing and Destabilization	Promotes a wider channel with greater flood plain connectivity and balances sediment. New sediment balance may be temporary unless incoming loads also increase.	Can provide for increased flood carrying capacity. Durability and project life depend upon the elevation of cleared surfaces and frequency of scouring flows and sediment deposition. Project life span could be many years or may be short lived. Level of confidence is low because there are not many examples of using this method.	Reduces further degradation of the channel and lowering of the water table. Sediments from destabilized areas may deposit new bars suitable for vegetation. Clearing and destabilization would result in the loss of this habitat. Islands/bars that are more connected to the main channel can provide greater variety of depth and velocity habitat types.
Longitudinal Bank Lowering (Compound Channel)	Lowered bank line can promote a wider channel width and decreases in main channel velocity, depth, shear stress, and sediment transport capacity. During subsequent years, sediment may deposit in the lowered bank line area occupied by vegetation, which may reduce overbank conveyance capacity.	Increased flood carrying capacity. If sediment transport is in balance with capacity project durability, design life and project life may extend several decades. Level of confidence is medium.	Promotes overbank flooding favorable for establishment of riparian vegetation. Reduces potential for channel degradation, thereby maintaining a higher water table and more connectivity with backwaters and side channels. Increases overbank flooding, creating variable depth and velocity habitat types, including potential spring runoff nursery habitat.
Side Channels (perennial, High Flow, Oxbows)	Important to natural systems for passage of peak flows. Sediment tends to fill in high-flow side channels over time. Can decrease peak flow water surface elevation and may decrease sediment transport capacity until sediment blocks the side channel.	Method provides for reduced main channel sediment transport capacity. Durability and reliability depend upon the size of the side channel and amount and timing of sediment deposition in side channel inlets and outlets. Maintenance could include periodic sediment removal. Level of confidence is medium.	Side channels result in raising the ground water table and surface flows to developing riparian areas. Maintains higher water surface elevation and ground water table, adding to the health of the riparian zone. Can reconnect the flood plain to the channel, creating nursery and variable depth and velocity habitats.
Bankline Embayments and Backwaters	Slow water velocity and shallow depth bank line habitat is restored/rehabilitated, in cases where this type of habitat existed in the past.	Level of confidence is low. Bank line embayments are zones of sedi-ment deposition and have a finite lifespan without periodic re-excavation. With continual maintenance of sediment deposition, long term fisheries benefits can be provided. Can provide areas for new tree growth.	Provides vital high-flow egg retention and nursery larval habitat that has largely been lost on many rivers with reduced flood peaks and sediment supply. Increases likelihood of native riparian vegetation growth.
Design of Vegetated Banks			
HE Riparian Vegetation Establishment	Can cause sediment deposition in overbank areas due to increased flow resistance. Sediment deposition in the overbank can increase main channel sediment transport capacity by raising the bank height.	Level of confidence is medium. Planting elevation and ensuing hydrology must provide appropriate conditions for plant growth.	Directly adds to the amount of riparian vegetation.
Bank Line Bio-Engineering-Vegetation Only	Vegetation has the lowest erosion resistance of all available methods. Plantings require time to establish before any bank protection is realized. Lateral and down valley bank line movement can continue because bioengineering does not permanently fix the bank location.	Level of confidence is very low. Plant roots do not prevent bank erosion below the base flow level (toe erosion). This is especially true in sandy bank material. For banks with toe erosion, bioengineering is not recommended as a standalone treatment. Generally, bioengineering includes toe erosion protection.	If the technique is successful, it could promote the establishment and development of riparian vegetation without significant armament to the bank line. Allows more natural movement of river channel.

Table 6–2. Summary of Geomorphic Response, Engineering Effectiveness, and Habitat Characteristics—Continued

Method	Geomorphic Response	Engineering Effectiveness	Habitat Characteristics
Deformable Bankline	The design is intended to allow lateral migration at a slower rate than is occurring, which leads to the need for maintenance by establishing a new vegetated flood plain that is erodible. Water surface elevations could be lower with bank lowering. After installation, and before the toe of the riprap becomes mobile, the channel bed may scour along the deformable bank line. Bank erosion occurs during peak flow events, which mobilizes the small sized riprap along the bank toe.	Level of confidence is medium. Lifespan of the biodegradable fabric is generally 3–5 years. Method depends on adequate vegetation growth in this time period. Level of durability and reliability can be great when there is sufficient land available for lateral migration.	If flood plain is created behind the stone toe and vegetation becomes established before the toe is lost, an expanded riparian area could develop. Future bank migration would allow new depositional surfaces to establish, which would become new riparian areas.
Longitudinal Stone Toe with Bio-Engineering	Stops local bank erosion; causes local scour and channel deepening. Studies about longer reach response are contradictory. Can be susceptible to flanking if upstream channel migration occurs.	Durable, high level of confidence in method provided that the elevation of the top of the riprap stone toe is adequately established to provide complete toe protection.	Prevents lateral migration and the establishment of new depositional zones where vegetation could become established. Reduces local sediment supplied from bank erosion. The steep bank angle on the outside of the bend limits fish cover, except for the riprap interstitial spaces. The point bar remains connected to the main channel and remains static. The flow velocity and depth are greater than typically found in natural channels along the outside bank of a river bend. Bio-Engineering provides shading and minimal benefits to riparian community.
Design of Wood and Boulder Elements			
Large Woody Debris and Rootwads	Creates pools, generates scour and substrate sorting, and increases depth and velocity complexity. Can promote side channel formation and maintenance. Can lead to sediment deposition, including formation of islands, in rivers with large sand loads.	Level of confidence is medium. Some design guidelines are available. Short design life span for some southwestern U.S. tree species such as cottonwood.	Adds complexity to the system. Sediment deposition can create areas where new riparian vegetation becomes established. Can create variable depth and velocity habitat. Reliability for providing fish habitat is high for a while logs/rootwads remain intact (5-25 years). Can provide structure and habitat for fish. Can provide low-flow refugia habitat during low-flow periods.
Native Material and Rootwad Revetments	Increases bank roughness and turbulence creating local bed lowering and scour. Reduces	Level of confidence is medium. Life span of native materials generally ranges from 5-15 years.	Adds complexity to the system. Creates cover , structure and habitat for fish.
Engineered Log Jams	Engineered log jams reduce flow energy, stabilize the bank, create local scour and narrow the stream. The thalweg generally shifts to the tip of log jams when used along the bankline. Can lead to sediment deposition and potentially bankline scalloping.	Level of confidence is medium. Can reduce or halt local bank erosion. A sufficient riparian buffer zone between the actively eroding bank line and important riverside features should exist because of the potential scallop formation between jams.	Sediment deposition between structures may allow establishment of riparian vegetation and backwater areas. Channel deepening and scour could locally lower the riverbed. Provides bank cover, habitat diversity and variable depth and velocity habitat.
Boulder Clusters	Creates a zone of local scour immediately downstream from the boulders. Creates variable depth and velocity habitat. Creates velocity shear zones. Effects are localized to the immediate vicinity of the boulders. Increases channel roughness at high flows.	Level of confidence is medium. Cost is low. Boulders can migrate into the downstream scour hole.	Can provide structure and variable velocity and depth habitat for fish.

Table 6–2. Summary of Geomorphic Response, Engineering Effectiveness, and Habitat Characteristics—Continued

Method	Geomorphic Response	Engineering Effectiveness	Habitat Characteristics
Channel Relocation			
Channel Construction	Can bring sediment transport capacity more in balance with sediment supply in supply-limited reaches. Re-establishes meanders, increases channel stability, and initiates new areas of bank erosion and deposition.	Effective for protecting riverside infrastructure by moving channel away from infrastructure. Level of confidence is medium to high.	Can provide overbank flooding and establish new areas of riparian vegetation. Can increase the complexity of habitat by creating connected flood plain/wetted areas for fish habitat.
Transverse Features			
Transverse Features in General	Flow is deflected away from the bank line, thereby altering secondary currents and flow fields in the bend. These methods may cause local sediment deposition between structures and/or local scalloping along the bank line. Eddies, increased turbulence, and velocity shear zones are created. Methods induce local channel deepening at the tip. Shear stress increases in the center of the channel, which maintains sediment transport and flow capacity.	Level of confidence is medium. Maintenance often required to provide rock at the tips.	Sediment deposition between structures may allow establishment of riparian vegetation and backwater areas. Channel deepening and tip scour could locally lower the riverbed. Depending on site-specific conditions, transverse features could allow for overbank flooding conditions improving the health of the riparian zone. Local scour could provide habitat diversity and deep habitat during low-flow conditions.
Bendway Weirs	The location of the thalweg is shifted away from the outer bank line. Local scour at the tip occurs because of the three-dimensional flow patterns. Secondary currents are interrupted, and flows are redirected away from the bank along the bank toe. Flow accelerates over the bendway weir crest which can lead to outer bank erosion.	Level of confidence ranges from low to medium. Can reduce local bank toe erosion, can cause upper bank erosion. Bendway weirs can erode away in gravel and sand bed channels due to downstream scour. Regular maintenance in the form of adding rock to the structures is necessary. The entrance angle must remain the same over time for continued redirection of the flow patterns. Durability and reliability are low to medium. Improved design criteria and methods are being developed at Colorado State University (CSU), which is important to improve the level of confidence.	Same as transverse features or flow deflection techniques described above.
Vanes or Barbs	These structures redirect flow from the bank toward the channel center and reduce local bank erosion while providing a downstream scour hole. Sediment deposition or bank scalloping can occur along the outer bank, depending upon spacing.	Level of confidence is medium; local bank erosion is reduced. The tip is relatively thin, owing to the sloping top. Tip stones often roll into the downstream scour hole, requiring replacement on a regular basis. A sufficient riparian buffer zone between the actively eroding bank line and the structure being protected should exist because of the potential scallop formation between vanes.	Same as transverse features or flow deflection techniques described above.
Spur Dikes	Spur dikes block the flow up to bank height, thus shifting the thalweg alignment to dike tips. Peak flow capacity can be reduced initially until the channel adjusts. The channel adjusts to the presence of spur dikes by forming a deeper, narrower cross section with additional scour downstream from each spur dike. Sediment deposition can occur between spur dikes.	Level of confidence is medium. Can halt local bank erosion. Spur dikes are more durable than bendway weirs and can remain functional if there are small changes to the upstream entrance conditions. Future maintenance (adding riprap on the spur dike tips) may be required, especially in gravel and sand bed streams.	Same as transverse features or flow deflection techniques above. There is a greater tendency for sediment deposition between spur dikes than the other transverse features.
J-Hooks	Redirects flow away from eroding banks the same as vanes or barbs with an added downstream pointing "J" configuration. The J-hook creates an additional scour hole pool and can produce a local downstream riffle. Remainder of the geomorphic response is the same as for vanes.	Level of confidence is low to medium. Engineering effectiveness is largely the same as for vanes, except the J-hooks may require the replacement of more rocks due to shape of the J-hook.	Same as transverse features or flow deflection techniques described above. Additional pool habitat is created by the J-hook.
Hardened Banks			
Riprap Revetment	Same as Longitudinal Stone Toe with Bio-Engineering	Durable, high level of confidence in method. Provides long-term bank protection	Same as longitudinal stone toe except without minimal benefits to riparian community (no bio-engineering)

Table 6–2. Summary of Geomorphic Response, Engineering Effectiveness, and Habitat Characteristics—Continued

Method	Geomorphic Response	Engineering Effectiveness	Habitat Characteristics
Longitudinal Peak Stone Toe (LPST)	Stops continued channel migration. Mid and upper bank sections may still erode during high flows which over top the LPST. Local bed lowering and scour.	Durable, medium to high level of confidence in method provided the elevation of the LPST is sufficient to reduce mid and upper bank erosion and provide scour protection.	Prevents lateral migration and the establishment of new depositional zones where vegetation could become established. Vegetation can become established between the bankline and the LPST where sediment deposition occurs during high flows either from suspended sediment or mid and upper bank sloughing. Toe velocity and depth is typically higher than found in natural channels.
Riprap Windrow and Trench Filled Riprap	Bank erosion processes continue until erosion reaches the location of the trench. After launching, response is the same as for riprap revetment.	Riprap placed in a trench can be placed below the high-flow water surface elevation, so this method is more durable than riprap windrow. High level of confidence, durability, and reliability provided that additional riprap is placed in gaps or where riprap is thin after launching.	Same as longitudinal stone toe except without minimal benefits to riparian community (no bio-engineering)

PART II – DESIGN AND CONSTRUCTION

Materials to Avoid

There should be no methods or figures in this manual that advocate the use of materials listed below. Materials on this list can induce failure regardless of how well the installation is designed or installed, or, may be listed due to negative environmental impacts. Subsequently they should not be incorporated into most projects.

- Geotextile in river banks (use granular filters)
- Cables to hold down materials on banks
- Coir logs on banks with any flow current
- Concrete blocks in or near channels
- Angular or crushed channel lining (all material should be rounded, but this does not apply to riprap bank stabilization)
- Car bodies, tires, crushed concrete, cabled crushed concrete and other waste material

7 Preserving the Floodplain

Relocating infrastructure or promoting riparian buffer zones provides long-term benefits to Reclamation on multiple levels of consideration. Riparian zones are significant due to their role in soil conservation, water quality, and temperature regimes; their characteristic of high habitat biodiversity; and the influence of riparian zones on fauna and aquatic ecosystems. The riparian zone in the floodplain serves as a bio-filter to protect the aquatic environment, helps regulate sediment continuity, is a source of food and shelter for aquatic animals, is influential in defining the water temperature regime, and of primary concern to the engineer, floodplain areas and associated riparian habitat help to dissipate stream energy during periods of high flow. Unaffected floodplains are self-regulating and therefore self-sustaining, minimizing project long-term maintenance costs. One additional benefit is the insulation gained from costly and time consumptive environmental issues when the floodplain and associated habitat are preserved.

Infrastructure relocations or setbacks that remove constrictions and the development of riparian conservation easements or buffers are both means of preserving and/or enhancing an adequate floodplain. The design of these methods can be aided by the inclusion of land-acquisition expertise on the project team. A disadvantage of these methods may be the increased demand on planning time and possibly costs for developing land easements or acquisitions. Advantages of these methods are often the reduced cost for maintenance if long-term evaluations are correctly used to assess the alternatives; and increased opportunities for partnering, since public and private agencies, nonprofit organizations, and community groups often share similar interests in the goal of floodplain preservation.

7.1 Infrastructure Relocations

Relocating riverside facilities requires adequate land and the practical geographic location and alignment of levee or other infrastructure relocation projects will often be dependent on local land use, land ownership and availability, economics, political preferences and constraints, and other physical constraints (Brookes, 1996). Relocating infrastructure can also be based upon site-specific flood plain and habitat objectives. For example, aligning the levee to preserve existing trees, shrubs, and riparian zones would preserve current habitat. Channel relocations are more successful when all geomorphic factors are considered. Refer to the Channel Relocation chapter for a detailed discussion of channel relocation design.

The width and location of relocated levees should be geomorphically based upon the river corridor, meander belt, or the braidplain width. Long-term considerations include estimating the length of the dynamic equilibrium channel and the river form and pattern based on future conditions. The river form is the shape of the river at a cross section and the river pattern is the shape when it is considered in airplane or plan view. Both meander and braided channel alignments tend to change more frequently when the channel has more energy (steeper slope and large flows), an abundant sediment supply, and a wide range of flows. An assessment of the meander belt or floodplain for the river can help determine the optimum geographic location and the degree of safety that is afforded by the project. If the relocated infrastructure remains within the active river corridor or meander belt width, bank protection measures should be investigated for estimated future conditions to protect against lateral migration for the life of the project. Consider the amount and frequency of overbank flooding and flood plain connectivity when evaluating project benefits.

Maintaining the existing function of levees, canals, drains, culverts, siphons, utilities, and riverside roads will usually be a requirement to provide the users with the same level of service realized prior to relocation. Relocated levee requirements include an adequate top width and stable side slopes, height, seepage protection, and a stable foundation. Selection of a spoil levee or an engineered levee will be dependent on the project site conditions and needs. Access to the levee and drain will need to continue for routine future maintenance activities after levee relocation.

7.2 Conservation Easements

Creative partnering can provide a riverine corridor for river migration and riparian zone rejuvenation and habitat cycling. The need for future bank stabilization is reduced as a result of a buffer zone between active river channel and flood control or other infrastructure. Risk of flooding and damage to riverside infrastructure is low due to providing physical space for river processes to occur. Reclamation lands are less likely to be fully developed in contrast to many eastern US locations and there is a cost advantage to introducing these projects now, rather than wait for the next round of maintenance attention. Land acquisition and/or relocation of infrastructure may appear costly at first glance but should be evaluated with realistic estimates of long-term costs.

Protection measures through conservation easements are all too often put into place after a site or species is seriously endangered, so protection measures are taken on an emergency basis (Karr et al., 2000). Protection is best served if accomplished in a proactive manner. Ideally, conservation easements would be large enough to preserve a full complement of native species and geomorphic processes (Karr et al., 2000). Identify areas or reaches where the most benefit would be realized by conservation easements.

Partnering can occur with private owners of riparian areas adjacent to the river that would not require land use change, or at locations where there are willing landowners regardless of the current land use. Consideration of preserving or restoring a floodplain should be approached with an open and creative outlook on the land inclusion that can occur. A creative approach and a land acquisition specialist can greatly aid the project team in pursuing mutually beneficial partnerships and teaming opportunities including land leases or easements. Private landowners, nonprofit organizations, businesses, large corporations, local and state entities, and other government agencies may all be potential partners, willing to team to reach goals compatible with protecting and/or expanding riparian corridors.

7.3 Design Procedure

Project design criteria, hydrology, geomorphic factors and general hydraulic and scour factors as described in previous chapters should already be complete. Important aspects of design criteria, hydrology, and permitting are described in Chapter 2. The designer should determine the geomorphic factors including sediment and sediment continuity issues that may impact channel stability as discussed in Chapter 3. The designer should also define general hydraulics including energy (Chapter 4) and potential for scour (Chapter 5) that may influence the channel and the extent of the floodplain. Some of these investigations may be revisited to obtain more detailed information for specific areas of this design. Steps of the design are outlined below.

Step 1: Determine the current floodplain. Careful evaluation of channel morphology is essential to the success of an infrastructure relocation or setback. Acquire historical photos of the project reach including upstream and downstream connecting reaches. Also map existing hard points in the study reach including riprap revetments that hold the bank and prevent channel migration. Hard points can make the prediction of channel change more difficult since the customary bend migration is altered to less predictable and patterned breakout avulsions.

Floodplain mapping from the Federal Emergency Management Agency (FEMA) will provide an estimate of the existing floodplain but is usually prepared for large areas so the available product may not be sufficiently detailed for the project site. Instead the floodplain can be defined using a one-dimensional or two-dimensional flow, or flow and sediment numerical model. These models could include Reclamation’s SRH-1D (Huang and Greimann, 2013) or SRH-2D (Lai, 2008), or the HEC-RAS model. The floodplain can be defined by altering the cross sections to represent future changes to the topography. Although they provide estimates of the future floodplain, even the 2D flow and sediment model cannot predict channel meander migration, i.e. future changes to the channel alignment, and the impacts of these changes to the future floodplain.

Step 2: Estimate the future floodplain. Estimate the future lateral migration zone given any proposed changes. Infrastructure relocated outside of the future lateral migration zone has the greatest opportunity for long term sustainability. The original design function of infrastructure in the floodplain should, at a minimum, be maintained in the relocation. For example, the relocation design for a levee that originally conveyed a 100-year peak flow with 3 ft of freeboard should be maintained in the proposed project.

The bends of meandering rivers, even in stable systems, can have a tendency to migrate upstream or downstream. Current evaluation methods will give an indication of meander migration potential, but there is not a high degree of confidence in accurately predicting future channel locations. A project has more promise for success if the riparian corridor is assigned for the full extent of the project area and for the full floodplain width.

Methods for estimating the future river corridor width are found in Shields (1996), Ward et al. (2002), and Rapp and Abbe (2003). A method utilizing aerial photography is described by Lagasse et al. (2003a and 2003b), and Lagasse et al. (2004) provides a method for predicting future channel migration. Potential migration zones, an estimate of the future changes in alignment, can also be developed from a computer simulation.

Step 3: Determine land availability, lateral extent and upstream and downstream potential transition zones, and identify potential partners. Conservation easements, leases or purchase may be required to provide land for infrastructure relocation or to expand the connectivity of floodplain. Land for the project should extend upstream and downstream of the project site to provide for transitioning relocated infrastructure and to accommodate associated construction access and staging. An estimate of future river form and location (Step 2) will also aid in determining project limits. Cramer (2012, p 5.3, T5-9) offers a means of determining riparian lands and selecting bio-engineering/vegetation techniques. Identify stakeholders and potential partners who may be interested in land exchanges, leases or easements, maintenance agreements and/or who have an equal interest in advancing the project. Sites where there will be future channel dynamics provide the opportunity for establishing new riparian forest communities and may offer more environmental benefit. These areas include low-lying surfaces that provide aquatic edge habitat and variable depth and velocity habitat during high flows.

Steps 1, 2 and 3 are normally carried out in the planning phases for the project since land easements, leases and purchases can require multiple years to conclude.

Step 4: Design the relocated infrastructure. If there are no additional project design requirements defined in the design criteria (chapter 2), a riverside or irrigation return flow drain, levees, or a water supply canal should be designed to the original capacity. In general, the size, shape, and capacity of relocated infrastructure remains as was originally designed or constructed. Levee setbacks or partial removals need to be considered. Requirements for roads, bridges, traffic control and signing will depend on the ownership and funding, and may require updates to current standards. Design requirements for relocated canals, levee's, pipelines, and underground or overhead cables or lines etc., will depend on the owners and/or operators, and will depend on the agreement reached between the parties. Contact owners/operators for permission to relocate their facility. In some cases, utilities and other owners may prefer to design and/or construct changes to their infrastructure for a fee. This work will require scheduling and integration into the general construction plan.

7.4 Discussion and Recommendations

Risk and Failure. There is always a risk that future river conditions could cause unanticipated changes to the river alignment making the estimated lateral migration zones insufficient. In general though, erosion prevention in an expanded floodplain is less costly to provide than bank protection at constricted locations.

Project Life. The project life can depend on whether the relocation is outside the meander belt or braidplain width. The project life may be shorter when the relocation is within the meander belt width or braidplain, depending upon the rate of future lateral migration.

Constructability Issues. Potential constructability issues include construction timing to avoid potential peak flow periods and the amount of material or infrastructure to be relocated. Generally, water quality permitting needs and requirements would be minimal for a relocation project when there is no construction planned within the river channel.

Cost Estimates. Costs for relocating infrastructure are highly variable, depending on the length, lateral distance of relocation, land acquisition (if necessary), and structure type. Typical costs could range from several hundred thousand dollars to several million dollars per mile. Initial costs as well as maintenance costs should be estimated based on design requirements and local conditions, and partnerships and teaming can reduce both construction and maintenance costs. Ongoing maintenance costs will depend on the type of relocated facility, but in general, a relocated facility should not require future bank protection.
Natural river processes can continue as long as the future lateral migration does not once again encroach upon riverside infrastructure. Most successful infrastructure relocation projects have levees which are set back a sufficient distance for river migration.

Monitoring and Maintenance. Regular inspections are suggested, preferably after less frequent, high-flow events, to assess river migration including the channel alignment, channel changes and river access. Monitoring could consist of visual observation and/or analysis of aerial photographs at river location, and analysis of movement within the conservation area. Key items to observe would be lateral migration which could lead to a need for future bank stabilization, and habitat cycling, the evolution of floodplain and habitat through successive changes. Bend migration can be a cyclic occurrence that regenerates riparian habitat. Maintenance needs may also be assessed if relocated infrastructure remains in the river channel migration zone; however, in general, no maintenance is required for this bank stabilization method.

8 Re-establishing Floodplain

In developed areas where it is not possible to recover or protect existing flood plain areas, one alternative is to develop or add elements that re-establish a portion of the previous floodplain. It may not be effective to re-establish the historical floodplain when the annual flow in the system has been reduced; however, reconnecting a portion or feature of the floodplain may be a feasible means of reducing the erosive pressure at the project site. Altering channel and flood plain shape can re-establish floodplain including island/bank clearing and destabilization, and lowering the river banks to allow overbank flow. Adding side channels and embayments can also re-establish floodplain area to take pressure off of narrowed channels that are subject to high erosive forces. Data needs for the design of floodplain measures include (WDFW 2004, update from WDFW Cramer, 2012):

- Current fish usage
- Topography and cross-sections
- Hydraulic profiles
- Profile and representative cross-sections of channel alignments
- Information on floodplain roughness and woody vegetation
- Vertical and lateral stability of mainstem; also look for evidence of a channel that has already degraded and left potential side channels perched
- Rating curves of water levels near upstream and downstream ends of project
- Water level rating curves of mainstem near upstream and downstream ends of project
- Site constraints and project limits
- Baseline monitoring data, which may include photo documentation of the site
- Elevation reference points should be set at least at three locations near the channel
- Description of existing fish and wildlife within side channel
- Sediment assessment
- Instream and riparian sources of mainstem and side channel
- Stability of side channel based on increased flows

A good understanding and identification of river form, sediment transport and pertinent geomorphic processes are integral to project success. A geomorphologist or river engineer should be included on the project team to help improve the sustainability of the project. The following examples illustrate how a project can go wrong if geomorphic considerations are ignored.

- Over-widening a reach of river can interrupt the sediment transport balance causing deposition and instability while a new stable channel form develops.
- Constructed side channels and embayments can silt in prematurely and side channels can also capture main channel flows leading to an avulsion that reroutes the river if geomorphic factors are ignored.
- When a river is teetering near a threshold in channel form, the addition of side channels can reduce the ability of a river to braid and transport sediment, pushing it closer to a meander system.
- Forcing a braided river to meander, or a meandering river to braid by adding or removing channel elements can be ineffectual and create a costly maintenance effort.

8.1 Island/Bank Clearing and Destabilization

The method of destabilizing islands and banks is suited to channels experiencing incision, flood plain disconnection, channel narrowing, and island formation, and specifically channels that previously had a more generous sediment supply that maintained a braided or multi-channel form of river. Under current conditions, peak flows and/or sediment supplies may have been reduced and the channel is now evolving towards a meandering river form. Incision is reducing the floodplain connection. Braided rivers require a steeper slope to maintain channel pattern and the high energy of a braided river is balanced by the transport of sediment that often saltates along the bed of a wide shallow channel (large width-to-depth ratio). If less sediment is available during peak flow events, system energy is expended in incising the bed to a flatter grade that causes a shift towards a meander form (narrow, deep channel), and more significantly, causes floodplain disconnection. Clearing and destabilization of islands and river banks is a method of preserving the floodplain element.

In braided and complex channels where flow regimes have been altered, vegetation encroachment can stabilize banks and bars and cut off the sediment supply from these features. Vegetation can also eliminate the open sand terrestrial habitat or variable depth and velocity habitat for aquatic species. If peak flows do not occur regularly, plants can attain a structure that makes the plant and the sand bar difficult to erode. Peak flows should occur every second or third year before plants have a chance to mature, stabilize and transform bars to islands, and trap sediment that raises the bar/island height and supports additional vegetation. If peak flows do not occur, sediment that was historically mobilized at high flows, now is stored on bars/islands, flood plains, and abandoned terraces. When peak flows are frequent, inundation and erosion control vegetation and the sediment supply from mobile bedforms (bars that have not stabilized to islands) and banks remain accessible for transport during peak flows. Clearing and destabilizing islands can be a means to restore the balance of vegetation limited by flow.

Island/bank clearing and destabilization is not needed annually or for every braided and anabranched river, but may be necessary if the discharge of the river has been reduced and it is crossing or dropping below the threshold for maintaining a braided river or a complex channel. A decrease in discharge reduces the energy of the river and the sediment load that can be transported. With a decrease in discharge, the river can shift from a braided form to a meandering form, or shift from an anabranched river with multiple sand bars, to a single channel meander river. A sediment balance can be sustained by removing vegetation to free up sediment supplies and by recurring and sufficient peak flow events in a managed system. These actions improve flood plain connectivity, reduce island area, promote channel widening, and sustain the dynamic bar building environment suitable as habitat to some fish and avian species (silvery minnow, pallid sturgeon, terns, plovers, sand hill cranes, whooping crane). Greater wetted area during high flows creates opportunities for egg entrainment and larval habitat for species like the silvery minnow, and spawning and rearing habitat for fisheries including the pallid sturgeon, adapted to braided river or complex channel environments.

Removal of vegetation and destabilization of bars/islands can also help promote the natural process of channel widening if there is a sufficient flow and sediment regime. Widening channels through excavation accelerates the natural process towards a wider and more shallow (>W/d) channel, but there are limits to the width that can be sustained. Over-widened fringes of the channel can quickly become colonized by new vegetation, narrowing the channel to a width defined by the most recent flow regime. A minimum width is maintainable by flows occurring annually, or occurring every other year, and also depends on a suitable sediment supply. Larger peak flows or mechanical destabilization may be needed to re-establish wide channels after multiple dry years. Widened channels create low-depth, low-velocity habitat (Figure 8–1) if the channel has been designed to a reasonable and sustainable width, consistent with the river flow regime.

If invasive plants are moving into the project area, this method may accelerate the spread rate and colonization by invasive plants. During a disturbance regime many invasive riparian plants can outperform native plants, and the destabilization of islands and banks imposes a large disturbance. Alternative actions to control the invasive plants may be necessary, especially if the invasive plant can stabilize banks.

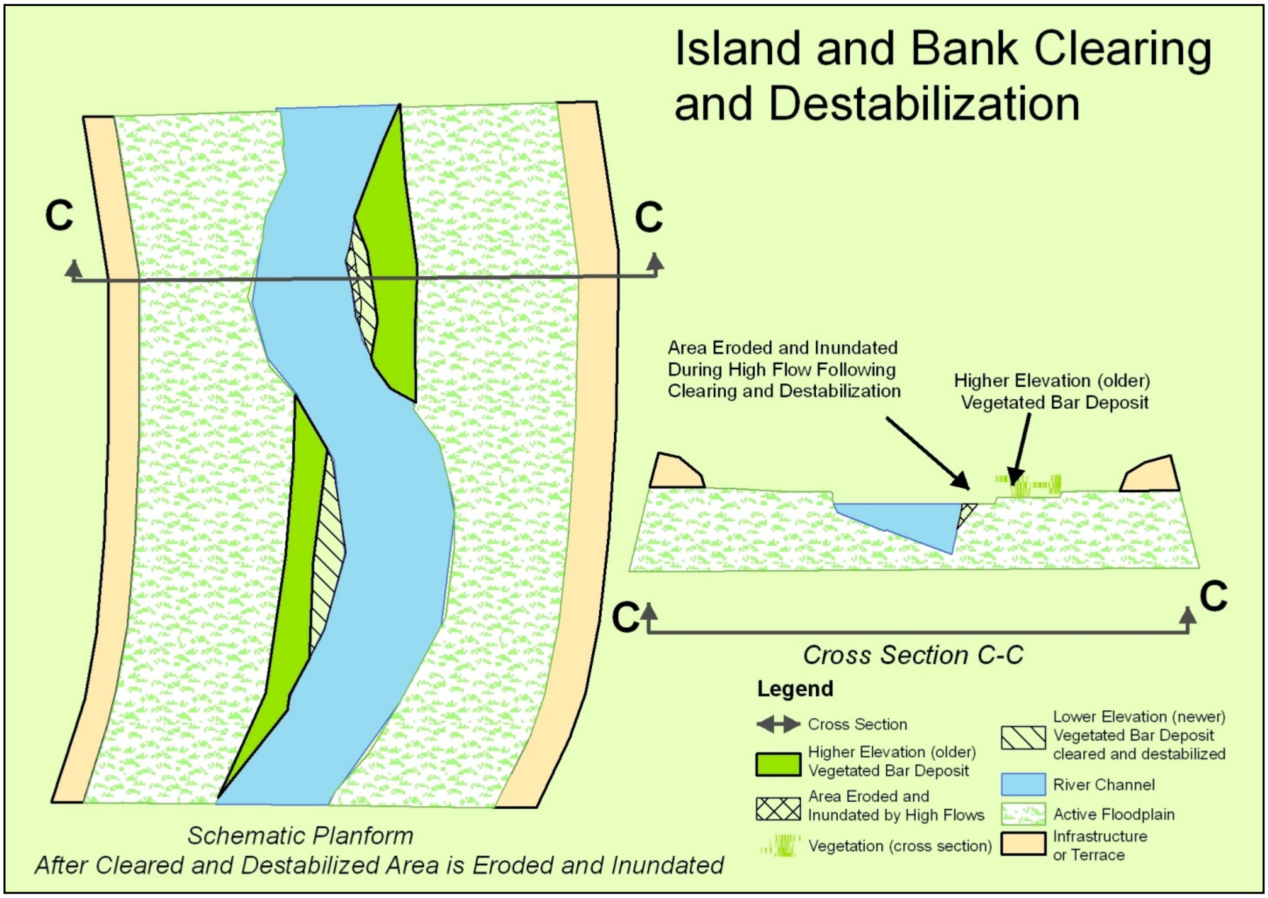


Figure 8–1. Island and bank clearing schematic (Baird and Makar, 2011).

8.1.1 Examples of Application

On the Dray River (gravel bed), the removal of riprap and channel widening has reduced bed shear stress, as well as increased lateral erosion processes and deposition of bed load. This led to channel aggradation and improved flood plain connectivity (Habersack and Piegay, 2008). A reduction in the sediment transport capacity that creates bed load deposits is the key to restoration on the Dray River (Muhar et al., 2008). No documentation was found on the success of this approach in sand bed rivers.

Island clearing is being completed on the Tama River in Japan to reduce water surface elevations during flooding. Historically gravel was mined in the Tama River. After gravel mining was banned, the river reach became a sediment trap catching gravels from upstream. The gravel deposition caused disequilibria in transport downstream. A power generation dam was constructed high in the watershed and also interrupted the gravel supply. This resulted in degradation of the channel and river narrowing. Many of the gravel bars in the river were excavated and lowered causing an increase in width and reduction in shear stress. This resulted in a negative effect including re-germination of floodplain vegetation. Island and bank clearing is currently being utilized, but studies must also identify the geomorphology of the reach (Kinzel, 2002).

Destabilization of bars and banks was used on the Platte River in Nebraska to clear cottonwood vegetation, increase sight distance for endangered birds, mobilize sand, and promote a wide, shallow river (Reclamation, 2004). Island and bar clearing and destabilization consisted of an initial clearing and stockpiling, after which the vegetation stockpiles were allowed to sit for a year before burning in place. Vegetation clearing on islands and river banks also occurred for studies in the middle section near Cottonwood Ranch. River flows after the management activities were at historical low rates, and therefore the potential to affect and the opportunity to detect possible geomorphic change within and downstream from the managed reach were limited (Kinzel et al, 2006). This technique has also been used, in some cases, on the Middle Rio Grande, such as Santa Ana Pueblo.

Channel widening has become a common practice in Switzerland and Austria to restore rivers (Habersack and Piegay, 2008) because the wider channel has a reduced bed shear stress (Bravard et al., 1999). Channel widening in these countries results in a locally steeper channel slope (Jaeggi and Zarn, 1999). Vegetation from the active channel has been removed on several European rivers (Habersack and Piegay, 2008), although no descriptions of the sites, criteria, or frequency of removal were given. It is interesting to note that Jaeggi (1989), Habersack and Piegay (2008) and Leon et al. (2009) report bed shear stress is reduced through channel widening. When the channel widens, the slope is increased to overcome increase resistance of flow associated with a larger wetted perimeter. Presumably this process is dependent on an adequate sediment supply. A key element of this method is estimating the dynamic equilibrium bed slope and elevation. If the channel incises after the project is initiated, then some excavation may be needed so that the newly established surface will be regularly inundated. Leon et al. (2009) have shown that the river can adjust to a new equilibrium slope when the width changes. Field evidence from European rivers supports the conclusion that channel widening can result in a sustainable wider channel with increased bed elevation (Jaeggi and Zarn, 1999). This may lead to the concept of initiating width changes instead of, or in combination with, island and bar clearing and destabilizing. The magnitude of slope change with width change may depend on the amount of the sediment load (Leon et al., 2009). Jaeggi (1989) also presented a method to predict the effect of width change on bed material load transport capacity. This method provides a family of solutions on a curve representing the relationship between sediment transport rate and channel width for a given slope and discharge.

8.1.2 Design Procedure

Project design criteria, hydrology, geomorphic factors and general hydraulic and scour factors were addressed in previous chapters to aid in selecting a suitable bank stabilization method. Important aspects of design criteria, hydrology, and permitting are described in chapter 2. The determination of geomorphic factors including sediment and sediment continuity issues that may impact channel stability are discussed in chapter 3. General hydraulics was also determined, including energy (chapter 4) and potential for scour (chapter 5) that may influence the channel and the extent of the floodplain. These investigations can be revisited to obtain more detailed information for specific areas of this design.

Step 1- Determine the cause of the problem and geomorphic implications
Compare aerial photographs on an approximate decadal scale to determine the locations and size of island formations and channel encroachment if the channel has narrowed through time. Geographic Information Systems (GIS) can be used to document the dimensions of channel encroachment and mid-channel islands. Equally useful is understanding the sediment and other river processes, the causes, and the historical progression of river changes. Understanding the root cause of the change helps in the selection of an appropriate action or bank stabilization method.

Step 2- Define ecological implications and acceptable outcomes
Include biologists or ecologists on the design team to help develop adequate understanding of suitable habitat. Understand habitat needs of the aquatic or avian species of interest, and the role of riparian plants, quality and coverage of existing habitat, and explore future options within the project restrictions. Depending on the size and goals of the project, this may require in-depth information on depth and velocity in main channels, feeding habits and life cycles of target species, consideration of slack water areas where species are found during their respective life stages, seasons and migratory patterns of species of interest and the role of riparian plants to species during access to the site.

Step 3- Analyze existing flow conditions and determine desired conditions
The cleared, destabilized, and/or excavated surface will need to be at an elevation that allows regular inundation to ensure function as a flood plain or as part of the high flow channel. To design suitable elevations, acquire a good set of topographic data from aerial photography, LiDAR, bathymetry field studies and other means. Use a one or two dimensional flow model (or preferably a flow and sediment model, see Step 4) such as SRH-1D (Huang and Greimann, 2013), SRH-2D (Lai, 2008) or HEC-RAS to develop estimates of water surface elevation at various discharges in the channel. Pre-project and post-project water surface elevations and average channel velocity and shear stress should be evaluated. In additional, habitat characteristics should be coupled with flow records and the numerical model to determine the elevation of excavation surfaces. Local knowledge of the discharge associated with important flow depth is useful. It is often desirable to have floodplain surfaces at different elevations with the lowest elevation located near the main channel elevation, and higher surfaces closer to the floodplain or abandoned terrace elevation. Floodplain

surfaces sloping downward towards the main channel can also provide variable depth and velocity conditions.

Step 4- Analyze sediment transport and determine desired conditions
Use the same numerical model to develop a sediment transport budget. From the sediment budget, check the volume of sediment required to counter erosive or incision processes. Consider if sediment from destabilized islands and banks is sufficient to maintain a balanced sediment budget, or if sediment augmentation may be necessary. Vegetation, flow and sediment transport modeling for the Platte River indicated annual scouring flows are necessary to maintain the area free from vegetation regrowth (Murphy et al., 2006). The modeling was also used to provide estimates of the location and volume of the sediment shortage, and to help develop an augmentation plan.

Step 5- Develop design: Feasible methods and schedule for destabilizing islands and banks
Site selection and topographic dimensions may be based upon a combination of channel characteristics, hydraulics and sediment transport, and based on the needs of the aquatic species of interest. Consider land access and participating land owners, scheduling, operating seasons, and workable time frames. GIS plan view channel analysis, habitat needs, discharges significant to the species of interest and the HEC-RAS model, can be combined together to determine optimum site locations and planview dimensions. A pilot channel or pilot cut, excavated through the island, may allow more sediment mobilization to take place.

Islands and banks need to be cleared of vegetation and root plowed for destabilization to occur. Consider the equipment that will be required. This may be a three-step process of clearing first with chains, stockpiling and burning debris or moving debris to suitable sites, and then disking of the ground to cause more rapid erosion. Large (2 to 3 ft) deep root plows should be used several times through the area to provide sufficient destabilization for erosion to occur. In some cases, special discs or other machinery may need to be acquired. Nongovernmental organizations on the Platte River had the discs fabricated (Figure 8-2). Check riverbanks for accessibility and consider the seasons when the machinery needs to access the banks. If the banks are silty and muddy, consider if the machinery pulling the discs can navigate the channel and island banks without becoming bogged down. Similar navigation requirements are considered for channel widening or island lowering operations. Assess the risk of increasing the spread of invasive vegetation and the need for an associated plant control program. Plants like the common reed (*Phragmites australis*) can propagate from any part of the plant and chopping the plants during the clearing process may increase the spread of this invasive plant.

Figure 8–2. Discs pulled by tractors on Platte River to remove vegetation.

Step 6- Develop Sediment Management Design: Feasible Methods and Schedule

Shortages in sediment supply can be managed by destabilizing bars and islands, widening channel and by augmenting sediment. Destabilized islands or bars need to be designed for an elevation that allows frequent inundation and sediment may be generated in the process of widening channels or lowering islands. Placing excavated material (sand or gravel) in the river will also provide for a small amount of sediment enrichment. Continued sediment augmentation for longer periods will require development of a long-term design. Sediment can be placed on low adjacent banks for transport during high flows, can be placed as low bars, or can be placed in main flow paths if the turbidity during placement is acceptable or managed. Material placed on banks awaiting high flows is dependent on the flow regime. As an example, a drought prevented the removal of material from a project bank of the Rio Grande for several years. Gravel was added to the Trinity River in California, initially through dumping from river banks into the water, and later approaches evolved to constructed gravel bars that were submerged and readily eroded. If excavation is involved and material is supplied to the river to provide sediment enrichment, periodic repositioning of the sediment deposits over a several-year period may be needed to ensure the excavated material becomes part of the sediment supply. Required volume will be one of the design criteria and a main goal of the design is to access local sources and minimize or eliminate transport costs.

Step 7- Test design (flow management and sediment augmentation plans) through numerical models prior to implementation. For Platte River studies, Reclamation used a vegetation, flow, and sediment model capable of estimating sustainability of vegetation removal and channel width. Reclamation now uses SRH-1DV (Fotherby, 2012) for modeling vegetation growth. Computations assess the effects of erosive flow, desiccation (ground water table drops faster than plan root growth), inundation, and established vegetation growth rates based on the unique characteristics of the represented riparian plants.

Step 8- Monitor Field Test Sites
If the cleared island or bar is not accessed by high flows, vegetation will rapidly return. A countermeasure could be to alter the height of the cleared island or bar so that inundation and scouring flows occur annually.

8.1.3 Discussion and Recommendations

Clearing and destabilizing banks and islands is most suited to braided and multi-channel rivers. Preserving the floodplain and acquiring lands for the natural processes of bend migration can be another effective option for meandering rivers and multi-channel rivers. This method is dependent on the occurrence of high flows that can transport materials from islands or stock piles on bed or banks of the channel. Unlike more typical constructed projects, this process will occur in stops and starts over time. Implementation complexity could be medium to high, depending on the location where bar/island sediments can be placed in the river for removal by high flows, as well as the difficulty of obtaining access routes.

Environmental Factors. Islands generally provide higher quality riparian vegetation and habitat for wildlife species than shoreline habitats. Island habitat is isolated from human disturbance and some predators. Removal would result in the loss of this habitat. Islands that are subject to overbank flooding and are not perched high above the water table should not be removed or destabilized.

Monitoring and Maintenance. The durability and project life depend on the discharge and elevation of the cleared/destabilized island and whether annual inundation occurs with scouring flows, without additional channel degradation. Assuming these conditions are met, the project life can be multiple years but active maintenance may be needed between these periods. Cleared and destabilized areas can be monitored visually and using cross-section and thalweg surveys to ensure that there are sufficient scouring flows to maintain the cleared area and that the channel stability is not threatened by sediment deposition due to increased high flow channel width. Depending upon the extent of upstream and downstream channel response monitoring may need to extend upstream or downstream. Additional monitoring may include physical aquatic and terrestrial habitat.

Risk and Failure. This method has low risk of bank erosion which could threaten the integrity of river side infrastructure. Cleared areas are susceptible to vegetation re-growth unless either sediment load and/or peak flows are increased.

8.2 Longitudinal Bank Lowering

Today's rivers, with flow and sediment regimes that have been altered by man, are often incised and the river channel is disconnected from the historical floodplain. A reduction in river flow or the process of incision prevents rivers from overflowing banks on a regular basis. Re-establishing floodplain can provide a high level of protection by addressing underlying causes of bank instability and reducing erosive forces. Main channel depths, velocities, and shear stresses can be reduced, leading to lower sediment transport capacity and reduced bank erosion (McCullah and Gray, 2005). Lower banks of the river increase the frequency of overbank flows, increasing the frequency that river flow can rework the floodplain. Using heavy equipment for excavation, natural levees that have built up along the edges of the channel can be lowered, benches can be cut into high banks, or the floodplain can be mechanically graded between the river banks and the terrace to provide the desired floodplain form. Isolating overbank areas from regular flow reduces overbank wetting, increases the vertical distance from the overbank surface area to groundwater (a major factor for riparian vegetation) and subsequently diminishes ecological benefits by excluding these areas as functioning riparian zones.

Compound channels (Figure 8–3) generally confine the range of normal flows to an inner channel, while flood flows expand to the larger channel, i.e. floodplain, formed above the mean annual or 2-year return period flow (Brookes, 1988; USACE, 1989). When there is a need to reduce main channel bed shear stress, the flood plain can be established based on water surface elevations for a design flow between the mean annual flood and the 2-year return period peak flow (Brookes, 1988; USACE, 1989; Haltiner et al., 1996). A main channel can also be widened to reduce sediment transport capacity in channels that are incising (Bravard, 1999).

The range of discharges and specific configuration of terracing or flood plain lowering or widening should be adapted for local site conditions. For example, it may be more desirable in cases where greater flood plain connectivity at lower flows is needed for habitat and sediment transport purposes, for the inner channel to contain flows lower than the mean annual or 2-year peak flow, and the enlarged channel would contain all other flows. In some cases, it may be more desirable for the outer channel to be accessed at larger flows than the mean annual or 2-year peak flow. The channel can also be enlarged along one bank (Brookes 1988), so that a compound channel exists along one bank only.

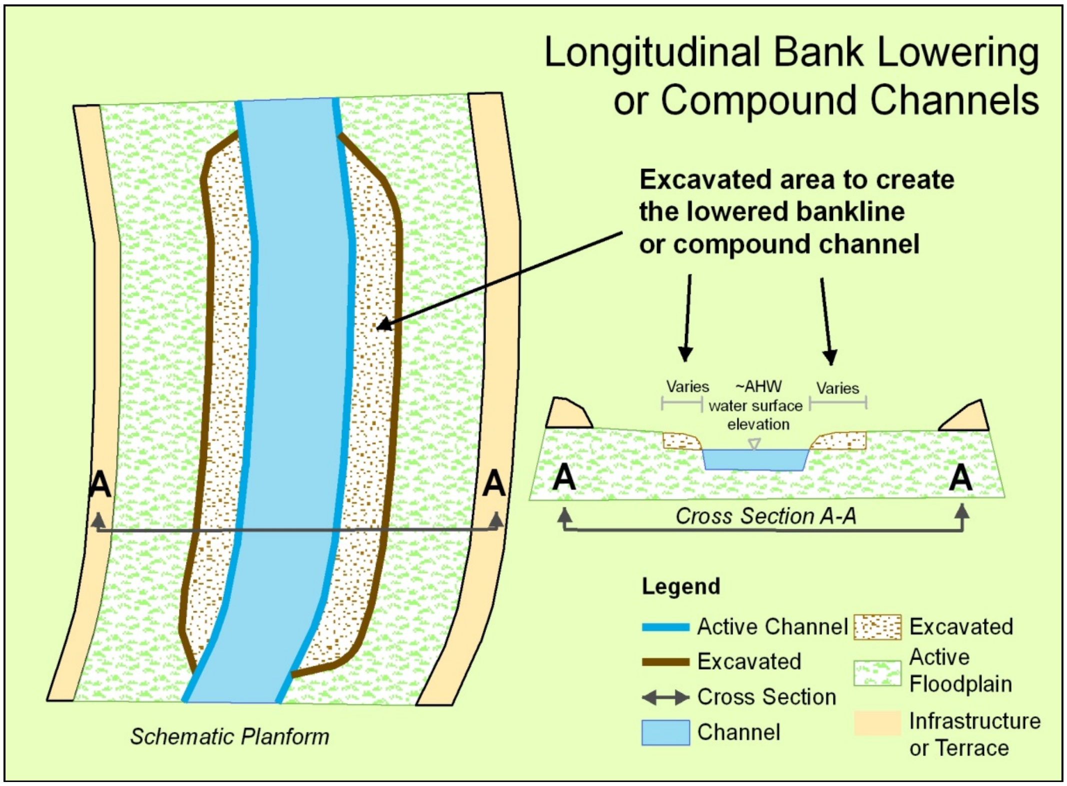


Figure 8–3. Longitudinal bank lowering or compound channels schematic (Baird and Makar, 2010).

A two-stage compound channel is comprised of an inner channel with a larger overbank channel/floodplain. Three- or four-stage channels with multiple flood plain elevations may be appropriate to establish different age classes of riparian forest vegetation and have greater variability of flow depth and velocity for a wide range of discharges. A levee that is constructed with some multi-stage form incorporates some geomorphic and ecological processes within the levee system. Creating channel floodplain through a multi-stage form uses geomorphic concepts to guide the channel from a present stage of instability towards dynamic equilibrium. A multi-stage form will increase storage of flood flows and accommodate meander migration in the inner channel.

8.2.1 Examples of Application

The widths of the flood plains throughout the Bitter Lake NWR vary from zero to several thousand feet. The combined width of the channel and floodplain would be excavated to be at least 350 ft. This minimum width provides room for a 150-foot-wide channel and adequate floodplain. The terrace may need to be excavated in some spots to create a 350-foot-wide floodplain. When implemented in combination with vegetation removal, lowering the bank elevation and removing the 2,500-foot-wide strip of saltcedar would improve the ability of the river to shift course across the floodplain and to create more floodplain by eroding into the terrace. Where the banks need to be lowered by more than two ft, the bank excavation would also remove the saltcedar root crowns at the same time. Where the banks need to be lowered by less than two feet, the saltcedar would be removed mechanically by bulldozing the trunks, cutting off the roots, and removing the root crowns.

8.2.2 Design Procedure

Project design criteria, hydrology, geomorphic factors and general hydraulic and scour factors were addressed in previous chapters to aid in selecting a suitable bank stabilization method. Important aspects of design criteria, hydrology, and permitting are described in chapter 2. The determination of geomorphic factors including sediment and sediment continuity issues that may impact channel stability are discussed in chapter 3. General hydraulics was also determined, including energy (chapter 4) and potential for scour (chapter 5) that may influence the channel and the extent of the floodplain. These investigations can be revisited to obtain more detailed information for specific areas of this design.

Step 1. Determine the design hydrology. Frequently used design flows for the two-stage channel are mean annual peak flow, 2-, 10-, and 25- year peak flow events. The design flow events depend upon existing or proposed channel terrace or floodplain elevation, and existing or proposed channel capacity. Also important to the design is the frequency when flows reach an elevation where they begin to flow into the second stage channel.

Step 2. Determine elevation of the inner channel banks and form of channel bends. The inner channel should be designed to be sustainable with the low-flow sediment size and load, as well as hydrology. Often the inner channel is sized to convey the mean annual flow, or the 2-year peak flow and corresponding sediment supply. Shape the inner channel to provide habitat complexity, but for sustainability, remain within the constrictions of the river form. For example, despite the desire to provide these types of habitat, a braided river normally does not have a meander configuration or deep pools, and meandering rivers with drop structures that have removed most of the energy, will not be able to sustain constructed deep pools in bends.

Step 3. Numerically model flow and design elevations for 2nd-stage, 3rd – stage or greater elevations. Hydraulic aspects of overbank flow are complex (Brookes 1988) and generally require multidimensional modeling to determine the effects of the compound channel on sediment transport, flood peak water surface elations, scouring flows, and sustainability. Use 1D (SRH-1D or HEC-RAS) or 2D (SRH-2D) flow models to determine elevations and hydraulics. If a 1D flow and sediment transport model is used, the same model can be used to address sediment concerns in the next step. A 1D or 2D flow, sediment, and vegetation model (SRH-1DV, SRH-2DV) can also be used for this step, the sediment step and the vegetation analysis and/or plant design in a later step. An iterative process for determining optimal design elevations is presented in the next step.

The elevation and width of the second stage channel should be designed to be inundated at the elevation that best suits local conditions for improving habitat and reducing shear stress, depth, velocity, and sediment transport capacity. The design flood for the two-stage channel could be a 10- to-25-year event, depending on the terrace elevation and existing channel capacity. More important to the design is the frequency when flows reach an elevation where they begin to flow into the second stage channel. The discharge when waters begin to flow in the second stage channel should be the value that best balances sediment transport capacity with supply in the main channel.

Significant shear stress on the interface between the main channel and the overbank, due to lateral momentum exchange, must be accounted for in determining roughness for modeling. Thorne and Soar (2000) have developed a method to determine the Manning's 'N' multiplier for compound channels. This method can have a relatively high reliability if the newly established channel process is sustainable. Consider the advantages and disadvantages of scouring flows, a natural process, in the second stage channel. This will be a project specific assessment. Peak design flow water surface elevation, construction cost, and excavated material disposal should also be considerations.

Step 4. Assess sediment transport to determine dimensions of the second-stage channel/floodplain. It is important to the successful operation of this method to correctly determine sediment supply reaches and/or sources, sediment transporting reaches and any depositional reaches. This information may have been identified in the geomorphic assessment. After this information is available, determining the dimensions of the second-stage channel in an iterative process involving a 1-D flow and sediment transport model.

A. Perform 1-D hydraulic model for the mean annual flood, 2-, 5- and 10-year return period peak flows.

B. Determine the sediment supply at the nearest upstream gage if data exists, tributaries, or main channel supply reaches for the mean annual flow 2-, 5-, and 10-year peak flows.

C. Compute sediment transport capacity of the existing channel for the mean annual flood and the 2-, 5- and 10-yr return period flood.

D. Select preliminary elevation, and plan view location and dimensions of bank lowering based upon the mean annual flood or the 2-yr return period flow.

E. Perform 1-D hydraulic model for the mean annual flood, 2-, 5- and 10-year return period peak flows based upon preliminary bank lowering elevation and plan view location and dimensions.

F. Compute main channel sediment transport capacity and compare with supply, and pre bank lowering capacity.

Return to step D until the main channel sediment transport capacity is in balance with the supply to the site and stakeholder interests.

Step 5. Grading plans and earthwork considerations.
Earthwork is a significant cost on bank lowering projects and a major goal is to minimize transport of the material offsite, transport of material onsite, and movement of stockpiles onsite. Carefully balance the material excavated against material placement to reduce transport costs. If material is excavated from a lowered terrace or cut bench, consider wasting the material within the contoured grading plan. Sloped surfaces can help drain the floodplain and prevent trapping fish on the declining limb of a hydrograph. Contoured berms at suitable locations can help slow active bend migration and channel alignment shifts. This grading is also aesthetically beneficial since only the downstream reaches of large rivers will produce a floodplain that has a consistently horizontal surface.

Consider the level of compaction that is required for locations of placed material. If the material is not supporting structures, it may not require compaction but unconsolidated material placed in the bends of rivers may erode at a faster rate than the in-situ ground. Compaction from loaded on-site trucks can also be specified as a minimum number of passes over the material placement. Conversely, very dense material and layers of clay are difficult for the roots of shrubs and trees to penetrate. Compact ground or ground that has been traversed by vehicles and heavy machinery will require raking/ripping to loosen soils before placing topsoil, and before seeding or planting.

Consider the clearing that is required prior to earthwork. Will cleared trees be removed or burned on site? Will the material be left in-place or buried in the fill to introduce natural irregularities in the floodplain, and serve as debris? Commonly, restoration projects need to preserve essential vegetation that may be anchoring channel alignments with root structures, preventing avulsions as roughness elements, serving as habitat, providing woody debris for geomorphic stability or the food cycle, or buffer sediment runoff to the river. Trees can be damaged by the placement of waste material around the roots or by the traffic passing over the roots, so protection should be called out in drawings if vegetation can be preserved at the existing location and elevation. Some advantageous rooting species like cottonwood can survive some burial depending on depth of material. Many riparian plants can survive a foot of burial. Trees that are damaged during construction may still serve as bird perches and other habitat, and provide natural woody debris. Also consider preserving mature trees and plants that serve as critical seed sources for desired plants, and remove undesirable seed sources. For example along the San Joaquin River in California, mature Fremont cottonwood (*Populus fremontii*) and black willow trees (*Salix gooddingii*) provide valuable seeds; elderberry plants (*Sambucus sp*) are habitat for the elderberry beetle; and mature scarlet wisteria (*Sesbania* *punicea*) shrubs, an invasive plant that forms a monoculture along bank lines, should be aggressively removed from upstream locations to prevent the rapid downstream spread of seeds from this plant.

Construction sites are locations of complete disturbance with vehicles and equipment transporting in odd seeds unnoticed. Invasive plants can be easily spread at construction sites and programs for invasive plant control may be required following construction.

Waste material used to anchor bends and slow alignment shifts can also serve as part of a sediment augmentation effort. The placed material may be eroded during high flows. Material wasted as sediment augmentation material may be dumped into the river at high flow locations, or placed directly in the channel as low bars. Augmented sediment placed in the channel or along the banks may need to be repositioned over a several-year period for flows to completely remove all sediments.

Step 6. Design vegetation planting plan and/or woody debris elements.

See Section 9.2 and 9.3 for information on vegetation planting, and Section 10.1 for designing woody debris.

Step 7. Develop and review the required construction steps/schedule for constructability. Construction issues include an assessment of access requirements, material transport, and construction timing. Consider size of equipment and width of blades and shovels required for the work. Small channel widths may be influenced by equipment size. A thumb for an excavator or backhoe is needed to place large boulders in specific locations. Ultimately this assessment is the contractors responsibility but the designer can review to ensure there are no major roadblocks (availability of large riprap meeting specifications, delays in permitting), no unnecessary and costly requirements (difficult access, no storage areas, neglected stakeholder requirements), missing elements to the process (access roads or consideration of species construction windows) or potential scheduling roadblocks (planting season missed by poor planning on the nursery schedule, delays in scheduled utility changes).

Check if access to the site is available. If access roads will need to be constructed, consider whether they should be permanent or removed after the project. Consider whether they will have to traverse wet areas from drainage or springs creating sites where machinery could bog down. Look at suitability of existing access roads for the size of trucks required, neighborhood noise issues, tight corners, and consider if adjacent landowners will have concerns. Are there material storage areas at the sight and pull-off areas for large trucks?

8.2.3 Discussion and Recommendations

If the sediment transport is in balance with supply over the long term, some floodplain deposition during high flow events, as well as floodplain erosion is expected. However despite occasional events disturbing the system, the project life should be long, potentially extending several decades.

Risks. At high flows, the greater cross sectional area could locally reduce the water surface elevation, lowering the risk of flooding riverside infrastructure. As a result of the increased cross sectional area, the potential for lateral migration may also be reduced. It is also possible for the lowered bankline to fill with sediment over time, reducing the access or the floodplain flow capacity, and also altering the channel alignment.

Monitoring and Maintenance. Maintenance requirements may include selective sediment removal from the second stage channel surface and repositioning excavated material in the river periodically until it is completely eroded by high flows. Maintenance actions may also include reshaping access and egress of flows on the floodplain, to improve sediment transport through this reach. Maintenance decisions to excavate should be balanced against an assessment of the impact excavation will have to vegetation and floodplain habitat.

The 1D flow and sediment transport model could be calibrated and used to determine the extent of sediment removal to maintain channel capacity. A 2D numerical model could be used to evaluate and develop improvements in the overbank configuration and grading plan, to prevent future sediment deposition. Monitoring could include visual observation or, in the case of larger scale bank lowering, comparative cross sections to determine sediment deposition. A 1D or 2D flow, sediment and vegetation model can provide information on both sediment deposition and the establishment and coverage by vegetation.

8.3 Side Channels

Side channels can convey flows most of the year or can be high-flow side channels that are only accessed by river waters during peak flow events. High flow channels can be located adjacent to the main river channel, located in the flood plain, or can cut across bars and islands (Figures 8–4 and 8–5). High flow side channels are often referred to as “flood bypass channels” (Richards, 2004), “flood relief channels” (Hey, 1994), or “overflow channels” (WDFW, 2003). The importance of high flow side channels has been documented in natural systems for passage of peak flows, reducing erosion potential of peak flows in the main channel, and ecosystem health (Richards, 2004). Both high flow and low flow side channels can reduce the potential for bed and bank erosion by reducing the discharge carried in the main channel during large flow events. Less discharge translates to reduced shear stress, velocity and flow depths in the main channel. Side channels also have the potential to decrease the peak flow water surface elevation in the main channel by increasing total conveyance area at a section.

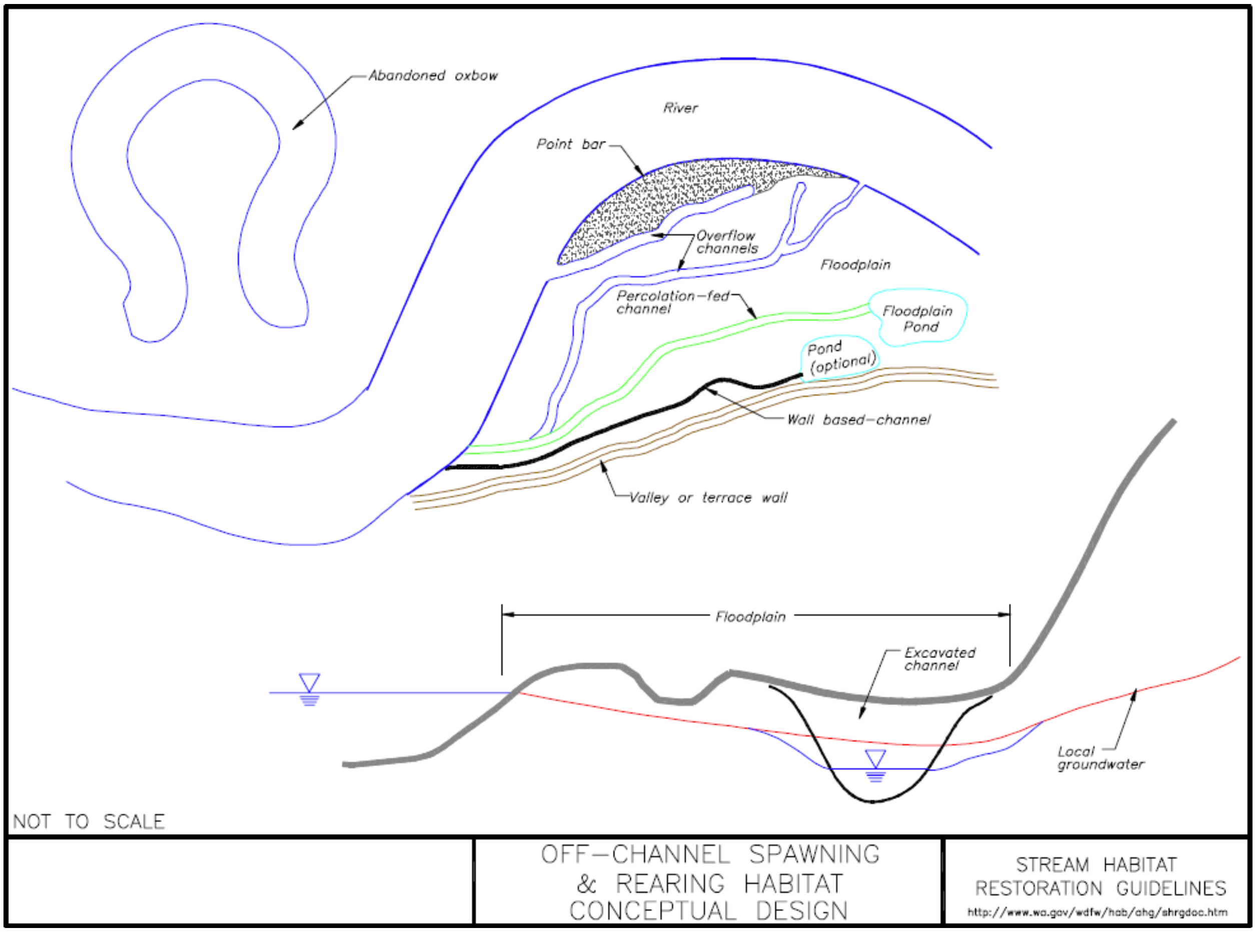


Figure 8–4. Side channel plan view. From Saldi-Caromile et al. 2004.

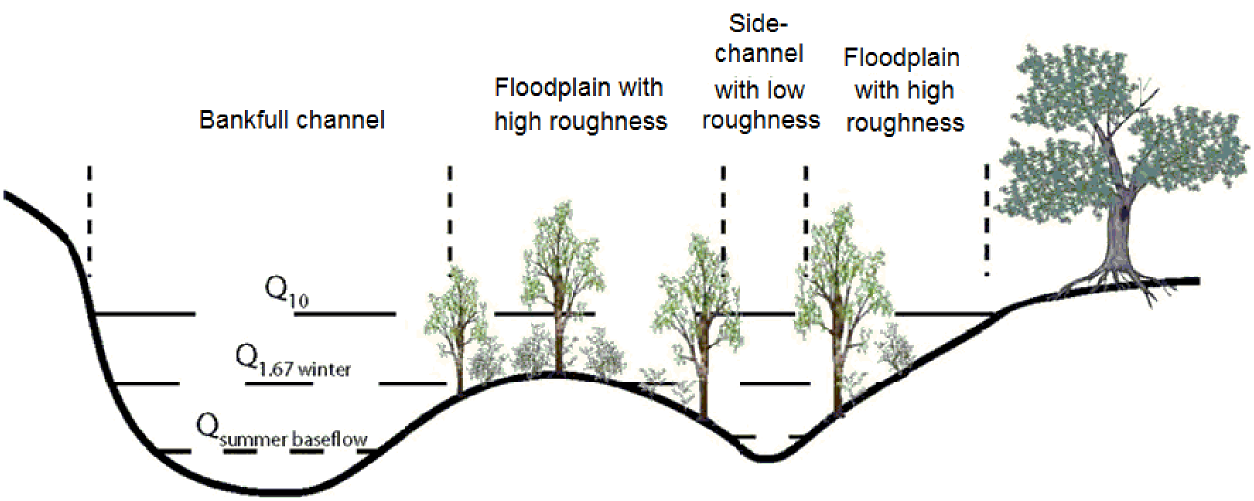


Figure 8–5. Side channels (HVTFD, M&T, and NHE 2011; used by permission of the Hoopa Valley Tribe, and McBain Associates).

Side channels may be reconnected where levees, roads, pushup berms, and other structures may have caused floodplain disconnection. Channel incision is another cause of side channel disconnection. New side channels can be constructed by enlarging natural topographic low areas on bars, abandoned flood plains, or abandoned terraces.

The most common mode of failure is sediment deposition in the side channel or oxbow exit and entrance. Oxbows are disconnected side channels that have separated at both ends from the main channel and appear as a *C* when viewed in plan view. In rivers with suspended sediment, flood plains and high flow side channels tend to store suspended sediment (Marriott, 1996). High flow side channels, in a natural system, tend to fill with sediment over time (Hey, 1994), and as the channel avulses or migrates laterally, new side channels are created. When this natural channel dynamic condition does not exist, high flow side channels tend to fill with sediment. Despite the potential costs for rehabilitation or modifying and improving the feature, high flow side channels are an important morphological feature that can be a feasible solution.

High flow side channels can be used to reconnect flood plain areas (Figure 8–4) and have greater sustainability in rivers with lower amounts of suspended sediment and sand bed load (Hoopa Valley Tribe, McBain & Trush, Inc., and Northern Hydrology and Engineering, 2011). In incising rivers, flow diverted to side channels may decrease main channel sediment transport capacity, possibly resulting in deposition and a rising bed in the main channel. This could potentially bring the sediment transport capacity to a closer match with the sediment supply, thus decreasing the tendency for continued channel incision. Corresponding characteristics are an increase in local slope, water surface elevation, and width-to-depth ratio (Schumm, 1977). When the high flow side channels have reduced the tendency for continued channel incision and deposition occurs in the main channel, the effects of change may be noted a considerable distance upstream of the project site.

8.3.1 Design Procedure

Project design criteria, hydrology, geomorphic factors and general hydraulic and scour factors were probably assembled previously to aid in selecting a suitable bank stabilization method. Important aspects of design criteria, hydrology, and permitting are described in chapter 2. Determine the geomorphic factors including sediment and sediment continuity issues that may impact channel stability as discussed in chapter 3. Define general hydraulics including energy (chapter 4) and potential for scour (chapter 5) that may influence the channel and the extent of the floodplain. Some of these investigations may be revisited to obtain more detailed information for specific areas of this design.

Step 1. Acquire good topography. Locate low topography areas or old oxbows that could be used for side channels.

Step 2. Assess the geomorphology to aid sustainability including sediment transport and alignment. The location of the entrance to a constructed side channel is important. To reduce sediment intake and deposition, locate the entrance in a non-depositional area such as on the outside of a bend where the mainstem approaches a floodplain or terrace. Side channel entrances should not be placed at the heads of point bars. One recommendation is to locate the intake on the outside of the meander bend or on the downstream portion, but upstream of the transverse bar to reduce risk of sedimentation. (Hoopa Valley Tribe, McBain & Trush, Inc., and Northern Hydrology and Engineering, 2011).

As with split flow channels, the entrance angle can be less than or equal to 40 degrees to maintain sediment transport competency. Entrance angles higher than 40 degrees cause flow separation during high flows, and if there is a local sediment supply, will cause eddy deposits and close the entrance. (Hoopa Valley Tribe, McBain & Trush, Inc., and Northern Hydrology and Engineering, 2011).

Side channels should not be placed adjacent to tributary deltas due to the locally high sediment supply. (Hoopa Valley Tribe, McBain & Trush, Inc., and Northern Hydrology and Engineering, 2011).

Step 3. Design the size and alignment of the side channel. The bed slope of the side channel should be greater than the bed slope of the main channel. The upper portion of the side channel should be steeper than the lower two thirds of the side channel. Roughness in the upper one third of the side channel should be managed to force flow down the channel. This would be accomplished by increased roughness on the banks. In the lower portion of the side channel, wood and other vegetation can be encouraged to increase habitat complexity.

Side channel length can be a function of slope and floodplain area. A channel that is too long will silt in and close relatively quickly. The side channels should be greater than ½ wavelength in length, and can be up to two meander wavelengths (Hoopa Valley Tribe, McBain & Trush, Inc., and Northern Hydrology and Engineering, 2011).

The upstream-most ½ of the side channel should maintain sediment transport competency for the same bed load sizes conveyed by the mainstem river, such that any sediment entering the side channel entrance can be routed down the side channel to preserve entrance integrity. Sediment transport competency should be maintained by preserving slope, managing roughness on the banks to encourage locally high shear stresses in the side channel entrance, and avoiding large wood placement in the side channel entrance. (Hoopa Valley Tribe, McBain & Trush, Inc., and Northern Hydrology and Engineering, 2011).

The side channel should probably not convey more than 10 percent of the summer and winter baseflows such that mainstem sediment transport capacity is maintained. (Hoopa Valley Tribe, McBain & Trush, Inc., and Northern Hydrology and Engineering, 2011).

Step 4. Design the entrance and exit conditions. The entrance should be in a non-depositional area such as the outside of the meander bend, on the downstream portion, but upstream of the transverse bar to reduce risk of sedimentation. It could also be located where the mainstem approaches a floodplain or terrace.

The side channel can be self-maintained by a constriction at the junction like a rigid structure or an engineered log jam or boulders. Hydraulic conditions would change over time with channel evolution or with sediment and debris. A constriction could make the channel self-sustaining by maintaining a scoured thalweg with a low flow water supply.

Side channels should not be placed at the heads of point bars. Side channel entrance angles should be less than 40 degrees to reduce flow separation and eddy formation during high flows that may form a deposit. As with the split flow channels, the entrance angle should be less than or equal to 40 degrees to maintain sediment transport competency. Entrance angles higher than 40 degrees cause flow separation during high flows, and if there is a local sediment supply, will cause eddy deposits and close the entrance.

Large wood placement at the entrance to “scour” the entrance clean is not recommended at this time, as improper placement could cause an obstruction on-induced backwater bar, or a local scour hole and bar that causes the entrance to close. (Hoopa Valley Tribe, McBain & Trush, Inc., and Northern Hydrology and Engineering, 2011).

On Reclamation projects, small barbs or spur dikes have been used downstream of the entrance and exit to discourage sediment deposition. Small barbs or spur dikes create a zone of local velocity acceleration to encourage sediment transport through the side channel entrance and exit. In gravel bed systems, placement of large woody debris across the entrance has been suggested to create a local roughness zone that has local velocity acceleration encouraging sediment transport through the side channel entrance.

Side channel exits with alcoves should occur downstream of riffle crests, but upstream of the next downstream riffle crest to retain sediment transport competence through the alcove, thus reducing risk or downstream sedimentation. (Hoopa Valley Tribe, McBain & Trush, Inc., and Northern Hydrology and Engineering, 2011). The alcove thalweg elevations should be the same as the thalweg of the mainstem river at that location to create the desired backwater conditions at summer base flows, but retain sediment transport competency during higher flows.

Step 5. Estimate and adjust the sediment transport capacity. When the side channel carries sufficient flow to reduce the main channel sediment transport capacity, sediment deposition can occur in the main channel.

8.3.2 Discussion and Recommendations

Side channels are an inexpensive way to encourage flood plain connectivity. However these channels can transition to depositional zones, possibly requiring maintenance or relocation of the side channel in the future.

Risk and Failure. Sediment deposition can occur in the main channel, and the entrance to the side channel can plug due to sediment because flow splits in natural streams are often transient. (Hoopa Valley Tribe, McBain & Trush, Inc., and Northern Hydrology and Engineering, 2011).

A second mode of failure is a major shift in the river that captures the constructed channel during a large flood (WDFW, 2004). Where the potential for flow capture exists, constrictions made of boulders can be used to control the flow passed by the side channel to reduce this risk. The Middle Rio Grande has a high level of flow control and this model of failure has not been observed for a period of 10 years previous to the first draft of these guidelines.

Monitoring. Ecological monitoring can provide a measure of project success if quantitative indicators can be developed from field based monitoring studies and from remote sensing (Golet, et al. 2013). Relevant indicators can be assessed in areas including:

- Flows and Flood Processes
- Native and Invasive Riparian Vegetation
- Birds and Wildlife
- Fisheries
- Macro-invertebrates
- Water Quality
- River Planform and Geomorphic processes

Monitoring should be done for several years and for longer periods of time to track trends of the response. Monitoring flows includes annual flow measurements collected at the same time of the year, and documentation on sediment transport and armor layers is also applicable to River planform and Geomorphic processes studies. Annual spawning counts and red surveys are the most direct measure of salmonid spawning, while trapping and counting adult and juvenile fish, and monitoring minnows are valuable investigations for most fisheries studies. If water quality is an issue, basic water quality parameters should also be measured (WDFW, 2003).

Side channels and the adjoining main channel can be monitored visually or monitored using cross-section and thalweg surveys to ensure main and side channels are stable and not threatened by erosion or sediment deposition. Depending on the extent of upstream and downstream channel response, monitoring may need to be extended upstream or downstream. Additional monitoring may include physical aquatic habitat, fish presence or absence, and surveys of macro-invertebrates.

Maintenance Requirements. Maintenance requirements may include repositioning of sediment deposits in the side channels or selective sediment removal to ensure connectivity is maintained. An alternative approach is to integrate side channels with a finite design life into the overall design.

8.4 Channel Embayments

Lower the surfaces along the active river's edge to simulate connected embayment areas. Channel embayments are most applicable at locations with minimal backwater where they can produce complex eddy currents and summary near-zero flows. Embayment elements are generally too small to impact existing river morphological processes and the most common mode of failure for embayment elements is sedimentation. Countermeasures to sediment deposition include re-excavation or relocation of the embayment. These are relatively simple features; excavation is the main construction component.

8.4.1 Examples of Application

Channel embayment features are studied on the Middle Rio Grande to aid the endangered silvery minnow (*Hybognathus amarus*). Historically the Rio Grande was a braided sand bed river but it is now transitioning to a meandering, multiple channel form with bimodal bed material. Under these conditions, semi-buoyant silvery minnow eggs can suffer from excessive downstream drift (Tetra Tech EMI, 2004). Embayment areas are a promising method that has improved silvery minnow egg retention.

Embayments can provide quality nursery habitat due to:

- A drift zone for retaining eggs and larvae out of the current
- Sufficient depth and area for rearing larvae
- The appropriate elevation to be inundated at minimal flows during spring runoff
- The capacity to remain inundated for 1 to 6 weeks to support larval growth prior to returning to the river (Massong et al., 2004)

8.4.2 Design Procedure

Project design criteria, ecological factors, hydrology, geomorphic factors and general hydraulic and scour factors were assembled previously to aid in selecting a suitable bank stabilization method. Important aspects of design criteria, hydrology, and permitting are described in chapter 2. Determine the geomorphic factors including sediment and sediment continuity issues that may impact channel stability as discussed in chapter 3. Define general hydraulics including energy (chapter 4) and potential for scour (chapter 5) that may influence the channel and the extent of the floodplain. Some of these investigations may be revisited to obtain more detailed information for specific areas of this design.

Step 1. Select potential embayment locations. Using aerial photographs the most favorable geomorphic plan view conditions can be determined. Identifying potential sites usually involves some knowledge of the depth and velocity and slack water areas where fish are found during their respective life stages. The more promising sites can be inspected in the field for more detailed information on geometrics and geomorphic characteristics.

For locations similar to the Rio Grande, consider constructing inlets at riverside drain outfalls so that they connect to the outflow channels. The outfalls can be widened to promote shelf development and to slow the outflow waters to create a slower water environment (Massong et al. 2004). Inlets can also be constructed at the downsteam end of a high flow side channel.

Embayments readily fill with sediment. They have high maintenance requirements since the design/project life is low without sediment removal. In the search for potential sites, consider reaches of river that may have less sediment transport and sites where the river alignment will make embayments less prone to sedimentation during the falling limb of the hydrograph.

Bank line location depends upon natural topographic and geomorphic features. Often these are constructed along the banklines in relatively straight reaches of sand bed main channels and islands. Locating embayments along the outside of mild, relatively stable channel bends may reduce embayment sedimentation. The upstream edge of islands may also be a good location to reduce sediment deposits. A rectangular shape directs currents into and out of the inlet (Massong et al., 2004).

Locating shelves and inlets on point bars or islands where they can be connected to river flow and convey surface water helps provide flows to flush fine sediments (Massong et al., 2004). Natural topographic low areas where some excavation and maintenance may provide ideal embayment locations can be found on the downstream edges of point bars. High flow side channels through the point bar can provide periodic flushing flows to remove sediments which may have deposited in the embayment.

Step 2. Assess potential sites with a numerical model. Use flow records, topography, and bathymetry data to develop a 1D or 2D flow model, or flow and sediment model. Output from the model, combined with knowledge of desirable habitat characteristics, can be used to assess the hydraulics of potential sites. Flow elevations at the embayments can also be assessed and these analyses will help form a general estimate of the excavation required at each site. Reclamation's SRH-1D, SRH-1DV, or COE's HEC-RAS are all well suited for determining general flow elevations over a large longitudinal distance. Consider using SRH-2D (Lai, 2008) for more detailed information on eddy patterns at individual sites. If the modeling is done with a flow and sediment transport model instead of a flow model, output on sediment transport may also help in selecting sites less susceptible to deposition.

Step 3. Select the location(s) and design the embayments. Finalize the location, dimensions, and elevations for each embayments. The shelf inlet should be connected to the river, with a shelf slope that allows a variety of depths to occur through a range of discharges which are known to provide habitat. Often these are 1/3, 1/2 and 2/3rd's of the 2-year return period flow peak. Shelves, inlets, and scallops have also been excavated to depths that allow inundation to begin at 1/10th of the two year peak depending on habitat needs. In Rio Grande studies, some vegetation on the shelf can help provide reduced flow velocities for silvery minnow egg and larval retention (Massong et al., 2004).

Embayments are constructed perpendicular to the main current, or oriented upstream 45 degrees. Dimensions are highly variable and depend on the natural topographic and geomorphic features found on each river, and on the habitat needs. Dimensions typically range from 30 to 130 ft wide and 50 to 80 ft long. These dimensions are for general guidance only and can vary. Natural analogs can also be used to determine dimensions with the assumption that size can scale with river width.

Step 4. Evaluate excavation and develop a grading plan. Compute the required cut and fill and make any possible adjustments to help balance the volume. Develop a grading plan for the area that allows for wasting all material on site. If this is not possible consider a sediment augmentation plan to the river or transporting the material to the nearest off site location.

Step 5. Assess constructability issues and revise the design accordingly. Constructability issues include access to the site; access to the site without disturbing vegetation habitat; and access that avoids wet areas where machinery cannot operate. Consider if long, excavator arms are needed for heavy machinery to work off a bank or other high ground. Also consider permitting requirements for machinery working in and around flowing water and seasonal flow restrictions. Evaluate the potential for coffering off the site and working in standing water, and evaluate the window/seasons for working on site. Look for storage sites for excavated materials and parking/storage for machinery. Assess utilities and determine if any lines need to be protected or moved. Develop an approximate scheduling plan and add permitting and utility scheduling considerations to the plan.

Step 6. Design the site recovery plan. Develop planting requirements and coordinate with nurseries. If irrigation is required to establish plants, design an irrigation plan. Develop seeding plans, design seed mixes and specify seasons and windows. Ensure the project design has accounted for temporary bank erosion protection during the establishment of vegetation (3 years) and includes a plan for invasive vegetation control during this period.

Define the period of all post-project work and proposed maintenance. Determine which access roads and tracks need to be removed, raked, and/or seeded. Evaluate the need for gates to control site access. Embayment sites will require continued access for maintenance. Design a site inspection/monitoring plan and specify the period of evaluation. Develop a maintenance plan for the removal and disposal of sediment that may deposit in the embayment area.

8.4.3 Discussion and Recommendations

General information, requirements, and conceptual drawings of inlets and shelves for bank line embayments are provided in Massong et al. (2004). Typical dimensions are not included on these drawings. For example the width and length (distance into the bank) necessary for inlets to provide a suitable drift zone are not shown for various river velocities and bank line locations. Additional information on embayment's including more extensive and systematic design methods, construction recommendations, and post-construction monitoring outcomes are needed.

Bankline embayments, shelves and backwaters can be an inexpensive method to simulate historical channel features and flood plain habitat types that are frequently inundated. Frequently, these types of structures fill with sediment, however, they are relatively inexpensive and easy to construct. Sediment removal may need to occur every few years to maintain function.

Risk and Failure. Level of reliability is relatively high because there is documentation of positive effects for egg retention and larvae usage (Massong et al., 2004). These structures are low risk and do not increase flood potential, initiate bank erosion processes nor affect sediment transport. Sediment deposition is the main cause of diminished effectiveness over time. Maintenance can include periodic sediment removal.

Monitoring and Maintenance. Biological monitoring provides a measure of project success. Annual spawning counts and red surveys are the most direct measure of salmonid spawning. Trapping and counting adult and juvenile fish is also useful. This should be done for several years. Monitoring of physical conditions, including extent of sediment deposition that could hinder effectiveness of embayments, and water quality monitoring should also be completed if water quality is an issue. This would include annual flow measurements at the same time of the year and documentation on coarse sediment in the side channel.

9 Design of Vegetated, Deformable Banklines

There are many ways to establish vegetation on banks and most provide less erosion resistance in comparison to the bank hardening materials of riprap and concrete. The lower erosion resistance of vegetation is, however, a benefit when the desired result is a natural deformable bankline. A deformable bankline will allow the adjustment of channel shape and floodplain shape in response to changing flow and sediment conditions. Like the previous chapters, sustainable and environmentally effective methods using vegetation and deformable banks can expand or utilize a floodplain to help stabilize banks.

Several methods for establishing vegetation are presented in this chapter: seeding with natural colonization, planting, planting with coir fabric, live-staking, and fabric encapsulated lifts (FES). The design procedure for a degradable toe that can be combined with vegetated banks is presented, and in the final section, a stone toe is combined with a vegetated bank.

When compared to inert materials, vegetation appears more complex to work with but also offers unique benefits. Vegetation as a live material can be self-sustaining under suitable conditions, it can adapt to temporal changes in topography, it provides habitat for other species, provides food sources for other species, and can prevent the warming of river flow temperature by overbank shading. One of the challenges of designing with vegetation is understanding how the installation factors, timing, and age of the plants affect the erosion resistance and the stability of the bank. The use of living vegetation to help stabilize banks is sometimes identified as bioengineering.[1] The following excerpt from the FHWA, HEC23 manual, summarizes the hesitation but also the growing recognition of bioengineered approaches to bank stabilization:

Level of installation complexity can be fairly high. Due to a lack of technical training and experience, there is reluctance on the part of many engineers to resort to soil bioengineering and biotechnical engineering techniques and stability methods. In addition, bank stabilization systems using vegetation have not been standardized for general application under particular flow conditions. There is a lack of knowledge about the properties of the materials being used in relation to force and stress generated by flowing water and there may be difficulties in obtaining consistent performance from countermeasures that rely on living materials. Nonetheless, stabilization of eroding stream banks using vegetative

[1] This label is shared with another branch of science that is defined as the application of engineering principals to solve medical problems.

countermeasures has proven effective in many documented cases in Europe and the United States (Lagasse et al., 2009).

9.1 Shape and Deformable Banklines

Channel and floodplain shape effect the magnitude of the erosive force. The erosive force can be reduced by altering the bank shape. Deformable banks are a self-sustaining means of balancing the erosive force of flow with some resistance from vegetation coverage. Limiting this balance is the size of the floodplain. When there is a suitable expanse of floodplain, banks can erode back, altering the cross section shape and lowering the erosive force. Vegetation on a steep bank may be insufficient erosion protection, but the same plants may prevent erosion if the bank slope is laid back and/or the floodplain is expanded.

Erosion resistance of plants will also vary depending on the age of the plant. Young plants have less erosion resistance than mature plants and may require irrigation for several years while their root system develops. Once established, however, plant erosion resistance increases and this protective material is self-sustaining for years. In comparison to inert materials, vegetation is more complex to work with during design and construction, but this is compensated for through the attributes of self-perpetuation and dynamic coverage, the ability to adapt to geomorphic change, and the additional functions of the living material as a habitat element and food source.

Five areas of a bankline are relevant to the design:

Toe Zone. This lowest horizontal segment of bank is bounded by the streambed and the average normal water stage. Due to long inundation, this area typically has no riparian vegetation and flow currents prevent colonization by most wetland or aquatic plants (NRCS, 2002). This zone often defines the limit of biologic conditions for aquatic organisms. During high flow events, secondary flow currents in bends may erode or undercut the base of the bank slope. Undercutting in the toe zone, commonly in bends in meandering rivers, is likely to result in bank failure. (NRCS, 1996; NRCS, 2002; Georgia Environmental Protection Division, 2011). The process of bank failure deposits additional material at the toe that can push erosive flows away from the bank and slow bank erosion at the toe. In higher and medium energy streams where larger rock or a gravel sediment supply is naturally present, the stream stabilizes with a gravel or boulder bed in the toe zone. Alternatively, in forested reaches, medium energy streams may have a stable toe zone due to tree roots, the root balls of shrubs, or woody debris in or impacting the toe zone.

Green Line. A horizontal elevation marks the divide between the frequently wet toe zone and the zone where riparian vegetation can establish on the banks. This is a critical element in planting design and bank stabilization, and can often be easily identified in the field by the distinct change from vegetation to substrate. A green line can even be identified in concrete lined channels from the plants growing in the cracks. During topographic surveys of the site, the green line elevation should be measured and recorded for the entire length of the project. This lowest elevation of vegetation establishment is tied to the series of recent hydrologic events (possibly an annual to 3-yr event) and the longitudinal slope of the green line is related to the channel water surface profile. The green line profile is suspected to shift a small amount vertically, from year-to-year, in response to local climate.

Bank Zone. The bank zone is located between the green line and the crest of the bank. This area can be exposed to erosive river currents, debris movement, and frequent human and animal traffic. The water table is frequently close to the soil surface. While it is generally in a less erosive environment than the toe zone, it is potentially exposed to wet and dry cycles, ice scour, debris deposition, and freeze-thaw cycles. The bank zone is generally vegetated with early colonizing herbaceous species and flexible stemmed woody plants such as willow, dogwood, elderberry, and low shrubs. Small to large trees can also grow on banks depending on the bank geometry and conditions. Like the toe zone, erosive forces are higher than erosive forces in the overbank area, also described as the terrace zones or flood plain. The toe zone, green line, and bank zone are most often within the main channel.

Overbank Area and Terrace Zone(s). The first terrace zone is more commonly described as the overbank area and is located inland from the bank zone and above the bankfull discharge elevation. This typically flat zone may be formed from sediment deposition. It is sporadically flooded, usually about every 2 to 5 years. Vegetation found in this zone is generally flood tolerant and may have a high percentage of hydrophilic plants (water-loving). Shrubby willow with flexible stems, dogwoods, alder, birch, and others may be found in this zone. Larger willows, cottonwoods, and other trees may be found in the upper end of this zone. There can be multiple terraces within the floodplain and the terrace zone can be at varying slopes. Terraces, depending on soils and terrain, may be easily eroded when vegetation is not present. This area is dominated by mature trees, shrubs, and herbaceous species (Georgia Environmental Protection Division, 2011). Multiple terraces will support distinct species of plants ranging from wetland tolerant species to highland arid plants. The variation in riparian plant species is often related to the depth roots can grow to reach the water table. The plants on the highest terraces may rely more on rainfall, and require less moisture through leaf or structure adaptions. It is also common in the west to see mature riparian trees like cottonwood on high terraces above incised creeks. These plants may be 50 to 100 years old and established at that location when there was more frequent wetting from the stream, and prior to a gradual lowering of the water table as the channel incised.

Floodplain. The main channel including the toe zone, green line, bank zones and the active terrace zones/overbank area are all contained within the floodplain. The floodplain is periodically inundated although the outer and highest extent of the flood plain may only be submerged during rare events.

Shown in Figure 9–1 are the NRCS riparian plant zones. The 5 bank design zones are similar to the NRCS planting zones. Terrace zones may be overbank or transitional zones, and the highest terrace may be an upland zone outside the floodplain. Transitional zones in the NRCS figure represent areas where riparian vegetation transitions to upland vegetation that is not dependent on water table depth. The transitional zone represented in the NRCS figure is located between the flood-prone overbank elevation and the upland zone, typically outside the floodplain. This zone may only be inundated every 50 years and is not generally exposed to high velocities except during high-water events. Larger upland species predominate in this zone. Since it is infrequently flooded, the plants in this zone do not need to be especially flood tolerant.

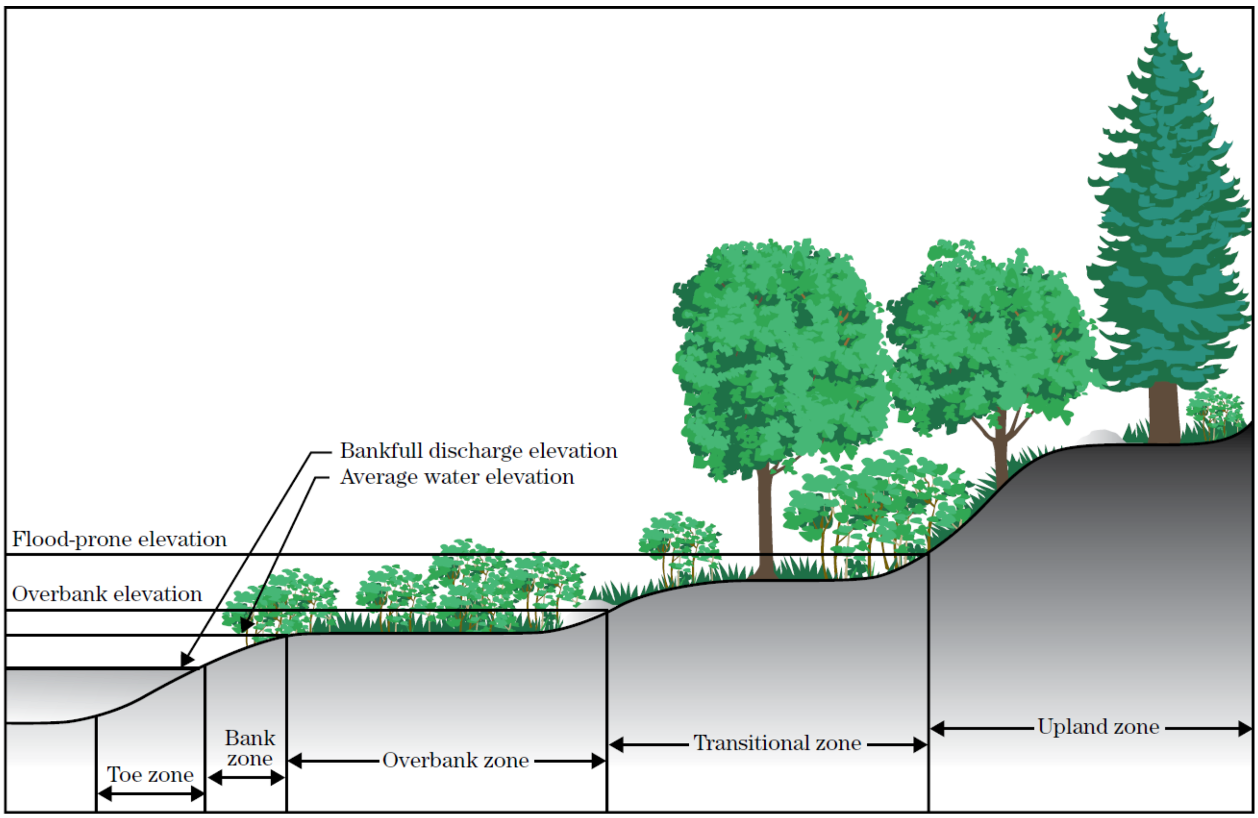


Figure 9–1. Bank zones (NRCS, 2007).

The upland zone is found above the flood prone elevation and is assumed in these guidelines to be generally outside the design floodplain. Erosion in this zone is typically due to overland water flow, wind erosion, improper farming practices, logging, overgrazing and urbanization. Under natural conditions this area is typically vegetated with upland species, not riparian species although mature riparian trees may persist from earlier decades if the channel has incised over time and the roots still reach the water table or rainfall is sufficient to maintain the mature trees.

9.2 Vegetation Bank Protection

Three ways to achieve the riparian vegetation goals are the natural recruitment approach, a planting design, and a planting design that is covered with biodegradable fabric, commonly coir fabric.

9.2.1 Natural Revegetation

When no additional planting is carried out, the river bank in ideal conditions will be colonized by riparian vegetation that is spread by airborne seeds and waterborne seeds or propagules. There can also be dormant seeds present in the ground and topsoil. A vegetation survey of the area can determine if there are mature parent plants of desirable species located sufficiently close to serve as a natural seed source. Vegetation surveys at appropriate periods may also help to determine the germination period for the species of interest. At a minimum, however, construction sites require a reseeding plan to control overland erosion. All disturbed areas are seeded with a mix of native grasses.

An invasive plant control plan should also be integrated with a natural colonization approach since invasive plants can often out-compete native plants after a disturbance like fire, flood, or construction. Invasive plants can be removed mechanically or with chemicals hand sprayed on individual plants or in the cases of extreme coverage, sprayed from an airplane and/or burned. Without an invasive plant control plan, some species can turn the riparian banks into a monoculture. Removing monocultures plants to allow native plants to recover can be costly and time-consuming. During the period of natural plant establishment, and with no other compensatory actions, the banks and exposed areas are susceptible to a worst case condition of erosion.

9.2.2 Planting Vegetation

Planting plans help accelerate the establishment of plants and helps ensure the appropriate species are included. It also reduces the exposure period to erosion. Revegetation consists of selecting the appropriate plant species for the management area and introducing them in a manner which promotes successful establishment. This includes watering until the plant is established, appropriately locating plants on banks and terraces so the water table will be accessible to the plant roots, and selecting appropriate species to match site conditions. Plants may be selected for their habitat value, their bank stabilization attributes, desiccation or inundation tolerance, resistance to invasive plants, or other characteristics. Commonly a cottonwood-willow community is desired in most riparian areas of Reclamation operation. There are, however, local adjustments within this community. In the southwest there may be more honey mesquite (*Prosopis glandulosa*) in place of cottonwood in the riparian areas. In northern California there could be more Oregon Ash (*Fraxinus latifolia*) and elderberry (*Sambucus* *nigra*). Fremont Cottonwood (*Populus fremonti*) and on the upper terraces, Valley Oak (*Quercus lobata*) are common in California. Narrow-leaf Cottonwood (*Populus angustifolia*) may be more common in Colorado and Plains Cottonwood or Eastern Cottonwood (*Populus deltoides*) may populate areas east of the Rocky Mountains. A dominant riparian willow shrub found in many areas of the country is *Salix exigua* and its common names include coyote willow, narrow leaf willow, and sandbar willow. This shrub is a staple of many planting plans.

Upland plants will require more drought tolerant plants that are distinctly different from riparian species. Selection should be based on surrounding upland or non-riparian plant bottoms (USDA, New Mexico Natural Resources Conservation Service, 2007).

There are a number of ways to plant vegetation and each species will have different requirements. The Plant Characteristics section of the USDA PLANTS database, by NRCS, reports how a plant is propagated: bare root, bulbs, container, corm, cuttings, seeds, sod, sprigs, or tubers. It may also list general seed germination periods, dormancy requirements and other factors. Some species are tolerant of most planting methods but others may require a specific approach for successful establishment. More information on planting can be found in The Fact Sheet or Plant Guide for individual species. These products usually contain more information on plants that are commonly used in river work. The following excerpt is from the USDA Plant Fact Sheet for narrow leaf willow (*salix exigua*).

> Planting rooted cuttings or un-rooted cuttings are both effective planting methods. The un-rooted cuttings should be used where moisture conditions are good. On droughty sites, the rooted cuttings are preferred. Plant rooted cuttings using techniques that are common to bare root seedlings. Un-rooted cuttings should be at least 12 inches long, with the lower 10 inches buried vertically in the sand. Plant spacing of 2 x 2 ft to 4 x 4 ft may work well.

Another valuable resource for individual plant species are the Fire Effects Information System papers from the US Forest Service. These papers are summaries of published material and contain a wealth of references.

Riparian zones are best established in reaches where there will be periodic inundation to maintain healthy coverage but not continual inundation. The toe zone with an upper border of the green line is normally devoid of riparian vegetation and marks the lowest elevation for planting plans. Below the green line, vegetation only provides bank stabilization through mature roots for a short depth of 1 to 3 ft. An upland plant zone borders the highest elevation of riparian plants. Grasses that do not rely on the groundwater table for moisture can provide sufficient erosion resistance in the highest terrace of the floodplain. Erosive forces in the bank zone can be reduced to a level that can be protected by vegetation through grading banks back to a flatter slope. The large benefit is future sustainable erosion protection, but the disadvantage is the loss of existing mature plants during slope grading. In some cases where existing vegetation needs to be preserved, it may be possible to install a permanent stone toe inside of a vegetated bank line. The stone toe can interrupt large secondary, circulating flow patterns, reducing the scour that occurs at the toe and up the banks.

9.2.3 Planting Vegetation with Coir Fabric

Young plants offer little resistance to erosion in the first 2 to 3 years. If a high flow occurs in this period after planting, there is a good chance that plants will be lost. Two of the approaches that can be taken for this initial period of higher risk following construction are to include in the project budget funding for a second planting, or to add a protective fabric cover to help protect banks until young plants establish and grow to a size that can help prevent erosion.

The lifespan of the biodegradable coir fabric depends on the amount of wetting and drying and exposure to sunlight, and it generally ranges from 3 to 5 years. The properties of bioengineered fabrics are documented by Hoitsma (1999) and are extracted from FHWA HEC15.

> Manufacturers have developed a variety of rolled erosion control products (RECPs) for erosion protection of channels. These products consist of materials that are stitched or bound into a fabric. Table 9–1 summarizes the range of RECP linings that are available from the erosion control industry. Selection of a particular product depends on the overall performance requirements for the design. RECPs offer ease of construction in climate regions where vegetation establishes quickly.

Table 9–1. Manufactured (RECP) Linings

From *Design of Roadside Channels with Flexible Linings*, Hydraulic Engineering Circular Number 15, Third Edition, Chapter 5: Manufactured (RECP) Lining Design (Kilgore and Cotton, 2005)

Type	Description
Open-Weave Textile	A temporary degradable RECP composed of processed natural or polymer yarns woven into a matrix, used to provide erosion control and facilitate vegetation establishment. Examples: jute net, woven paper net, straw with net.
Erosion Control Blankey	A temporary degradable RECP composed of processed natural or polymer fibers mechanically, structurally or chemically bound together to form a continuous matrix to provide erosion control and facilitate vegetation establishment. Example: curled wood mat.
Turf-reinforcement Mat (TRM)	A non-degradable RECP composed of UV stabilized synthetic fibers, filaments, netting and/or wire mesh processed into a three-dimensional matrix. TRMs provide sufficient thickness, strength and void space to permit soil filling and establishment of grass roots within the matrix. Example: synthetic mat.

Listed in Table 9–2 are values for resistance of live stakes with and without a coir fabric. More information on planting live stakes is presented in a later section.

Table 9–2. Erosion Resistance of Live Stakes with and Without Protective Fabrics (Sotir and Fischenich, 2007)

A. Live Stakes in Bare Soil Before Established		
Soils	**Velocity, ft/sec**	**Shear, lb/ft**
Silts	.05	.001
Sands	.5	.01
Large Gravel	2	.5
Large Cobble	4	2
Firm Loam	2.5	.08
Stiff Clays	3–4	.25
12” Rounded Riprap	6	4

B. Live Stakes with Erosion Control Fabrics Prior to and After Establishment		
Fabric	**Velocity, ft/sec**	**Shear, lb/ft**
Jute		
Before Est.	1–2.5	.45
After Est.	3–7	2.1–3.1
Woven Coir – 700gm wt.		
Before Est.	3–5	2–2.5
After Est.	3–10	2.1–3.1

9.2.4 Design Procedure

At this point, project design criteria, ecological factors, hydrology, geomorphic factors and general hydraulic and scour factors have been assembled previously to aid in selecting a suitable bank stabilization method. It is assumed that this information is available now as a resource to the design. Important aspects of design criteria, hydrology, and permitting are described in chapter 2. Determine the geomorphic factors including sediment and sediment continuity issues that may impact channel stability as discussed in chapter 3. Define general hydraulics including energy (chapter 4) and potential for scour (chapter 5) that may influence the channel and the extent of the floodplain. Steps of the design will recommend revisiting some of these investigations, to obtain more detailed information for this design.

Step 1. Determine erosive forces. When designing with vegetation, similar to designing with inert materials, design to a safe percentage of the maximum strength of the material. Determining whether vegetation will be sufficient to protect a bank from erosion is dependent on the erosive forces or velocities acting on the bank (chapter 4). Using 1D or 2D flow models (SRH-1D, HEC-RAS, SRH-2D), or other means, estimate the velocity or shear force acting on the bank of the channel. Velocity values from 1D models need correction based upon the river radius of curvature. A less accurate approach for estimating the erosive force of secondary flow in a bend is to double the average flow velocity in a river section. The same rule can be applied in scour studies for estimating velocity of the diving flow in a roller at the nose of a pier.

Assessment of river form and energy level of the river, as discussed in chapter 4, will also help in this analysis. In nature, vegetation is the main form of erosion control but it is most effective when the channel banks are laid back. Vertical banks in bends are normally stripped of vegetation since steep banks in bends are locations of high erosive energy. Naturally steep banks and channels with large depths and low width- to- depth ratios are more difficult to protect since they are subject to a higher erosive force.

Step 2. Compare the erosive force to the erosion resistance of vegetation. The erosive forces acting on the river bank are compared with erosion resistance values for similar plant species and soils. There is a limited availability of resistance values for plant species and the resistance values can vary with plant age, height, coverage, vigor, density, soils, and local climate. Also the erosive forces are not equal for all locations along the bank and at all vertical locations up the bank, so this approach is not as direct as the same analysis using traditional construction materials, but the availability of these values is improving. A valuable guide on the erosion resistance of plants is presented in Figure 9–2 (Gray and Sotir, 1996). For flow duration of 3 to 5 hours, the limiting velocity would be 1.5 to 3 m/s. Additional velocity data is shown in Table 9–3 from Chen and Cotton (1988), also found in Gray and Sotier (1996). Often the erosion resistance for one species and soil has to be estimated from soils and plant species with a similar root and stem structure.

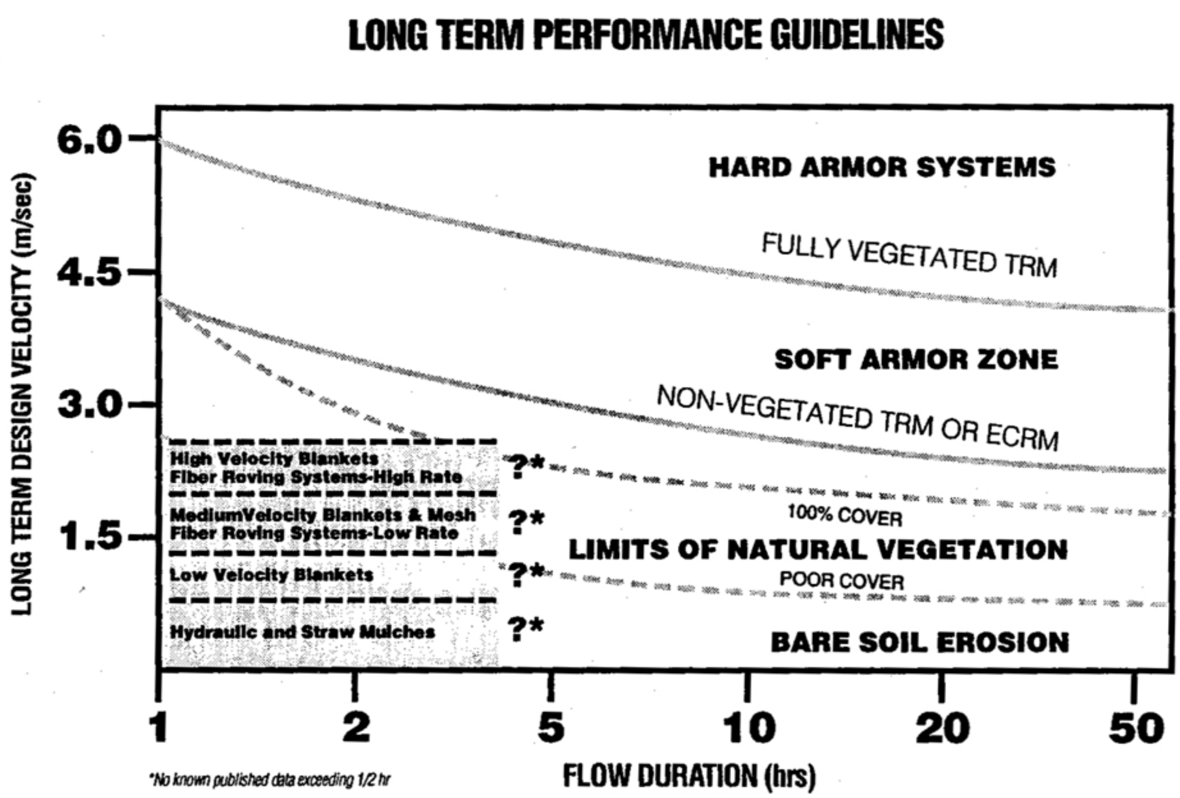


Figure 9–2. Limiting velocities for erosional resistance of vegetation (Gray and Sotir, 1996).

Table 9–3. Permissible Shear or Tractive Stresses for Selected Lining Materials

From Chen and Cotton (1988). Note: Non-degradable rolled erosion control products (RECP's) are not recommended for most river projects due to a natural tendency for stream and river banks to shift.

Lining Category	Lining Type	Permissible Unit Shear Stress	
		(psf)	(N/m2)
Temporary degradable RECPs	Jute net	0.45	22
	Straw with net:		
	Single net	1.55	74
	Double net	1.65	79
	Coconut fiber with net	2.25	108
	Fiber glass roving	2.00	96
Long-term non-degradable RECPs	Synthetic mats:		
	Un-vegetated	3.00	144
	Partially established	4.0 → 6.0	192 → 288
	Fully vegetated	8.00	384
	Three-dimensional woven geotextiles	10.00	480
Vegetative	Class A	3.70	178
	Class B	2.10	101
	Class C	1.00	48
	Class D	0.60	29
	Class E	0.35	17
Gravel riprap	1-inch	0.33	16
	2-inch	0.67	32
Rock riprap	6-inch	2.00	96
	12-inch	4.00	192

In table 9–3 vegetative cover permissible shear stress are for grass species as reported for Classes A through E by Chen and Cotton (1988). Class A vegetation are excellent stands of grass species with average height of 30 to 36 in. Examples of class A grasses are weeping love grass, and yellow bluestem ischaemum, Class B are dense grasses on the order of 12 to 24 in tall. Example of Class B grasses include kudzu, bermuda grass, alfalfa, and blue gamma. Class C grasses are typically 6 to 12 in in height. Examples are crab grass, common lespedeza legume mixture and kentucky bluegrass. Class D usually range in height from 3 to 6 in and include bermuda grass, buffalo grass, grass legume mixture, and lespedeza sericea. Class E is usually less than 2 in and includes bermuda grass. high Additional information about the classification of grass vegetative covers can be found in FHWA (2005), or Chen and Cotton (1988).

Step 3. Determine greenline elevations and groundwater regime. Survey the elevation of the greenline for the longitudinal length of the project. Determine the most frequent groundwater elevation and the range in groundwater elevation values. The riparian planting zone can extend up from the green line to the top of bank (mean annual peak flow or Q_2) and higher. This is usually the riparian zone and plantings should be local riparian plants.

Step 4. Investigate the need for a stable or degradable toe, or a hard submerged bench. Determine the period of time a stable toe for the bank is needed. Guidelines on designing a degradable toe are presented in a succeeding section. If a more permanent, erosion resistant foundation for the river bank is required, consult the guidelines for a stone toe design are available from sections 13.2, 13.3 and 13.5.

Step 5. Prepare a plant design for the riparian zone. Each species of riparian plants have their own specific niche. Consider the planting seasons for the project location when installing vegetation. Determine the correct elevation and location for the plants. The green line is the lowest elevation for establishing riparian plants.

Biodegradable filter fabric, often coir fabric constructed from coconut husks, is used and added after the bank is graded. The filter is fastened or “toed in” a minimum of 12 inches. Plants can be installed through the fabric by cutting slit openings and placing the plants directly in the bank substrate. The coir fabric prevents soil erosion for 3-5 years before the fabric begins to break down. Vegetation should have sufficient time to establish and provide bank stabilization before the coir fabric biodegrades. The re-establishment of vegetation can be accomplished by seeding, installing plant cuttings, rootwads, bare root or containerized specimens. For locations with larger erosive forces, fabric encapsulated lifts (FES) may be more effective at helping establish plants.

Rehabilitation sites built between 2005 and 2009 on the Trinity River have had little success germinating and establishing riparian hardwoods on surfaces above 4,500 cfs (McBain and Trush, 2006; McBain and Trush, 2007; USFWS et al., in prep). Cottonwood seedlings are very susceptible to desiccation and take about five days from germination to primary root formation (Schriener, 1974; Young and Young, 1992; Pregitzer and Friend, 1996). Even after five days, the young root must still grow rapidly to find a perennial source of water. Field studies at rehabilitation sites indicate that woody riparian plants currently initiate along the channel and on surfaces where the ground is saturated for 34 days. Studies of seedling mortality relationships to groundwater drawdown have shown that seedlings must be 18-21 days old before beginning ground water recession in order to survive (Stella and Battles, 2010). Therefore, future design surfaces where natural riparian regeneration is desired should target a minimum of 21 consecutive days of saturation during Extremely Wet, Wet, and Normal water year types.

The broad TRRP goal for riparian vegetation is to “establish and maintain riparian vegetation on different geomorphic surfaces that contributes to complex channel morphology and high quality aquatic and terrestrial habit” (TRRP and ESSA 2009). Physical designs are one tool that the TRRP employs to rehabilitate the channel and improve the structural quality of riparian vegetation along the mainstem Trinity River. A primary component of physical channel rehabilitation design is to “prevent riparian vegetation from exceeding thresholds (that) lead to encroachment that simplifies channel morphology and degrades aquatic habitat quality” (TRRP and ESSA, 2009). Wherever channel rehabilitation activities impact existing riparian vegetation, the TRRP is required to “recover riparian vegetation area equal to or greater than that disturbed by physical rehabilitation” (TRRP, 2008). The logic supporting the overall programmatic riparian vegetation goals and objectives is detailed in the Integrated Assessment Plan (TRRP and ESSA, 2009).

Step 6. Determine a planting density for each species. Consult planting guidelines to determine planting densities for the project area. Tree planting density may be 6 to 15 ft or 10 ft average on center, and shrub spacing may be in the range of 4 to 6 ft on center. Applications of bioengineering techniques can be site specific to a location with variation typical for germination seasons, growing seasons, soils, and groundwater level fluctuations. Valuable resources include local native plant experts, Fish and Wildlife offices, and Natural Resources Conservation Offices (NRCS) to determine recommended planting densities and plants to be applied at the site.

Step 7. Prepare a plant design for the upland zone. Plants that have a low tolerance of frequent wetting and require less water are suitable for the upland areas of the floodplain.

Step 8. Assess constructability. Investigate a supply for the age and type of plants that are needed. Determine if the supplier can provide a specified number of plants during the construction period when planting will occur. If the plants require irrigation, design an irrigation plan. Determine the level of risk for establishing plants and consider if the coir fabric will provide enough protection for this period. If there is a high level of risk for the plants to be disturbed before they are fully established, consider reserving sufficient funding to re-plant in the following year. Include in specifications detailed information on when the plants can be stored on site, watering and shade requirements for on-site storage. Also include detailed specifications on planting techniques.

Step 9. Plan active monitoring. Inspections to determine vegetation loss should occur after each major flood, or at least annually for the first 3 years, or until the vegetation is well established. Bioengineering projects become more erosion resistant with time, once any bed degradation is controlled, toe scour and erosion is arrested, and plants become established (Allen and Leech, 1997). The key to success is to ensure that there is early-on establishment of plants, establishment of monitoring, and a plan to replant or remediate if high, erosive flows occur. Monitoring should focus on signs of plant survival and development of the stream bank integrity (Allen and Leech, 1997).

Remedial planting may be required within the first year or two due to drought conditions, or high than normal peak flows, or longer levels of inundation than plans can tolerate. Replacement of bio-degradable fabrics, or bioD blocks sometimes used for FES lifts may be required if they have washed away during high flows. Ocular evaluation and description of bioengineering and stream bank conditions accompanied by photographs can document processes such as: slumping from geotechnical failure, riling, gullying, toe undercutting or launching, flanking at upper or lower ends of treatment and scouring along the bankline. These same descriptions can be made of plant survival and health, signs of beaver, rodent or other mammal effect.

9.2.5 Discussion and Recommendations

The design and implementation of riparian zone restoration and management is given in Saldi-Caromile et al. (2004). Guidelines for planning riparian restoration in the Southwest can be found at
<http://www.nm.nrcs.usda.gov/news/publications/riparian.pdf> and
<http://www.nm.nrcs.usda.gov/news/publications/dormant-willow-planting.pdf>
(NRCS accessed 2012).

Risk and Failure. There may be some increase in water surface elevation as a result of increased hydraulic roughness; however, for bankline bio-engineering these effects are generally small. Failure can occur if flooding occurs after planting in the spring. High flows can exceed the maximum resistance of plants particularly in the first few years while vegetation is becoming established. Vegetation established on the banks can introduce more woody debris to the channel.

Products including hay bales, coir logs and bioD blocks are bulky and catch higher flow forces. These materials, if used (coir logs are on the constructed materials blacklist and are not recommended for most projects), need to be restricted to locations with low flow velocity, inset and well anchored, or else include an active maintenance program.

If not planted correctly, not planted at correct elevations, or not adequately watered, the plants can die. Small and large mammal browsing, beaver harvesting, trampling or rubbing by livestock, deer or elk can limit establishment of planted vegetation. Physical factors such as excessively low or high groundwater table or inundation levels and vandalism may also limit establishment of planted vegetation.

This method can be used to provide long-term bank stability under specific circumstances. Vegetation is useful for stabilizing soils to the depth of the root zone. Only aquatic plants should be used below the mean low water elevation.

Monitoring and Maintenance. Monitoring and maintenance will vary based on the stream system and associated parameters such as flood frequency, flood stage and timing (Sotir and Fischenich, 2007).

Inspections should also be made to determine trampling by humans and animals or plant removal by animals and waterfowls (Sotir and Fischenich, 2007). Maintenance can include re-planting where necessary.

9.3 Live Staking/Pole Planting

Establishing plants by live staking, also described as pole planting, has different requirements from plants established from seeds, seedlings or potted plants and is treated separately here. Not all plants can be established from live stakes but willows that provide sturdy erosion protection and spread rapidly, are particularly well-suited. Although willows can be planted from seed, they root so readily from stem or root cuttings other methods are usually not used. Live staking/pole planting is a deceptively simple approach to establishing willow plants on river banks and the floodplain. Stakes, or the stems of willow plants, are harvested in the field and inserted in holes at the planting site. A guiding principal is that roots of willow must have access to water. Good site selection greatly increases the chance for project success, and poor site selection can end the project as a wasteland of dead sticks.

A site suited to live stake or pole planting treatments requires a hydrologic regime that:

1) keeps the stake wet during most of the growing season where the establishment of woody plants are desirable (Sotir and Fischenich, 2007),

2) allows the roots to reach the water table or vadose zone during most of the growing season, and

3) sustains flows sufficient to keep woody plants growing well but not large and long-duration of flows so as to exceed the plant’s flood tolerance.

Streams that are best suited have perennial flows and are small to moderate in size. A second important factor is choosing a site that is not subject to massive amounts of sediment deposition (Sotir and Fischenich, 2007).

9.3.1 Design Procedure

The USDA Plant Guide on coyote willow (salix exigua) describes the plant as rooting freely from cuttings and as an easy species to propagate:

> Coyote willow is a shrub 3 to 15 ft in height with multiple branches and deciduous leaves. Its architecture is resilient to disturbance such as high

velocity floodwaters, sediment deposition, medium to high flooding (anoxic conditions), high winds, heavy precipitation, or pruning from beaver, deer or wildlife. Beaver browsed more than 5,000 willow cuttings to ground level in New Mexico, and all the willow resprouted (Los Lunas Plant Materials Center 1998). These cutting also survived over two months of continuous inundation.

The following pole planting design procedure is based on the guidelines for willow and cottonwood from the NRCS Plant Materials Center at Los Lunas in cooperation with the U.S. Fish and Wildlife Service (USDA, NRCS).

At this point, project design criteria, ecological factors, hydrology, geomorphic factors and general hydraulic and scour factors have been assembled previously to aid in selecting a suitable bank stabilization method. It is assumed that this information is available now as a resource to the design. Important aspects of design criteria, hydrology, and permitting are described in chapter 2. Determine the geomorphic factors including sediment and sediment continuity issues that may impact channel stability as discussed in chapter 3. Define general hydraulics including energy (chapter 4) and potential for scour (chapter 5) that may influence the channel and the extent of the floodplain. Steps of the design will recommend revisiting some of these investigations, to obtain more detailed information for this design.

Step 1. Select a harvest site and a planting site. Try to select sites that are close to each other to conserve the genetic diversity of the plant. Conditions at the project site should match conditions at the donor site including soils, elevation, hydro-dynamics, permanent groundwater table, and soil salinity. Soil salinity should be low.

Step 2. Monitor ground water levels. Prior to planting, monitor the water tables at proposed planting sites for at least one year to determine minimum groundwater elevations. Take a reading at least once a month, and more often during the driest months of the year. Poles planted where the water table fluctuates widely will have lower survival rates than those planted where the water table is relatively stable. If groundwater monitoring shows the water level will drop more than 3 ft during the growing season (May-October), another site should be selected, or, alternative methods considered such as tube seedlings with irrigation. Monitoring of observation wells for at least one calendar year before planting will help determine the best planting depth to ensure establishment (USDA, NRCS).

Step 3. Develop design, drawings and specifications. Include in the specifications all care related to harvesting and planting the poles and care of the poles while at the construction site. A description of what should be included in the design is addressed in steps 5-9.

Step 4. Review constructability. Review plans and designs for constructability issues and to ensure all aspects of plant handling are covered in specifications. Develop the monitoring plan in this step or in the previous step.

Step 5. Control invasive plants on site. Salt cedar (Tamarix chinensis), Russian olive (Eleagnus angustifolia), and giant reed (Arundo donax) should be controlled before poles are planted. However, young cottonwoods and willows can grow successfully in quite small openings in stands of salt cedar. Study of natural stands suggests they will eventually shade out the salt cedar (USDA, NRCS).

Step 6. Select and prepare willow cuttings. In previous sections, Table 9–2 and 9–3 and Figure 9–2 indicate a jute or coir fabric covering of the ground around the planted cuttings will increase the shear resistance to flow of the newly planted cuttings. Willows are commonly used. Willow (or cottonwood) cuttings from the donor site should be from a healthy, native stand. Select vegetation species from Table 9–4. Cut poles while the plant is dormant in January and February. Leave at least 1/3 of plants in an area. Willow cuttings should be at least 1/2 inch in diameter. Select the longest, straightest poles available and use only two to four-year old plants. The length of each pole depends upon the depth to groundwater. Strip most side branches from the poles but leave the top two or three. Trim off the terminal bud on top. This allows most of the energy in the stem to go to rooting and buds in branches. Soak poles for a minimum of 5 to 7 days before planting.

Trial plantings on well adapted sites indicate more than 80% survival of cottonwood and willow poles when dormant poles are cut and planted between November and February (USDA, NRCS).

Step 7. Planting poles. The USDA, NRCS Plant Guide for coyote willow provided the following instructions on planting poles:

> Dig holes to the depth of the lowest anticipated water table. Sites where the water table will be within one foot of the ground surface during the growing season are better suited for willows than cottonwoods. The cuttings should extend several inches into the permanent water table to ensure adequate moisture for sprouting. At least 1/2 to 2/3 of the cutting should be below ground to prevent the cutting from being ripped out during high water flows. Usually, at least 2 to 3 ft should be below ground. It should also be long enough to emerge above adjacent vegetation such that it will not be shaded out. Place cuttings in the hole the same day they are removed from the soak treatment. Set the butt as close to the lowest annual water table elevation as possible. Electric hammer drills (Dewalt model DW530) fitted with one-inch diameter, 3-foot bits were used to plant thousands of coyote willows in New Mexico. With one drill, two people installed 500 willows per day to a 3-foot depth. A power auger or a punch bar can also be used. Coyote willow pole cuttings were generally planted on 10 to 20 foot centers

in New Mexico. Areas with a shallow water table (4-6 ft) were generally planted with a higher number of pole cuttings to enhance overall survival of the project; in this case, coyote willow was planted on 1-foot centers or even closer. Often understory species were planted under the canopy of pre-existing overstory (cottonwoods, tree willows) since they are often observed occupying this niche. It is critical to ensure the soil is packed around the cutting to prevent air pockets. "Mudding" (filling the hole with water and then adding soil to make mud slurry) can remove air pockets. When necessary, install tree guards around the poles to protect from beavers, other rodents, or rabbits. Coyote willows tend to be fairly resistant to pruning from beavers, so tree guards may not be necessary. Fence off the area from grazing for 2 to 3 years.

In New Mexico willow poles have been driven up to 8 ft deep to intersect ground water (USDA, New Mexico Natural Resources Conservation Service, 2007). Another source suggests specimens can be 3 in to 20 ft long, the bottom of cutting should be cut at a 45-degree angle, live cuttings should be taller than surrounding vegetation, live cuttings should be placed on 2-4 ft centers and dormant post planting are installed in a similar manner to live cuttings.

Step 8. Clean buds. In April or May, clean the buds on the lower 2/3 of the pole, as they begin to swell. This helps reduce evapotranspiration water loss and stimulate root growth.

Step 9. Manage flows for plant establishment. Salix and cottonwood plants share similar establishment and growth traits 1) flood flows that precede Salix seed dispersal produce suitable germination sites; 2) flow recessions following a peak expose germination sites and promote seedling root elongation; and 3) base flows supply soil moisture to meet summer and winter seedling water demand (Shafroth et al. 1998; Mahoney et al. 1998). The combination of root growth and capillary fringe defines the successful recruitment band for seedling establishment, which is usually from about 0.6 to 2 m in elevation above the late summer stream stage (Mahoney et al. 1998). The rate of stream stage decline is also critical for seedling survival and should not exceed 2.5 cm per day. If flow can be managed, declines in the water surface, and presumably in the water table, should not exceed this rate.

Step 10. Monitor. Monitor pole planting success annually or after major flow events, for several years. Also review harvest site for impacts.

Table 9–4. Woody Plants with Fair to Good Ability to Root from Dormant, Unrooted Cuttings and Their Soil Bioengineering Applications

Note: States are identified at the end of the table

Scientific name	Common name and cultivars*	Procure from	Region of adaptation**	Rooting ability	Soil bioengineering technique
Species with very good to excellent rooting ability from live hardwood material					
Populus balsamifera	Balsam poplar	Local collections	1,2,3,4,5,8,9,0,A	Very good	Live cuttings, poles
Populus deltoids	Eastern cottonwood	Local collections	1,2,3,4,5,6,7	Very good	Poles, live cuttings
Populus balsamifera ssp. *trichocarpa*	Black cottonwood	Local collections	4,8,9,0,A	Very good	Poles, live cuttings
Salix alaxensis	Feltleaf willow	Local collections	A	Very good	Poles, live cuttings
Salix amygdaloides	Peachleaf willow	Local collections	1,2,3,4,5,6,7,8,9	Very good	Poles, posts, live cuttings
Salix barclayi	Barclay's willow	Local collections	A	Very good	Poles, posts, live cuttings
Salix brachycarpa	Barren Ground willow	Local collections	A	Very good	Poles, posts, live cuttings
Salix boothii	Booth's willow	Local collections	7,8,9,0	Excellent	Fascines, poles, brush mattress, brush layering, live cuttings
Salix cottetii	Bankers' Dwarf willow (cultivar)	Nursery	Introduced 1,2,3	Very good	Fascines, brush mattress, brush layering, live cuttings
Salix discolor	Pussy willow	Local collections	1,2,3,4,9	Very good	Fascines, poles, brush layering, live cuttings
Salix drummondiana	Drummond's willow	Local collections	7,8,9,0	Very good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix interior	'Greenbank' Sandbar willow (cultivar)	Nursery	1,3,4,5	Excellent	Fascines, poles, brush mattress, brush layering, live cuttings
Salix interior	Sandbar willow	Local collections	1,3,4,5	Very good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix melanopsis	Coyote willow (green stem)	Local collections	8,9,0	Excellent	Fascines, poles, brush mattress, brush layering, live cuttings
Salix eriocephala	Missouri River willow	Local collections	7,8,9,0	Very good	Fascines, poles, brush layering, live cuttings
Salix fluviatilis	'Multnomah' River willow (cultivar)	Nursery	9 (Coast only)	Excellent	Fascines, poles, brush mattress, brush layering, live cuttings
Salix fluviatilis	River willow	Local collections	9	Very good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix geyeriana	Geyer willow	Local collections	7,8,9,0	Very good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix gooddingii	'Goodding's willow	Local collections	6,7,8,0	Very good	Poles, posts, live cuttings
Salix hookeriana	Clatsop' Hooker willow (cultivar)	Nursery	9, 0 (Coast only)	Excellent	Fascines, poles, brush mattress, brush layering, live cuttings

Table 9–4. Woody Plants with Fair to Good Ability to Root from Dormant, Unrooted Cuttings and Their Soil Bioengineering Applications—Continued

Scientific name	Common name and cultivars*	Procure from	Region of adaptation**	Rooting ability	Soil bioengineering technique
Salix hookeriana	Hooker willow	Local collections	9,0	Very good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix laevigata	Red willow	Local collections	7,8,9,0	Very good	Poles, live cuttings
Salix lasiolepis	‘Rogue’ Arroyo willow (cultivar)	Nursery	9,0	Excellent	Fascines, poles, brush mattress, brush layering, live cuttings
Salix lasiolepis	Arroyo willow	Local collections	6,7,8,9,0	Very good	Poles, live cuttings
Salix lemmonii	‘Palouse’ Lemmon’s willow (cultivar)	Nursery	8,9,0	Very good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix lemmonii	Lemmon’s willow	Local collections	8,9,0	Very good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix eriocephala spp. *ligulifolia*	‘Placer’ Erect willow (cultivar)	Nursery	9,0 (Coast only)	Excellent	Fascines, poles, brush mattress, brush layering, live cuttings
Salix ligulifolia	Strapleaf willow	Local collections	8,9,0	Very good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix lucida ssp. *lasiandra*	‘Nehalem’ Pacific willow (cultivar)	Nursery	9 (Coast only)	Very good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix lucida ssp. *lasiandra*	Pacific willow	Local collections	7,8,9,0,A	Excellent	Poles, live cuttings
Salix pentandra	‘Aberdeen Selection’ Laurel willow (cultivar)	Nursery	Introduced 8,9,0	Excellent	Poles, live cuttings
Salix purpurea	‘Streamco’ Purpleosier willow (cultivar)	Nursery	Introduced 1,2,3	Excellent	Fascines, brush mattress, brush layering, live cuttings
Salix sericea	‘Riverbend Germplasm’ Silky willow (cultivar)	Nursery	1,2,3	Excellent	Fascines, poles, brush mattress, brush layering, live cuttings
Salix sericea	Silky willow	Local collections	1,2,3	Very good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix sitchensis	‘Plumas’ Sitka willow (cultivar)	Nursery	9,0 (Coast only)	Very good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix sitchensis	Sitka willow	Local collections	9,0,A	Very good	Fascines, brush mattress, brush layer
Sambucus nigra ssp. *anadensis*	Common elderberry	Local collections	1,2,3,4,5,6,8,0,A	Very good	Fascines, brush mattress, brush layering, live cuttings

Table 9–4. Woody Plants with Fair to Good Ability to Root from Dormant, Unrooted Cuttings and Their Soil Bioengineering Applications—Continued

Scientific name	Common name and cultivars*	Procure from	Region of adaptation**	Rooting ability	Soil bioengineering technique
Species with fair to good rooting ability from live hardwood material					
Baccharis pilularis	'Coyote' brush	Local collections	7,9,0	Fair	Fascines, brush mattress, brush layering, live cuttings
Baccharis salicifolia	Mule's Fat	Local collections	6,7,8,0	Fair	Fascines, brush mattress, brush layering, live cuttings
Cephalanthus occidentalis	'Keystone' Common Button-bush (cultivar)	Nursery	1,2,3,5,6,7,0	Good	brush mattress, brush layering, Fascines
Cephalanthus occidentalis	Common buttonbush	Local collections	1,2,3,5,6,7,0	Fair	brush mattress, brush layering, Fascines
Cornus amomum	'Indigo' Silky dogwood (cultivar)	Nursery	1,2,3,4,5,6	Good	Fascines, brush mattress, brush layering, live cuttings
Cornus amomum	Silky dogwood	Local collections	1,2,3,4,5,6	Fair	Fascines, brush mattress, brush layering, live cuttings
Cornus sericea	'Ruby' Redosier dogwood (cultivar)	Nursery	1,3,4,5,7,8,9,0,A	Good	Fascines, brush mattress, brush layering, live cuttings
Cornus sericea	Redosier dogwood	Local collections	1,3,4,5,7,8,9,0,A	Fair	Fascines, brush mattress, brush layering, live cuttings
Cornus sericea ssp. *occidentalis*	'Mason' Western Redosier dogwood (cultivar)	Nursery	9,0 (Coast only)	Good	Fascines, brush mattress, brush layering, live cuttings
Cornus sericea ssp. *occidentalis*	Western Redosier dogwood	Local collections	9,0,A	Good	Fascines, brush mattress, brush layering, live cuttings
Lonicera involucrate	Black Twinberry	Local collections	3,7,8,9,0,A	Fair	Fascines, brush layering, live cut- tings
Philadelphus lewisii	'Lewis' Mock-orange	Local collections	9,0	Fair	Fascines, live cuttings
Physocarpus capitatus	Pacific ninebark	Local collections	9 (Coast only)	Fair	Fascines, brush layering, live cut- tings
Physocarpus opulifolius	Common ninebark	Local collections	1,2,3,4,5	Fair	Fascines, brush mattress, brush layering, live cuttings
Populus angustifolia	Narrowleaf cottonwood	Local collections	4,5,6,7,8,9,0	Fair	Poles, live cuttings
Populus fremontii	Fremont cottonwood	Local collections	6,7,8,0	Fair	Poles, live cuttings
Rubus spectabilis	Salmonberry	Local collections	8,9,0	Fair	Fascines, live cuttings
Salix alba	White willow	Local collections	introduced 1,2,3,4	Fair	Poles, posts, live cuttings
Salix bebbiana	Bebb willow	Local collections	1,3,4,5,7,8,9,0,A	Fair	Poles, live cuttings
Salix humilis	Prairie willow	Local collections	1,2,3,4,5,6	Fair	Fascines, poles, brush mattress, brush layering, live cuttings
Salix drummondiana	'Curlew' Drummond's willow (cultivar)	Nursery	7,8,9,0	Good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix drummondiana	Drummond's willow	Local collections	7,8,9,0	Fair	Fascines, poles, brush mattress, brush layering, live cuttings

Table 9–4. Woody Plants with Fair to Good Ability to Root from Dormant, Unrooted Cuttings and Their Soil Bioengineering Applications—Continued

Scientific name	Common name and cultivars*	Procure from	Region of adaptation**	Rooting ability	Soil bioengineering technique
Salix exigua	'Silvar' Coyote willow (cultivar)	Nursery	6,7,8,9,0,A	Good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix exigua sp interior	Sandbar willow (grey stem)	Local collections	6,7,8,9	Fair	Fascines, live cuttings, poles, brush mattress, brush layering
Salix lucida	Shining willow	Local collections	1,3,4,5,7,8,9,0,A	Fair	Fascines, poles, brush mattress, brush layering, live cuttings
Salix lutea	Yellow willow	Local collections	4,5,7,8,9,0	Fair	Fascines, poles, brush mattress, brush layering, live cuttings
Salix nigra	Black willow	Local collections	1,2,3,5,6	Fair	Fascines, poles, brush mattress, brush layering, live cuttings
Salix planifolia	Plainleaf willow	Local collections	1,3,4,5,7,8,9,0,A	Fair	Fascines, poles, brush mattress, brush layering, live cuttings
Salix prolixa	'Rivar' Mackenzie's willow (cultivar)	Nursery	8,9,0,A	Good	Poles, live cuttings
Salix prolixa	Mackenzie's willow	Local collections	8,9,0,A	Fair	Poles, live cuttings
Salix scouleriana	Scouler's willow	Local collections	9,0 (Coast only)	Fair	Fascines, poles, brush mattress, brush layering, live cuttings
Sambucus racemosa	Red elderberry	Local collections	9	Fair	Brush layering, live cuttings
Spiraea douglasii	'Bashaw' Douglas Spirea (cultivar)	Nursery	9 (Coast only)	Fair	Fascines, brush mattress, brush layering, live cuttings
Spiraea douglasii	Douglas Spirea	Local collections	0,9	Fair	Fascines, brush mattress, brush layering, live cuttings
Symphoricarpos albus	Common snowberry	Local collections	9 (Coast only)	Fair	Fascines, brush mattress, brush layering, live cuttings
Caribbean area					
Batis maritima	Barilla, Saltwort	Local collections	C,H	Good	Brush mattress, brush layering, live cuttings
Bucida buceras	úcar, gregre	Local collections	C	Fair	Poles, live cuttings
Bursera simaruba	almácigo, turpentine tree	Local collections	C	Good	Poles, live cuttings
Clusia rosea	Cupey	Local collections	C,H	Good	Pole, live cuttings
Commelina ssp.	Cihítre	Local collections	C	Good	Brush mattress, brush layering, live cuttings
Cordia sebestenea	Vomitel, geiger tree	Local collections	C,H	Fair	Poles, live cuttings
Erythrina poeppigiana	Bucayo, bucare, mountain immortale	Local collections	C,H	Good	Poles, live cuttings
Glyricidia sepium	Mata ratón, Glyricidia	Local collections	C,H	Good	Poles, live cuttings

Table 9–4. Woody Plants with Fair to Good Ability to Root from Dormant, Unrooted Cuttings and Their Soil Bioengineering Applications—Continued

Scientific name	Common name and cultivars*	Procure from	Region of adaptation**	Rooting ability	Soil bioengineering technique
Hibiscus spp.	Hibiscos	Local collections	C,H	Good	Fascines, poles, brush mattress, brush layering, live cuttings
Hymenocallis caribaea	Lirio blanco, Spyder lilly	Local collections	C	Fair	Brush mattress, brush layering, live cuttings
Lagerstroemia indica	Astromelia	Local collections	C	Good	Fascines, poles, brush mattress, brush layering, live cuttings
Mangrove species	(Rhizophora, Avicenia, Cono-carpus)	Local collections	C,H	Good	Pole, live cuttings
Nicolaia elatior	Flor de cera, Torch ginger	Local collections	C	Fair	Brush mattress, brush layering, live cuttings
Pictetia aculeata	Fustic	Local collections	C	Fair	Fascines, poles, brush mattress, brush layering, live cuttings
Rhoeo spathacea	Sanguinaria	Local collections	C,H	Good	Brush mattress, brush layering, live cuttings
Sansevieria hyacinthoides	Lengua de chucho, sweet Sansevieria	Local collections	C	Good	Brush mattress, brush layering, live cuttings
Sphagneticola trilobata	Margarita, Bay Biscayne creeping oxeye	Local collections	C	Fair	Brush mattress, brush layering, live cuttings
Zingiber spp.	Jengibre, Ginger	Local collections	C, H	Fair	Fascines, poles, brush mattress, brush layering, live cuttings
Cordyline terminalis	Ti	Local collections	H	Good	Live cuttings, poles
Polyscias guifoylei	Panax	Local collections	H	Good	Live cuttings, poles
Erythrina variegate	Tropic Coral' Tall Erythrina (cultivar)	Nursery	H	Good	Live cuttings, poles

**Region code number or letter:

1–Northeast (ME, NH, VT, MA, CT, RI, WV, KY, NY, PA, NJ, MD, DE, VA, OH)
2–Southeast (NC, SC, GA, FL, TN, AL, MS, LA, AR)
3–North Central (MO, IA, MN, MI, WI, IL, IN)
4–North Plains (ND, SD, MT eastern, WY eastern)
5–Central Plains (NE, KS, CO eastern)
6–South Plains (TX, OK)
7–Southwest (AZ, NM)
8–Intermountain (NV, UT, CO western)
9–Northwest (WA, OR, ID, MT western, WY west)
0–California

A–Alaska
C–Caribbean
H–Hawaii

*Cultivar

The NRCS Plant Materials Program is responsible to locating native species to address conservation problems

Once a species is identified, the Plant Material Centers make multiple collections of this species, plant them out, compare them against each other, select the best ones, and release them to the public market.

The release notice describes where the cultivar was collected and how and where it was tested. This release notice, or pedigree, also explains how the cultivar performed in various soil series, precipitation zones, and provides other information regarding its growing requirements

9.3.2 Discussion and Recommendations

If done well, this is a relatively low cost solution to vegetating banks. Plantings at small to medium size creeks appear to be more successful.

Risk Analysis. If poles are not adequately located in the ground water, or the groundwater table drops too rapidly in the planting season and season following, the entire site can fail.

Live stakes can also be scoured out at some locations if high flows occur before successful rooting.

Monitoring and Maintenance. Groundwater monitoring is required before the planting season and the vegetated site should be monitored for several successive years.

9.4 Fabric Encapsulated Lifts

Fabric encapsulated lifts (FES) are bioengineered bank stabilization consisting of constructed lifts of fabric wrapped soils, interlaced with rooted willow plants (Figure 9–3). This system provides two stages of bank protection. Similar to standard covers of coir fabric, biodegradable fabrics provide erosion resistance of soils in the first years following construction. Fabric wrapped soils that extend a couple ft into the bank also provide soil reinforcement similar to construction of some forms of retaining walls. This reinforcement is temporary lasting only 3 to 5 years, depending on manufacturer's specifications, until the fabric decomposes (Miller and Skidmore 1998). Essential to the continued stability of an FES bank, are the establishment and growth of rooted willows placed between the lifts as the bank is constructed. Resilient willows are installed between the lifts with 12 inches of previously grown root, in a fabric sock with soil, are laid horizontally with roots extending into the bank. FES, like planted vegetation, are intended to be deformable with time. Vegetation slows erosion, but is not intended as a fixed or hardened bank. Planting pre-grown willow roots between fabric wrapped earth lifts accelerates the natural process of soil and root integration that reinforces the earth banks.

9.4.1 Design Procedure

At this point, project design criteria, ecological factors, hydrology, geomorphic factors and general hydraulic and scour factors have been assembled previously to aid in selecting a suitable bank stabilization method. It is assumed that this information is available now as a resource to the design. Important aspects of design criteria, hydrology, and permitting are described in chapter 2. Determine the geomorphic factors including sediment and sediment continuity issues that may impact channel stability as discussed in chapter 3. Define general hydraulics including energy (chapter 4) and potential for scour (chapter 5) that may influence the channel and the extent of the floodplain. Steps of the design will recommend revisiting some of these investigations to obtain more detailed information for this design.

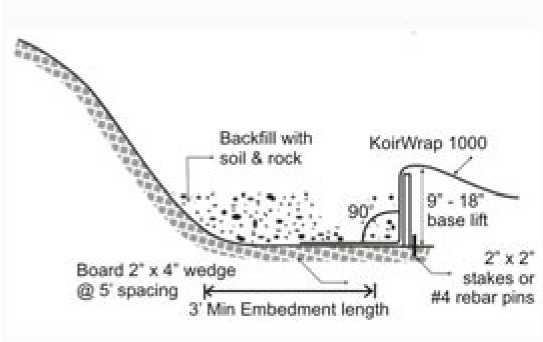


Figure 9–3. Constructing fabric encapsulated lifts (FES). From Nedia Enterprises, Inc.2013 (http://www.nedia.com/Soil_wrap_fabric_inst.html).

Step 1. Acquire land or permitting for proposed floodplain. The level of reliability can be high, provided that there is sufficient land available for lateral migration as the bankline deforms. This method is not suited for areas where the bankline has eroded near valuable infrastructure without channel relocation. The project life and design life can be long because the method allows geomorphic and flood plain processes.

Step 2. Flood plain and green line modeling. The amount of flood plain lowering can be analyzed using hydraulic modeling. The main channel should be connected to the flood plain at least above the mean 2-yr discharge.

The greenline, the lowest elevation where native woody species grow on the channel banks should be surveyed, along with channel transects. Using a hydraulic model such as HEC-RAS, the discharge can be determined corresponding to the vegetation elevation. Then the hydraulic model can be used to estimate lowest connectivity surface elevation for the project reach or site.

Step 3. Assess maximum scour depth and design bank toe. Evaluate the potential for scour at the site and compute the maximum scour depth, including bend scour as described in chapter 5. At a minimum, scour depth should be based upon the 2 to 5-year flood. Design a toe if it is required to prevent flow erosion from undercutting the bank. Guidance on designing a stone toe is presented in Chapter 13. The toe depth should be set at the maximum scour depth and the top of the toe can be set to the elevation of the green line. Vegetation on the bank toe, below the green line, is not expected to survive due to wet soil conditions. A bank toe should not require vegetation to be stable.

Large voids in the stone toe should be avoided since soil can settle into the rock, making it difficult for the initial layer of vegetation to establish. Fill some of the voids of large rock (if the designed riprap gradation still contains large voids) with stone, not earth; however, avoid a smooth surface or over-thick layer of intermediary soil between the toe and the first FES layer. During an early high flow event, the soil layer could be washed out collapsing the upper wrapped lifts before plants have a chance to establish.

Step 4. Design fabric encapsulated soil lifts (FES). FES lifts can be used in banks from the green line to a distance up the banks where the higher resistance provided by a FES lift is no longer needed. For example, the bank could consist of a stone toe and FES lifts to the top of bank; or could consist of a stone toe, FES lifts, and a vegetated slope above the lifts to the top of bank. If there is no vegetated slope above the FES lifts, ensure the surface wrap of FES geotextile is adequately extended back and well anchored in the bank.

Typically, the soil lifts are constructed by placing soil on top of a portion of two horizontal geotextile fabrics. An outer layer of a biodegradable fabric, such as a coir fabric of twisted coconut fibers woven into a strong mesh, provides high tensile strength to reinforce the soil. The inner layer of nonwoven coir, burlap or other matting prevents piping of soil fines through the coarser outer fabric. After the soil is compacted, the remaining fabrics are wrapped over the front and top of the soil mass and staked in place. These lifts are built one on top of another and set back to form a geotextile retaining wall. FES lifts are usually about 1 foot thick, but could range from 6 to 18 inches thick. Lifts higher than 18 inches can erode out more readily. Soil lifts need to be lightly compacted and both neatly and tightly wrapped to prevent drag forces on loose fabrics from high flow events. An attentive and experienced construction inspector can improve the stability of the installation. FES lifts are more erosion resistant than fabric covered slopes if they are well constructed.

Live plant cuttings, sometimes dormant willows, can be placed between the layers, protruding from the face of the constructed bank. Willow plants can also be rooted in a sock of soil in the nursery and placed between the layers similar to cuttings. The advantage to the willow socks are the advanced root system that is already formed under the controlled moisture and soil conditions in the nursery. Willows are planted between the toe and first layer of lifts, and planted between succeeding layers of lifts. Plants located below the green line have increased risk of mortality from prolonged root or plant inundation and plants on upper lifts need to be able to send roots down sufficiently fast to keep up with a declining water table during the growing season. Fabrics provide temporary stability only until the fibrous root systems of the willows bind the soil particles to anchor the lifts. The long term success of the project is dependent on the survival of willow plantings. Proper handling and placement of willows should be well covered in the drawings and specifications, and well monitored during construction and in the post-construction period.

Ideally, once the willows have rooted and the fabric materials degrade, the willows should be well established and stabilizing the bank. Branches of established willows protruding from the face of the lifts also reduce the shear stress on the bank and can provide cover and shade for fish and wildlife.

Step 5. Design erosion protection for upper slopes. There may be less erosive conditions on the upper banks if the slopes are less steep and opening up to the flood plain. Erosive forces on the upper slopes should be evaluated to determine if densely planted native vegetation may be sufficient to provide protection. Each line should have several rows of plantings. Vegetation plantings should be native riparian species using local native cuttings, or nursery stock depending upon availability, local experience and cost. Coir fabric can also be used to cover the upper banks and temporarily increase the erosive resistance until plants can establish.

Step 6. Design tie-ins for the end points of all bank elements. Develop methods for wrapping each treatment into the banks. End points are the most susceptible to failure and should be securely finished. Often treatments are sloped back into the bank with the end points securely covered. These transitions should fall where flows are no longer impacting the bank and ideally are locations where secondary flows, even during high flow events, are non-erosive.

Step 7. Determine plant sources for the construction period, including nurseries. Prepare a draft planting plan and determine the species, number and types of plants (potted, socks, cuttings, etc). With this estimate, begin contacting nurseries to determine possible sources and potential time frames for successful delivery of these materials to the construction site. It is not unusual to request plants a year or two early to give the nursery a chance to provide suitable plants. Coordinate with the nursery on the estimated number of plants and most suitable plant varieties and spacing. Also assess the need for irrigation and expand the draft planting plan to include irrigation methods.

Step 8. Develop drawings and specifications. Put together a plan set for the project and include plant specifications that can aid the contractor.

Step 9. Determine post-construction and other maintenance needs. Maintenance requirements include replanting vegetation that dies, plant irrigation and/or flow management until plants become well established, and review of tie ins and fabric stability prior to the vegetation becoming firmly established.

Step 10. Assess Constructability. Constructability issues include access, available land area to construct the toe, and bioengineering on the upper bank. If a cofferdam is being used, construction plans must include dewatering. If toe protection is constructed subaqueous, suitable equipment and construction timing during low flows would be important considerations. Assess on site storage and wetting for vegetation.

9.4.2 Discussion and Recommendations

Complexity. Level of installation complexity can be fairly high for FES banks.

Risks. An important aspect of FES is good construction. Smooth, well-constructed lifts have higher erosion resistance in comparison with loosely packed lifts with wrinkled and sagging fabrics. Lifts have to be constructed tight to maintain smooth flow lines. When the lifts are not well constructed, flooding can damage fabric-wrapped soil lifts. If fabric is damaged, gravel and soil can be washed out from the fabrics, leaving the loose and baggy fabric susceptible to increased drag. Similar to fabric geotextile filters, the flapping motion of exposed loose fabric in the current can rip out sections of material that fall out and harm adjacent sections.

Ice can cause a second form of failure. Ice freezes to the face of the lifts and with subsequent movement of the ice, the outer protective fabric and inner coir fabric are both ripped, spilling the soils. Loss of spoils prevents the establishment of plants, retarding the second stage of bank strength development.

Alternative Methods. One alternative method that has not been verified by the authors is the use of soil-filled burlap bags covered with erosion control blankets to build soil lifts. A second method that is advertised online with some articles reporting satisfaction over the installation (life of product and success of vegetation establishment not reported) are the BioD-Block products. These products rely on a coir block system to provide an all-in-one structure to shape and construct the bank. Photos of the installation show a loose outer fabric that could create more drag in higher flows. More information on the stability and long term success of the installations after high flows, and establishment of now mature plants after high flows should be investigated.

Maintenance and Monitoring. Maintenance requirements include replanting vegetation that dies to hard toes if erosion occurs prior to the vegetation becoming firmly established.

9.5 Degradable/Deformable Stone Toe

Bank stabilization and channel restoration or channel maintenance should strive to create a condition of sediment supply in balance with sediment capacity. A second goal is to maintain mobile channel boundaries so that the channel section can evolve in response to changes in flow and sediment, the process of dynamic equilibrium. Under conditions of higher flows and lower sediment availability, there is more energy in the river system acting on the channel banks and on the floodplain. Natural alluvial channel systems have dynamically stable planforms with gradual erosion and accretion of riverbanks (Miller and Skidmore, 1998) while transporting the incoming sediment supply.

Vegetated banks are deformable during high flow events. Constructed banks often require a hardened toe at elevations below the green line, the zone where vegetation cannot grow. The objective of the degradable toe method is to provide a stable toe of rock or other materials, while vegetation is becoming established along the bankline above the green line, and after vegetation has had adequate time to establish on the banks, the toe is allowed to become mobile (Miller and Skidmore, 1998). Established vegetation reinforces or stiffens the upper bank, and vegetation root structure provides some additional bank stability in the toe of the bank below the vegetation green line, after the temporarily rigid toe degrades. Vegetation is more flexible than a hardened bank, and allows for some bank erosion and channel migration that is part of the dynamic, but still relatively stable, river condition. The end result is both a deformable bank and a deformable toe that allows for some channel and floodplain adjustment, in response to changes in flow and sediment regimes.

Banks can be stabilized with vegetation in the upper bank and a bank toe that breaks down after a few years, if the floodplain is frequently accessible to flows. An active floodplain effectively dissipates energy, which reduces main channel velocities and sediment transport capacity. Lower main channel velocity reduces bank velocity and shear stress. Lowered sediment transport capacity can reduce or halt channel incision, a form of excessive channel flow energy. Figures 9–4 and 9–5 show the features of this method and a schematic after bankline deformation occurs.

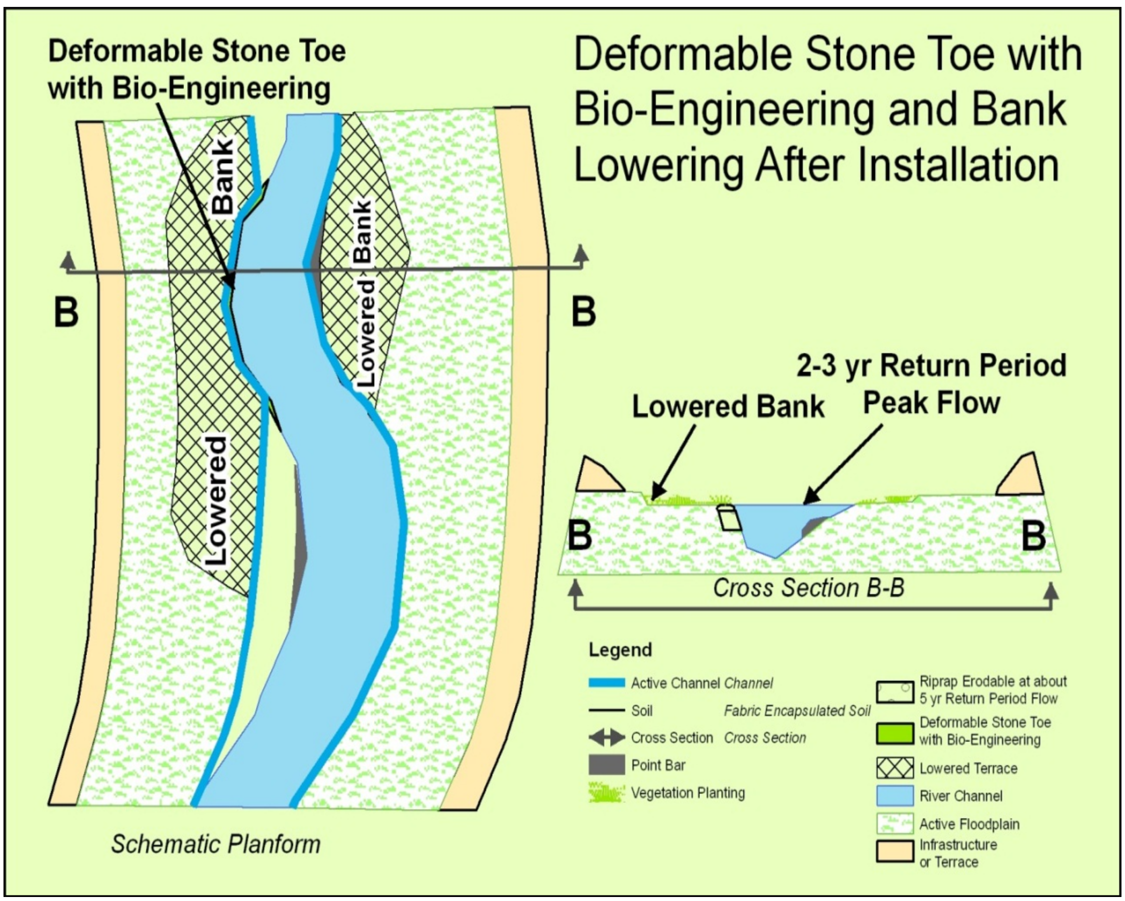


Figure 9–4. Deformable stone toe with bioengineering and bank line features (Baird and Makar, 2011).

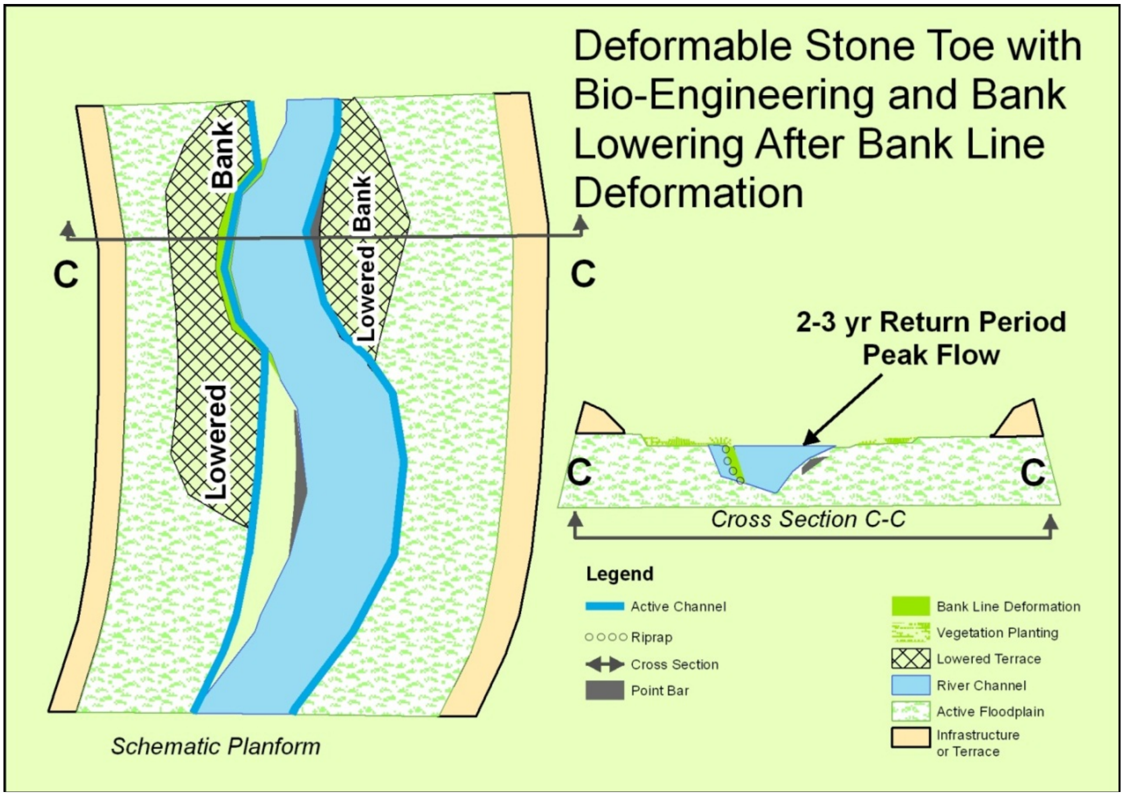


Figure 9–5. Deformable stone toe with bioengineering and bank lowering after bank line deformation (Baird and Makar 2011).

Figure 9–6 contains examples of degradable toes that are intended to produce deformable toes. Both examples in this figure use coconut fiber roll (coir logs) listed on the materials blacklist at the start of Part II. These materials may be suitable for backwater and wetlands applications, but are very difficult to use near any flow current. Despite stakes, they can catch flows and be ripped out of banks disturbing the partially established vegetation. Straw bales can also be used as deformable toes, but like coir logs, may survive high flow events better if well inset into the bank.

A third method of a deformable toe is to wrap smaller rock into a degradable fabric. As a single mass, the stone weight is sufficiently stable but as the fabric breaks down, individual gravels or cobbles can transport downstream slowly degrading the stability of the toe consistent with controlled bank deformability. Rounded river rock should be used in these installations instead of crushed, angular material. Deformable bank toes are best suited to smaller streams and creeks.

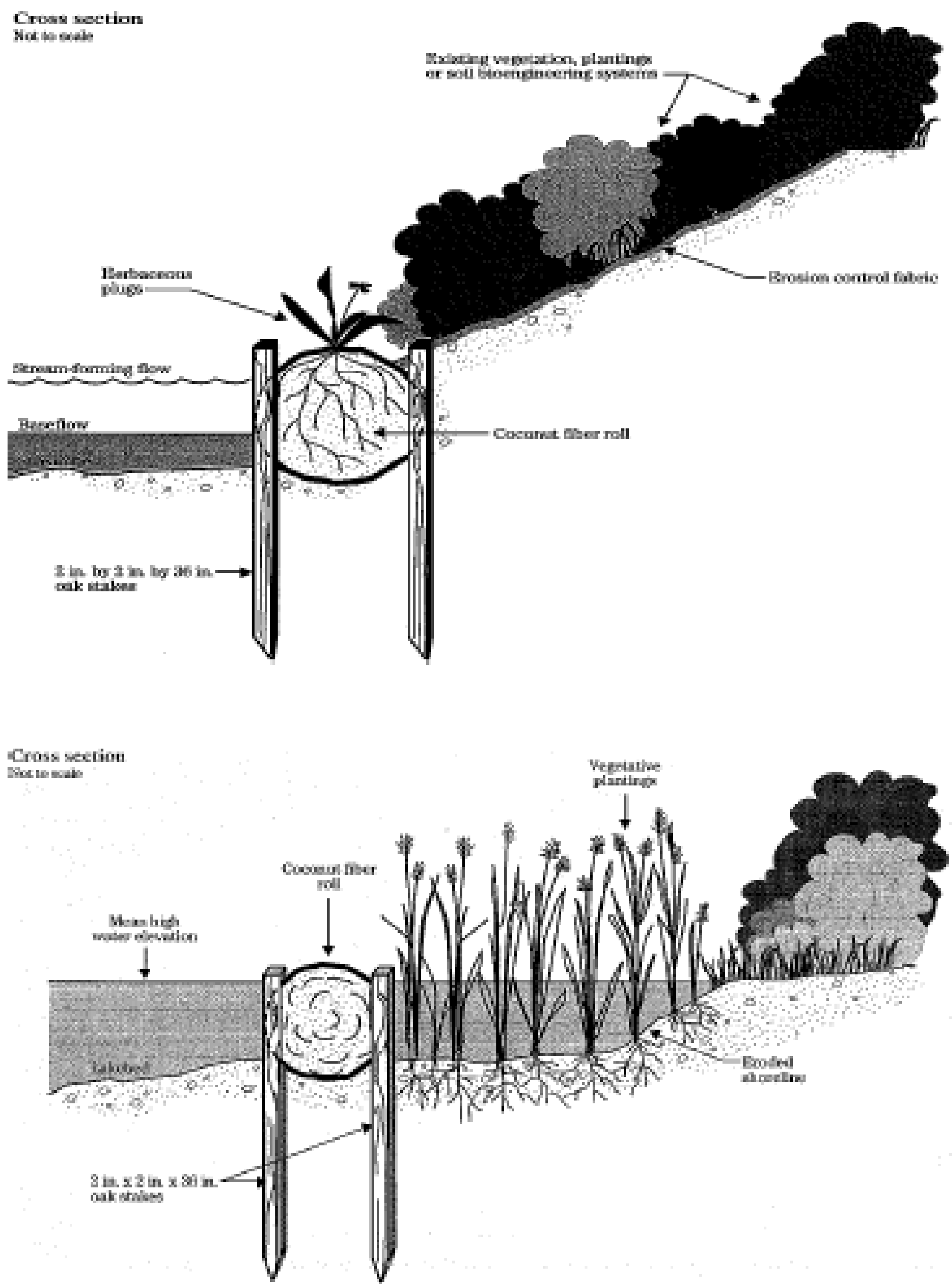


Figure 9–6. Examples of a degradable bank toe.

9.5.1 Design Procedure

At this point, project design criteria, ecological factors, hydrology, geomorphic factors and general hydraulic and scour factors have been assembled previously to aid in selecting a suitable bank stabilization method. It is assumed that this information is available now as a resource to the design. Important aspects of design criteria, hydrology, and permitting are described in chapter 2. Determine the geomorphic factors including sediment and sediment continuity issues that may impact channel stability as discussed in chapter 3. Define general hydraulics including energy (chapter 4) and potential for scour (chapter 5) that may influence the channel and the extent of the floodplain. Steps of the design will recommend revisiting some of these investigations, to obtain more detailed information for this design.

Step 1. Perform hydraulic modeling for determining flood plain elevations, erosive forces, and the green line flow event. Determination of necessary floodplain and the design values for flood plain lowering can be analyzed using hydraulic modeling. The main channel should be connected to the flood plain at or above the bank-full discharge. Check the floodplain to ensure adequate land or right-of-way is available for proposed floodplain using results from the floodplain and hydraulic model.

For an incising channel, re-connecting the floodplain and the main channel can help balance the incoming sediment supply with sediment transport capacity. There could also be a reduction in the main channel velocity. Floodplain connectivity is an important component of establishing an effective zone of riparian vegetation which provides some bank stability after the degradable toe is removed by high flows.

The discharge corresponding to the green line (minimum elevation of vegetation establishment) should be compared to the peak flow frequency, and flow duration curves to assign a return flow interval. The greenline, the lowest elevation where native woody species grow on the channel banks should be surveyed, along with channel transects. Using a hydraulic model such as HEC-RAS, the discharge can be determined corresponding to the vegetation elevation. Then the hydraulic model can be used to estimate lowest connectivity surface elevation for the project reach or site. The elevation of the deformable toe is the water surface elevation corresponding to the discharge below which vegetation does not grow. The terrace should be at the elevation of the mean annual flow peak or Q_2.

Guidelines on the elevation and plan view dimensions flood plain surfaces could include: 1) available land area, 2) local riparian habitat requirements, 3) overbank topography, and 3) volume of earthen material removal and availability of disposal areas. In order of lowest surface elevation to highest, recommended elevations for multiple floodplain connectivity surfaces are: 1) vegetation elevation, 2) mean annual peak flow, 3) 2-yr return period peak flow, and 4) 3-5 year return period peak flow.

Step 2. Estimate Scour. Compute maximum scour, including bend scour as described in chapter 5. At a minimum, scour depth should be based upon the 2 to 5-year event used to size riprap.

Step 3. Design deformable toe. If the toe is fabric wrapped stone, the material should be sized to be stable at the 5 year return period peak flow. Thickness of the degradable rock should be large enough to contain launchable rock of sufficient volume to fill the estimated scour hole. If coir logs or straw bales are used, explore inset of the materials and a simple yet solid anchoring system that is also biodegradable. Cables and non-degradable, geotextile filter fabric are on the construction blacklist and should not be used.

Similar to the design for a riprap revetment, the stone in the toe can be installed to the maximum scour depth, or designed with no excavation to have launchable rock. The volume of stone in the toe should provide launchable rock for the scour estimated to develop for discharges up to the flow event for which the stone as individual particles is intended to erode. Stone toe protection should be sized using the channel hydraulics for the 2 to 5-year return period flow event.

An alternative to the riprap sizing equations of chapter 13 is an alluvial fill comprised of the largest bed material sizes in the stream, wrapped in coir fabric. Sand bags have also been used in sand bed streams but the success of this method is less well-documented. Fabric wrapped alluvial fill requires construction in the dry and is, therefore, limited to cases where a cofferdam can be economically constructed. Construction of the stone toe using river gravel will likely be more erodible, after coir fabric degradation, than a stone toe constructed from riprap, depending on the size of the riprap. Submerged toe construction using river gravel wrapped in biodegradable fabric offers initial immobility for the life of the fabric (e.g., 3 to 5 years) and eventual but gradual deformation after fabric degradation (Miller and Skidmore 1998). This benefit is in contrast to riprap toes that are intended to be immobile for the life of the project.

Sizes within the toe range from the median sizes, which are transported at the 2- to 10-year events, and the largest sizes, which are transported at the 50-year event (Miller and Skidmore 1998). These sizes depend on the desired level of protection. In cases where higher bank deformation is desirable, the maximum size can be reduced, for example, to become mobile at the 10- to 15-year event.

Design of the riprap toe is well established and could be adapted for use as a deformable toe if rounded rock was used in place of angular rock (although the design is based on the increased stability of angular rock). See section 14.2 for information on riprap sizing, gradation and thickness.

The elevation of the top of the stone toe section is generally based on the elevation of depositional surfaces where vegetation is growing. Miller and Hoitsma (1998) recommend using the elevation where perennial vegetation usually grows. The width of the flood plain relative to potential channel movement has not been well established and would depend on local channel characteristics, valley geology, channel morphology, and amount of flow area increase needed to reduce sediment transport capacity to be in balance with supply.

The lifespan of the biodegradable coir fabric depends on the amount of wetting and drying and exposure to sunlight, and it generally ranges from 3 to 5 years. During this period of time, the toe of the slope must remain stable, and the vegetation must grow to a sufficient size and density to provide future bank stability, while allowing a natural rate of bank migration. The properties of bioengineered fabrics are documented by Hoitsma (1999).

For incised channels, constructing a two stage channel is advantageous to balance sediment transport supply with capacity. This also establishes an inset flood plain where erosion rates will be less than untreated areas. This is especially true where the bank height exceeds the riparian root depth, resulting in high bank erosion rates. The inset flood plain should be designed to balance sediment transport capacity with supply. See section 9.2 for more information on bankline lowering and two stage channel requirements.

Vegetation should be species indigenous to the riparian zone and can be planted using methods presented in previous sections. Other measures such as pre-vegetated mats, grown sod, live fascines, or wattles (Gray and Sotir 1996; Schiechtl and Stern 1994; Benthrup and Hoag 1998) may also be considered on the bank above the deformable toe.

Step 4. Design vegetated banks above toe based on erosive forces. See earlier sections for descriptions of some bank vegetation methods that would be deformable and may be suitable for the bank protection. Coir or other biodegradable fabric should be used that has an expected life span of 3 to 5 years, over which time the vegetation would be firmly established.

The lowest connectivity surface should be is based upon the deformable bankline design and the green line elevation at which vegetation grows naturally in the river. The deformable bankline can be designed with the riprap toe protection elevation being the same as the green line elevation, then adding a more erosion resistant vegetation level, possibly one or two fabric encapsulated soil (FES) lifts (section 9.4) each about 1 foot thick (see figure 10–2), but could range from 6 to 18 inches thick. The elevation of the upper zone of planted vegetation then would be the vegetation elevation plus the thickness of the two FES lifts. For this design bankline plantings would be incorporated in FES lifts and on the upper slope.

Step 5. Design edge treatments/transitions for toe and bank. Edge treatments should tie bank treatments and stone toes into the bank to protect structure from upstream lateral movement within the range of expected meander migration, and protect the downstream bank from exit velocities and secondary flow turbulence. These are often needed for stone treatments that increase flow velocities along the toe, and form scalloped erosion where a stone surface meets vegetated bank. See section 14.3 for more guidance on riprap edge treatments. Edge treatments for vegetation will depend on the vegetation methods but should also tie back into the bank to prevent erosion failure.

Step 6. Assess Constructability. Constructability issues include access, available land area to construct the toe, and bioengineering on the upper bank. If a cofferdam is being used, construction plans must include dewatering. If toe protection is constructed subaqueous, suitable equipment and construction timing during low flows would be important considerations. Bank should be in a smooth alignment and the bank slope graded prior to riprap placement. There should be sufficient overlap between sections of the fabric encapsulated soil to prevent slippage. Vegetation should be available at time of construction to limit on-site storage time. Vegetation should be stored in the shade and watered while on-site to prevent the plants from being stressed prior to planting.

9.5.2 Discussion and Recommendations

The level of reliability can be great, provided that there is sufficient land available for lateral migration as the bankline deforms. This method is not suited for areas where the floodplain is excessively constricted, the bankline has eroded near valuable infrastructure, and there is no option for a channel relocation. The project life and design life can be long because the method accommodates geomorphic and flood plain processes. This method provides a unique combination of allowing natural fluvial process of bank erosion, lateral migration and point bar deposition to occur but with a slower rate.

The rate of bank erosion using this method is not known, but an approximate range might be determined with a geomorphic assessment, depending on reach and system characteristics.

Where high value infrastructure needs protection from bank erosion, this method may be coupled with additional bank protection such as trench filled riprap. Trench filled riprap is an esthetic means to provide longer term protection which would require riprap augmentation after lateral migration and riprap launching. See chapter 13 for more information.

Complexity. Level of installation complexity can be fairly high.

Risk and Failure. Common modes of failure would be: erosion of toe rock prior to the vegetation on the bank becoming firmly established; large die-off of plants due to excessively wet or dry conditions, or due to poor handling on-site and during construction; flanking of the bank structure; abrasion of the coir fabric by a substantial bed material load; planting vegetation at an elevation along the bankline that is too low (where it would be subject to waterlogging and the vegetation will not grow); and placing the vegetation plantings too high (resulting in reduced root density above the toe of the bank and high future bank erosion rates) (Miller and Skidmore 1998).

Means of reducing risks include:

- using a sufficient size and volume of well-graded riprap to launch into the scour hole and provide protection until vegetation is firmly established,

- replanting vegetation that dies,
- providing flanking protection with tiebacks, using riprap that is erodible at lower frequency events,
- planting vegetation at the elevation where perennial vegetation grows along the riverbank, and
- planting vegetation between and through several FES soil lifts to establish vegetation at different elevations for maximum root density.

The necessary distance between the deformable bankline and river side infrastructure to prevent encroachment is difficult to predict. Geomorphic analysis of historical channel planform and channel location changes through time, should be used to determine if enough land is available for this method to provide long term protection of infrastructure.

A key element of success is establishing a dense riparian zone on the bank and having appropriate floodplain connectivity to prevent excessive erosive stress on the bank.

Monitoring and Maintenance. Maintenance requirements include replanting vegetation that dies and potential riprap replacement should riprap erosion occur prior to the vegetation becoming firmly established.

9.6 Bio-Engineering on a Stone Toe

Described here is bank stabilization with a combination of vegetation or bioengineering on the upper bank, and a stone toe base. Design of a longitudinal peak stone toe is presented in more detail in chapter 13. When construction work can be accomplished from the riverward side of the bank, it may be possible to leave existing vegetation in the upper bank. If construction is from the bank side, then the existing vegetation is cleared and replanted.

A stone toe in this application is not deformable and is used to hold the bank and prevent undercutting and scour in a medium to high energy system. Bio-engineering on the upper bank should also be designed to match higher erosive forces but there is an assumption that erosive forces will decrease up the bank as the channel widens out. Also the stone toe should inhibit undercutting and erosion by interrupting the erosive secondary flow pattern, even if the upper vegetated bank is lost due to erosion.

An example of stone toe with bio-engineering is shown in Figure 9–7. Although not notable from the figure, the stone toe should extend below the deepest estimated scour depth or should include additional riprap for shifting into the scour hole. A granular figure, not a geotextile should also be used. See stone toe design in the bank hardening chapter.

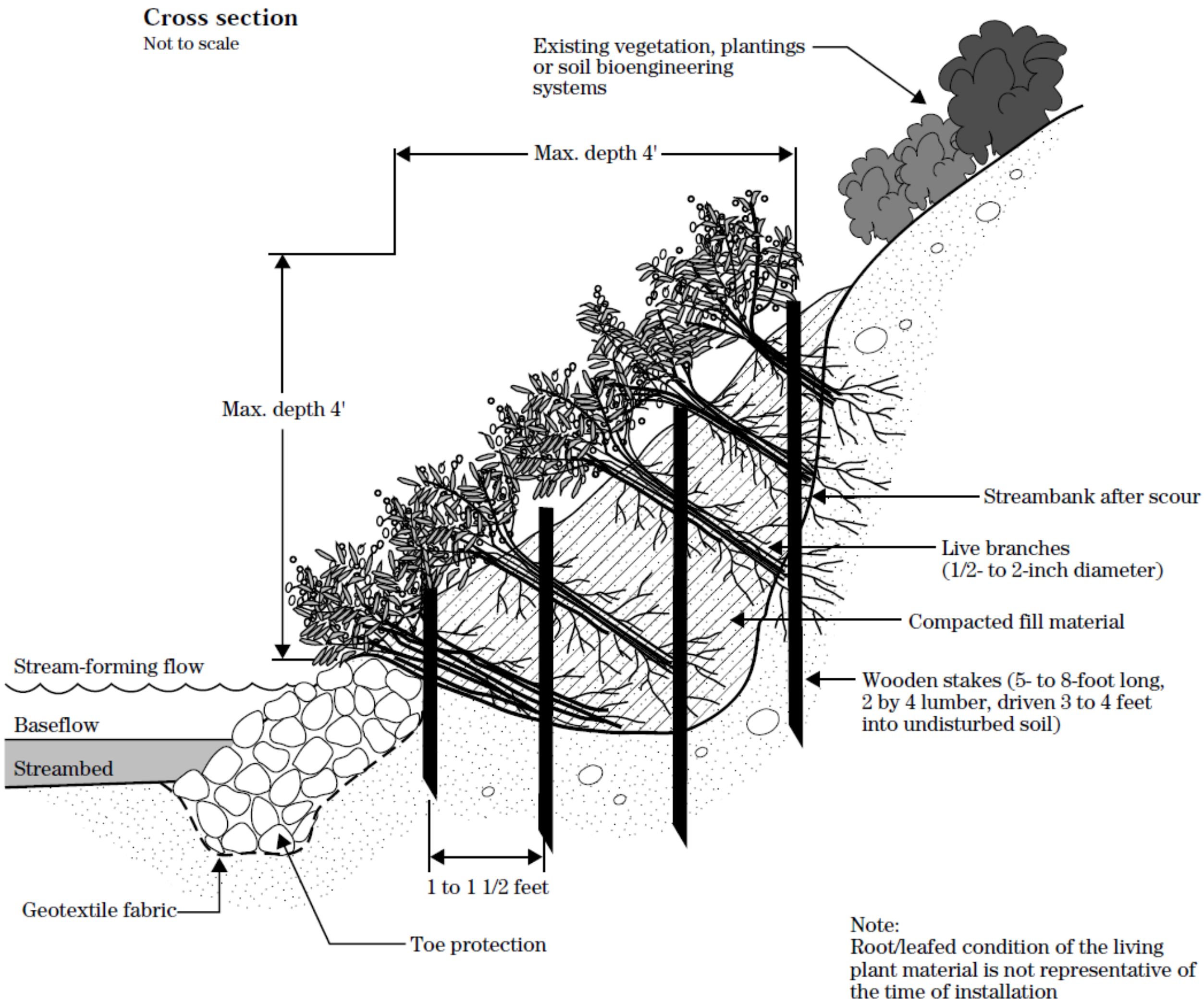


Figure 9–7. Longitudinal stone toe with bioengineering (after NRCS 1996). Branch Packing Details. This method can be used to avoid disturbing existing vegetation. A note reminds readers that live stakes should be installed while they are dormant, not with roots or leaves as shown. Three shortcomings of this figure are the shallow depth of the stone toe that does not appear to extend below a scouring depth or contain a triangular shape for launchable stone; a geotextile filter that should not be used in bioengineered river banks (a granular filter should replace geotextile), and the significance of the 4' maximum depth dimensions are unclear (no slope steeper than a 1V to 1H).

9.6.1 Design Procedure

At this point, project design criteria, ecological factors, hydrology, geomorphic factors and general hydraulic and scour factors have been assembled previously to aid in selecting a suitable bank stabilization method. It is assumed that this information is available now as a resource to the design. Important aspects of design criteria, hydrology and risk assessment, and permitting, are described in chapter 2. Determine the geomorphic factors including sediment and sediment continuity issues that may impact channel stability as discussed in chapter 3. Define general hydraulics including energy (chapter 4) and potential for scour (chapter 5) that may influence the channel and the extent of the floodplain. Steps of the design may require revisiting some of these investigations, to obtain more detailed information for this design.

Step 1. Perform hydraulic modeling for determining flood plain elevations, erosive forces, and the green line flow event. Durability and design life depend on the design flood used. Design flood can range from the 2- to 100-year return period discharges. Typically, the return period design flood corresponds to the same level of the protection scheme itself (Escarameia 1998). A 25-year event sometimes provides an economical design life and a reasonable design flood in the absence of other requirements, and when the bank is not located near high population areas or infrastructure that cannot be disturbed.

Determination of necessary floodplain and the design values for flood plain lowering can be analyzed using hydraulic modeling. The main channel should be connected to the flood plain at or above the bank-full discharge. Check the floodplain to ensure adequate land or right-of-way is available for proposed floodplain using results from the floodplain and hydraulic model.

For an incising channel re-connecting the floodplain and the main channel can help balance the incoming sediment supply with sediment transport capacity. There could also be a reduction in the main channel velocity. Floodplain connectivity is an important component of establishing an effective zone of riparian vegetation which provides some bank stability after the degradable toe is removed by high flows.

The discharge corresponding to the green line (minimum elevation of vegetation establishment) should be compared to the peak flow frequency, and flow duration curves to assign a return flow interval. The greenline, the lowest elevation where native woody species grow on the channel banks should be surveyed, along with channel transects. Using a hydraulic model such as HEC-RAS, the discharge can be determined corresponding to the vegetation elevation. Then the hydraulic model can be used to estimate lowest connectivity surface elevation for the project reach or site. The elevation of the deformable toe is the water surface elevation corresponding to the discharge below which vegetation does not grow. The terrace should be at the elevation of the mean annual flow peak or Q_2.

Guidelines on the elevation and plan view dimensions flood plain surfaces could include: 1) available land area, 2) local riparian habitat requirements, 3) overbank topography, and 3) volume of earthen material removal and availability of disposal areas. In order of lowest surface elevation to highest, recommended elevations for multiple floodplain connectivity surfaces are: 1) vegetation elevation, 2) mean annual peak flow, 3) 2-yr return period peak flow, and 4) 3 to 5-year return period peak flow.

Step 2. Determine alignment. Draw an arc that represents the desired bankline location if it is different from the current eroding bankline. The desired bankline could be the existing concave bank or a new bankline that reverses past erosion. The curve should fall within geomorphic guidelines for the bend radius based on type of river plan form. Note that the flow alignment and bankline locations may need to be drawn several times after both are first sketched to provide for smooth transitions into and out of the curve, and a proper thalweg. Movement of the bankline from the existing should be minimized to save fill costs. If the bend to be protected has an irregular shape then these can be smoothed to balance cut and fill. The bankline may need to be re-shaped to have a constant suitable slope and should be a maximum of 2H:1V. A 1.5H:1V is too steep for long term sustainability of riprap during frequent high flow conditions.

Step 3. Evaluate scour. Bend scour and other and other types of scour can be estimated using the methods found in chapter 5.

Step 4. Design stone toe. The top elevation of the stone toe is the green line, the water surface elevation corresponding to the discharge below which vegetation does not grow. A longitudinal green line can be surveyed in the field and matched to a flow return interval using the hydraulic model. This elevation can also be based on the mean annual water surface. Bioengineering techniques generally employ woody plant species that are limited to growing at bank elevations above a base flow level (Fischenich 2000; NRCS 1996); thus, the top of the longitudinal stone toe should, as a minimum, be the elevation at which vegetation grows in the river system. In arid climate zones or situations where there can be large fluctuation in the mean annual flow, the long-term mean annual water surface may be above depositional surfaces where vegetation is growing. In low rainfall climate zones, plants need to have a root zone that extends down to the ground water table; thus, plants may need to be placed at lower bank elevations than in climates with sufficient rainfall to provide for plant growth. Vegetation in riprap has been shown to increase bank stability (Shields, 1991). Large voids in the riprap may need to be filled with moisture retaining soils near the top of the toe, to aid the establishment of vegetation located directly on top of riprap. The green line can vary from year to year depending on the hydrologic regime. Vegetation is used to protect the remainder of the slope up to the top of the bank or a peak flow design discharge such as the 25-year event water surface elevation.

An alternative method if the toe is not excavated to the maximum depth of scour, is to specify a weight or volume of stone per unit length of streambank to be placed in a triangular shape rather than to specify a given finished elevation and cross-section dimensions. Volume of stone is determined by estimating the total depth of scour and elevation of crown, assuming a uniform thickness of rock. A typical cross section can be specified on drawings along with relatively smooth alignment to fit site conditions. For more detail on the design of a longitudinal stone toe refer to chapter 13.

Step 5. Design vegetated banks. For a limited set of bio-engineering applications maximum permissible tractive force can be found in tables 4–2, 9–2 and 9–3. The lower range of values in table 9–3 is recommended because of the variability of plant growth rates and density and soil types. Sandy soils are especially susceptible to erosion around bio-engineering plants. An erosion control bio-degradable fabric is recommended in most bio-engineering applications.

Vegetation should be species indigenous to the riparian zone, collected live from nearby riparian zones, and can be planted using methods described previously in this chapter. Planting with a coir fabric cover, live staking (Sotir and Fischenich, 2007), containerized plants, or fabric encapsulated lifts with live willows are all potential methods depending on site conditions. Other bioengineering measures include pre-vegetated mats, brush trench, brush layers, pole plantings, grown sod, live fascines, live crib walls, brush mattresses, or wattles (Gray and Sotir, 1996; Schiechtl and Stern, 2000; Bentrup and Hoag, 1998). Coconut fiber rolls, also described as coir logs, are not recommended for streams with even mild flow currents due to the difficulty in securing these bulky materials firmly, to resist high flow events. Methods such as brush mattress, brush layers, and fabric encapsulated soils can initially provide the most bank stability. Examples of these methods are shown in the figures 5.2.2, through 5.2.7.

Planting to the depth of the water table in arid climates is essential for plant growth and survival or irrigation tubes may need to be installed down to the root zone so that plants will have sufficient water to grow roots to the water table. Irrigation tubes require replenishment as needed to maintain soil moisture.

The site may require earthwork before installing soil bioengineering. The disadvantage to this method is that established vegetation on the banks will need to be removed. However steep, undercut, or slumping banks should ideally be graded to 1V to 2H or flatter, in preparation for planting. Planting is best accomplished during the dormant season.

Armor benefits of bioengineering located above the riprap toe along the bank are not immediate, so many schemes employ biodegradable fabrics, including fabric encapsulated soil lifts, biodegradable blocks, and fabric rolls (Fischenich 2000; NRCS 1996). Using biodegradable fabrics prevents bank erosion above the stone toe until vegetation is established. In arid climate zones, complete covering of the bank with vegetation can take many years to establish.

Step 6. Design end transitions. The upstream and downstream ends of riprap revetment should be protected against erosion by increasing the revetment thickness or extending the revetment to areas of non-eroding velocities and relatively stable banks. Transitions or tiebacks should be designed to locations that are zones of slackwater upstream of and downstream from the project site (NRCS 1996).

Tiebacks are sometimes used instead of transitions on larger rivers but this can be a tradeoff since excavation of the bank for the tiebacks makes the bankline more susceptible to erosion. Length of tiebacks is based upon expected channel migration, and maintenance capability. If there is no planned maintenance, length should be sufficient to protect the structure from upstream lateral movement within the range of expected meander migration. Its length should be increased near high value riverside protected infrastructure. In some cases the length of tiebacks should be the historical width of the meander belt width. A downstream tieback can be constructed a sufficient length to protect against erosion due to high exit velocities and turbulence. Usually 30-50 ft is sufficient. Tiebacks should be angled about 30 degrees from the primary flow direction. Tiebacks with an angle of 90 degrees have resulted in failures at the downstream end of the structure due to flow expansion (McCullah and Gray 2005). Tieback riprap volume should be the same as the stone toe volume per linear foot or increased by 20% due to increased riprap erosion potential from turbulence.

Step 7. Use modeling or other methods to check sediment transport. Compare existing and planned banklines for sediment transport issues to identify channel stability and any significant changes under proposed conditions. Flow, sediment and vegetation models can be used for comparison of existing and proposed conditions.

Step 8. In addition to drawings, prepare specifications, a planting and irrigation plan, and a monitoring plan. Define appropriate materials and installations, and also define the care of plants before and during construction.

Step 9. Contact potential nursery suppliers. Prepare a draft planting plan to determine number, species and size of plants to be included. Estimate the schedule timeline and determine if there are sufficient suppliers for the project. It is not unusual to approach nurseries 1-2 years ahead of the delivery date. Determine the need and approach to irrigating plants for the first few years while they get established.

Step 10. Search for possible riprap sources and suppliers. Check for sources of good quality and durable rock, and ensure transportation costs are not cost prohibitive. Trucks may be limited on the number of transportable rocks to carry if they are large.

Step 11. Review Constructability. When construction work can be accomplished from the riverward side of the bank, it may be possible to leave existing vegetation in the upper bank. If construction is from the bank side, then the existing vegetation is cleared and replanted. Ensure there is sufficiently large equipment for large stone placement. Also check if banks can support the large machinery. To prepare for planting, bank should be in a smooth alignment and the bank slope graded to a 2H:1V slope or less prior to riprap placement.

When riprap is dumped or pushed off the bank top for placement on a slope there is sorting with the large sized material resting near the bottom of the bank toe. A hydraulic excavator with sufficient reach should be used to bring the large material up the slope, shape the launchable toe section, and ensure that the revetment is uniform thickness, and uniform size distribution.

Review the plan and designs for construction issues. Check for accessibility and roads that can support heavy machinery required for the project. Consider nearby neighborhoods and the level of tolerance for truck traffic, noise and dust. Consider how congested thoroughfares during rush hour impact the project. Investigate construction consistency with permitting requirements. Construction issues include access, bank clearing and shaping, and turbidity due to bank shaping and stone placement.

Review planting methods and ensure root tips of plants will be sufficiently close to the water table during the first several years of establishment. This may require adjustments to flow releases and flow management or may entail a good irrigation plan.

9.6.2 Discussion and Recommendations

Complexity. The level of complexity for installation is medium since rock placement is low and vegetation and bioengineering are higher. Complexity also depends highly on local site condition, habitat and environmental needs, and landowner preferences.

The longitudinal stone toe technique may be appropriate where the existing stream channel is to be realigned. However, for the stone toe to be effective in realigning the channel, the top elevation of the stone toe must be high enough so that is not overtopped frequently by high flows.

The Longitudinal stone fill toe protection is often used as the toe protection with other methods for upper bank protection and can be notched in the same manner as a transverse dike or retard in order to provide an aquatic connection between the main channel and the area between the structure and the bank slope.

Risk and Failure. The stone toe raises the level of reliability of a stabilized bank with the exception of channel instabilities such as continuing incision and channel migration processes in the river. Durability is also high since vegetation is a self-sustaining method once the plants are established. Depending on the design flows selected for riprap design, the stone toe may require maintenance after high flow events (see risk assessment in chapter 2).

Common risks are that riprap is undersized, poorly graded, separates during installation, or is placed on too steep a slope. There is the risk the site cannot be sufficiently dewatered during construction and muddy water makes it difficult to determine if the rock toe is well placed.

There is a risk the stone toe is constructed too low there will be a strip of planted vegetation more susceptible to death from inundation or erosion until vegetation establishes during the dryer conditions. This concern is preferred to a stone toe placed too high creating an excessive rock bank. Exposed rock in the bank area above the green line will separate vegetation from the water table because of the quickly draining riprap. With reduced moisture retention in this area, less seedlings can establish.

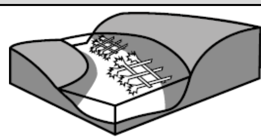
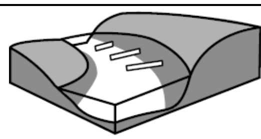
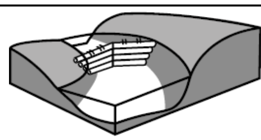
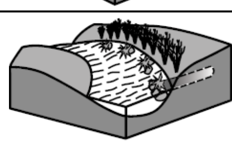
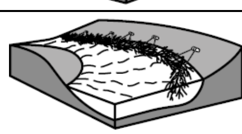
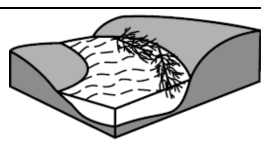
Vegetation is susceptible to erosion from high flow events for up to 3 years after planting. This risk can be reduced by covering the bank with coir or other degradable fabrics, and/or funding should be set aside for repeat planting after erosion damage.

10 Design of Wood and Boulders

Adding wood to streams consistently provided positive effects on physical aquatic habitat (Kiem et al., 2002) including increased cover for fish. Evidence has been validated that the addition of wood to a site is valuable for stream habitat and provides enhancement of bio-diversity (Lester et al., 2007). However, at this time, there is less control of material dimension and quality when logs or rootwads are used in comparison to traditional bank stabilization materials including riprap and gravels. The size of wood necessary to provide absolute stability may not be available or practical to install, especially in engineered log jams where multiple pieces are required.

Table 10–1 contains classification system for large wood instream structures. Presented in this chapter are methods for placing large woody debris and rootwads, rootwads and native material revetments, engineered log jams (ELJs), and boulder clusters.

Table 10–1. Classification of Large Wood Instream Structures

Configuration	Sketch	Description	Strength	References
Engineered logjams		Intermittent structures built by stacking whole trees and logs in crisscross arrangements	Emulates natural formations. Creates diverse physical conditions, traps additional debris	Abbe, Montgomery, and Petroff (1997); Shields, Morin, and Cooper (2004)
Log vanes		Single logs secured to bed protruding from bank and angled upstream. Also called log bendway weir	Low-cost, minimally intrusive	Derrick (1997); D'Aoust and Millar (2000)
Log weirs		Weirs spanning small streams comprised of one or more large logs	Creates pool habitat	Hilderbrand et al. 1998; Flosi et al. (1998)
Rootwads		Logs buried in bank with rootwads protruding into channel	Protects low banks, provides scour pools with woody cover	
Tree revetments or roughness logs		Whole trees placed along bank parallel to current. Trees are overlapped (shingled) and securely anchored	Deflects high flows and shear from outer banks; may induce sediment deposition and halt erosion	Cramer et al. (2002)
Toe logs		One or two rows of logs running parallel to current and secured to bank toe. Gravel fill may be placed immediately behind logs	Temporary toe protection	Cramer et al. (2002)

These guidelines include the design and analysis methods available at the time of publication. Planning and designing large woody debris and rootwads, native material revetments and engineered log jams, is evolving over time as more experience is gained and design methods improve. Reclamation is involved in an effort with the U.S. Army Corps of Engineers to develop a design guide for wood structures and engineered log jam which will be forthcoming.

10.1 Large Woody Debris and Rootwads

One of the challenges of placing a single element of large woody debris or a rootwad is anchoring these features against buoyancy, shear forces and the pressure from the buildup of debris. Cables are on the construction materials blacklist and are not recommended for installation. Some materials can work loose during high flows and cause severe erosion to banks with repetitive or cyclical pounding. They can also create hazards to boaters and other recreationalists when still attached to the bank, or when washed downstream with trailing cable. Alternative methods include burying an end of logs and debris, using rocks or other materials as ballast for the single features, and evaluating risks for the eventually loss of these features to downstream flow.

Rootwads can be placed as single features into the banks or bed of the channel with the root mass or root ball placed in the flow in the upstream direction (Sylte and Fischenich 2000), (Saldi-Caromile et al. 2004), (McCullah and Gray 2005). A rootwad is shown in Figure 10–1. Rootwads can be used successfully on many types of rivers. Woody debris and a rootwad feature, or the rootwad revetment presented in the next section, are considered to be more natural habitat features where there is large wood in the stream (McCullah and Gray 2005).

Due to bank instability, rootwads have limited application when the banks of the river are comprised of uniform sands. If there is less than 15% silt or clay in the bed and bank, the banks tend to erode (Sylte and Fishenich, 2000) potentially leading to the rootwad being dislodged by river waters. Habitat value is localized and is present as long as the rootwad is functional, remains in position, and the bank line does not erode and outflank the rootwad.

10.1.1 Design Procedure

Step 1. Embedment Length. The rootwad tree trunk (i.e. the "bole") should be attached to the rootwad fan (Sylte and Fischenich, 2000). The necessary embedment length is the criterion that determines the required length of the bole and footer log. The embedment length should be sufficient to maintain the position of the rootwad structure both vertically and laterally (Sylte and Fischenich, 2000). As a general rule, after the projected scour behind the rootwad, three quarters of the length should remain securely embedded. For streams with widths less than 15 ft, bole length can be as short as 10 ft, where larger streams may require an embedment length of 20 ft or more (Sylte and Fischenich, 2000).

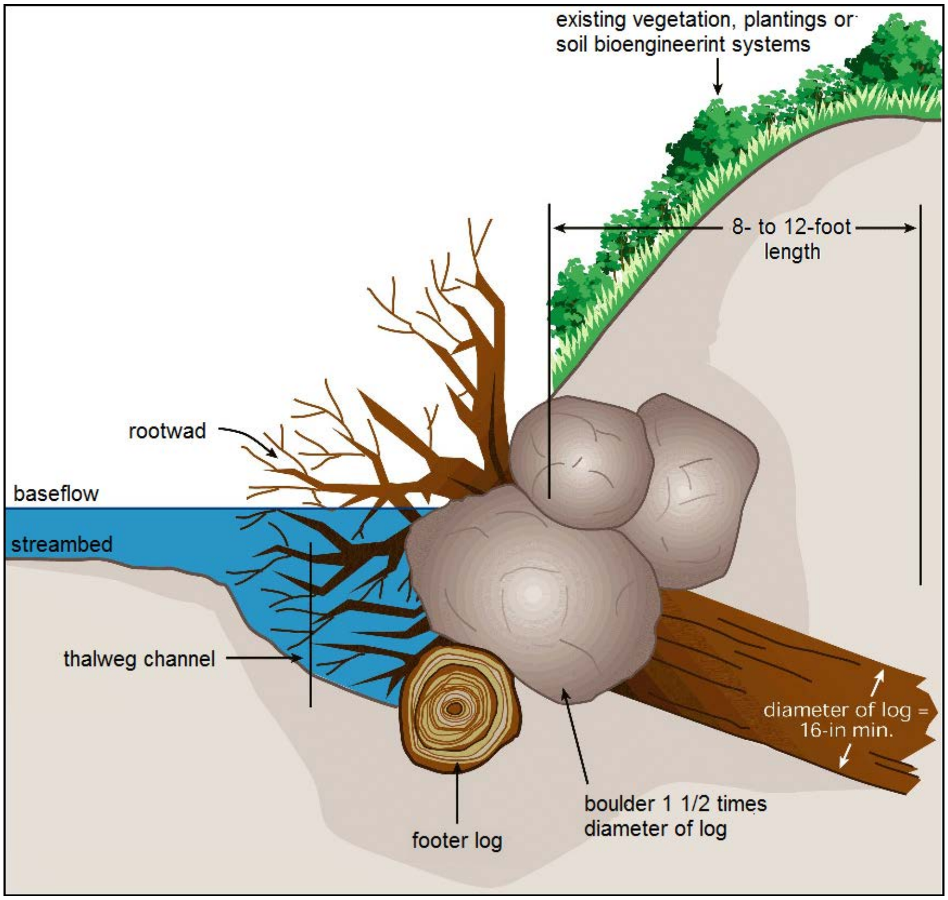


Figure 10–1. Typical rootwad installation (NRCS 1997).

Step 2. Stabilize rootwad in bank. Installation involves excavation into the bankline for the placement of rootwads. The excavated area can be more erodible even with re-compaction around the root wad tree trunk (bole). When the bank may be erodible after excavation, gravel material from river bars may be placed on the excavated bankline face to stabilize the bank. Refer to the next section on riprap revetments for additional guidance on rootwad installations.

10.1.2 Discussion and Recommendations

Risk and Failure. Large features can get caught in downstream bridges with small waterway openings creating stress on the structure and scour on the channel bed. This is not a problem at all sites and each structure should be evaluated individually based on factors including site conditions, flow regime, and debris characteristics.

Cables are not reliable and can cause erosion and safety hazards. The use of cables for securing woody debris or rootwads is not recommended.

10.2 Native Material Revetments

Native material revetments are a composite of rootwads with other logs and boulders to form an erosion resistant, continuous, interlocking wood material lining the bankline (McCullah and Gray, 2005; Sylte and Fischenich, 2000). Revetment materials move high velocity away from the bank reducing bank erosion (McCullah and Gray, 2005). Since the wood material usually decomposes within about 15 years, integrating live vegetation into the revetment is essential for long term erosion control (McCullah and Gray, 2005). Bio-degradable fabrics can also be used to stabilize planting areas while vegetation establishes sufficient root structure to provide erosion control. These revetments should only be used when failure of the revetment would not endanger lives or riverside infrastructure.

A native material revetment is shown in section view in Figure 10–2 and plan view in Figure 10–3. Native material revetments can provide bank protection in streams with high erosive forces, yet also provide inflow habitat and allow vegetation to establish on the banks. Due to bank instability, rootwads and rootwad revetments have limited application where the banks of the river are comprised of uniform sands (Sylte and Fischenich, 2000). If there is less than 15% silt or clay in the bed and bank, the banks have a tendency to erode (Sylte and Fishenich, 2000) potentially leading to structure being dislodged by river waters.

The site must be accessible to large construction equipment to place rootwad revetments and native material revetments usually require the use of heavy equipment for collection, transport and installation (State of Alaska, 2008). For flow depths greater than 3 ft, an underwater longitudinal stone toe may be necessary (Heaton et al, 2002). Bed elevation and slope need to be stable because these structures have a fixed elevation along the bank. Primary design considerations for rootwad composites are a) material dimensions including length and width, b) configuration including number of rootwads, spacing, orientation and entrance conditions, c) habitat requirements, d) revegetation, and e) failure mechanisms.

10.2.1 Design Procedure

Step 1. Determine if the geomorphology is suitable. There should be one stable meander, or a stable straight reach, upstream and downstream of the project.

Step 2. Determine applicable hydrology and hydraulics. Maximum scour depths around the structure should be calculated, and the design configuration should be checked to ensure the structures do not cause constriction scour.

Step 3. Locate suitable materials. Select 16 inch or larger diameter logs that are crooked and have an irregular shape. Select rootwads with numerous protrusions and eight (8)- to 12-foot long boles (tree trunk). Select irregularly shaped boulders as large as possible, but at least one (1) and one-half times the log diameter.

Step 4: Determine number of rootwads, footers and boulders. Based upon the dimensions of the located materials determine if two the number of rootwads and footer logs to be combined up the bank. Determine the number and size of boulders. If flow depths are less than 3 ft at the design discharge, no additional stone toe protection is usually necessary. If depths are greater than 3 ft, then an underwater stone toe revetment is necessary (Heaton et. al, 2002).

Material sizes primarily depend on stream size (Sylte and Fischenich, 2000). If one rootwad is not sufficient, two or more can be combined, provided that backfill and structural integrity are not jeopardized. The rootwad bole should be firmly attached to the rootwad fan (Sylte and Fischenich, 2000). The necessary embedment length dictates the length of the bole and footer log. The embedment length should be sufficient to maintain the position of the rootwad structure both vertically and laterally throughout its design life (Sylte and Fischenich, 2000).

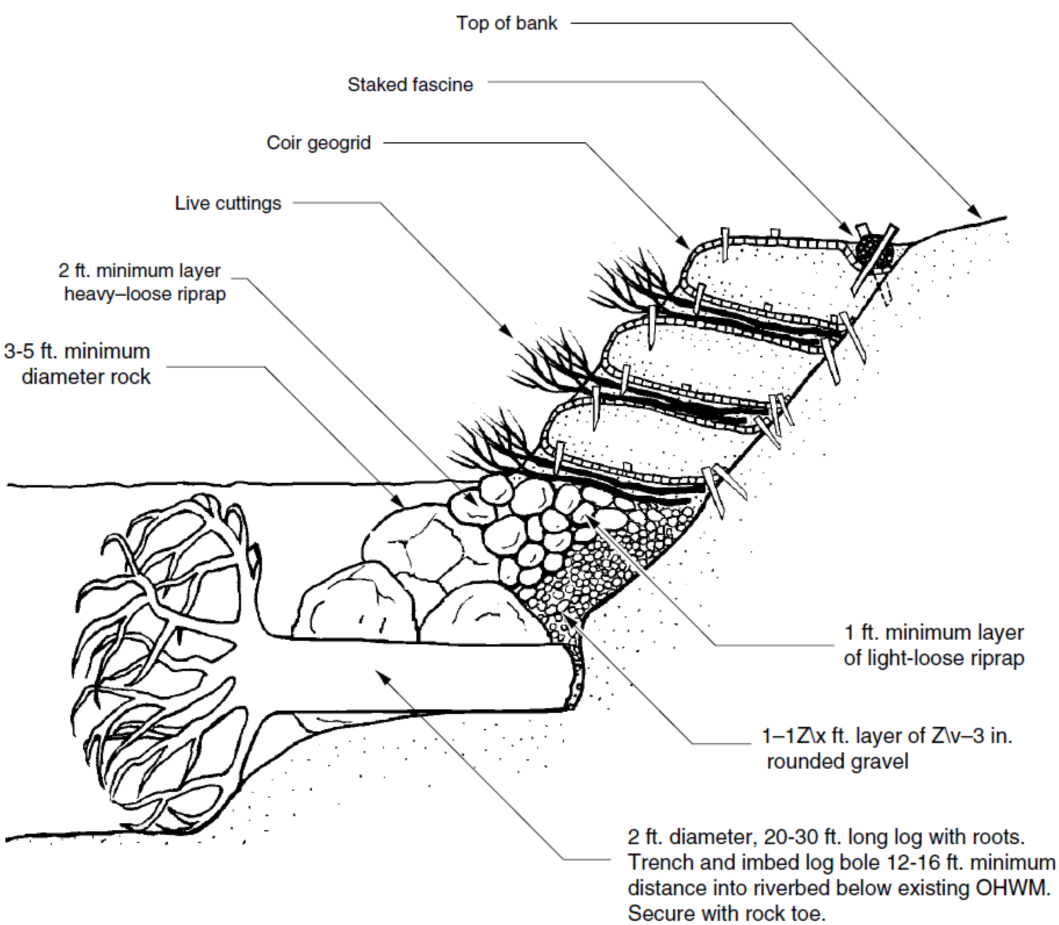


Figure 10–2. Rootwad and native material revetment, sectional view (Johnson et al. 2003). Shows three fabric encapsulated soil (FES) lifts. Coir fabrics can be used in lieu of geogrid to prevent soil erosion from inside the FES.

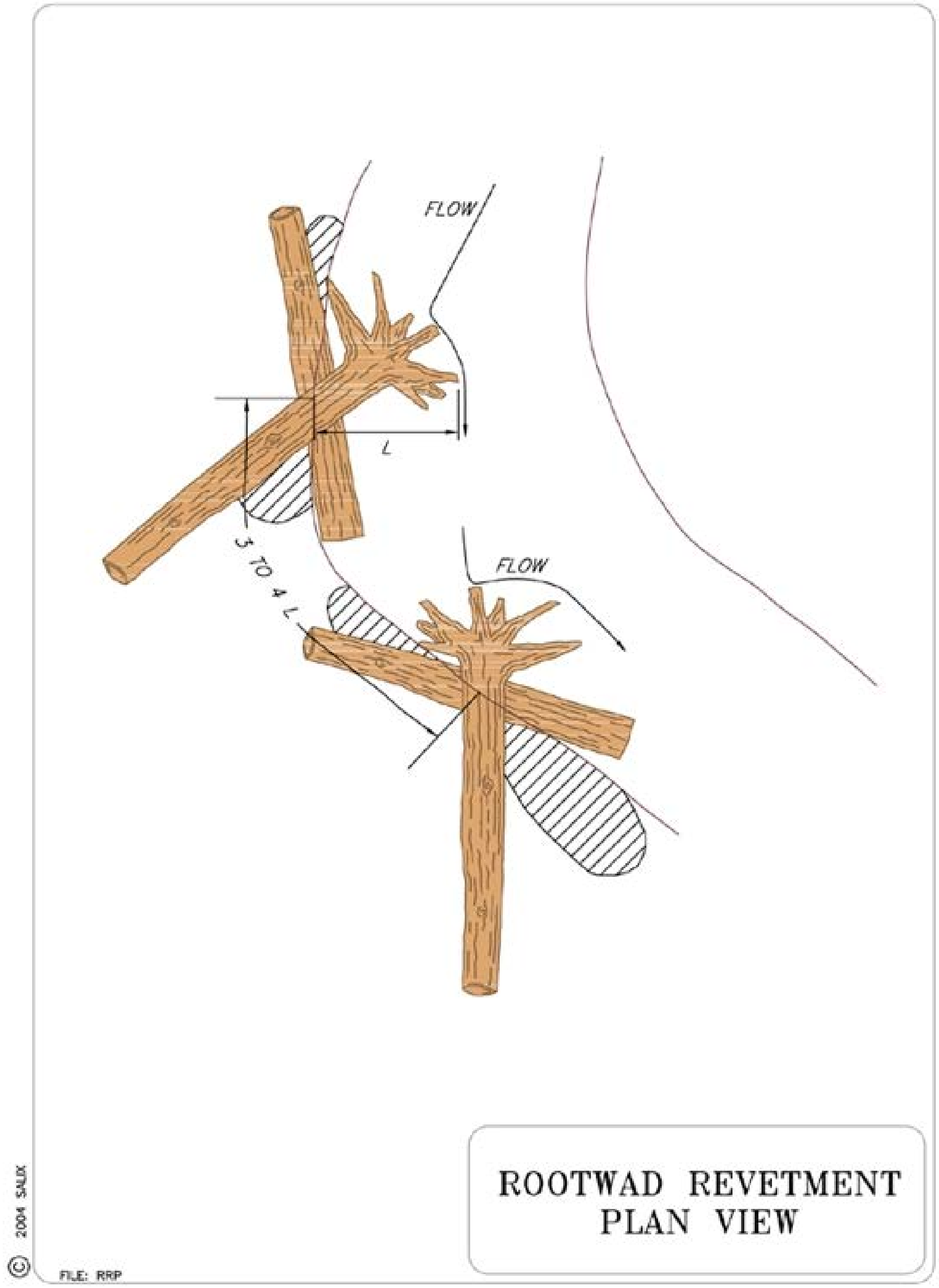


Figure 10–3. Rootwad revetment, plan view Used by permission from Transportation Research Board (Report #544), and Salix Applied Earthcare (NCHRP 2005).

Step 5. Design configuration and embedment length. Proper configuration of the rootwad fan in relation to flow and channel elevation is very important. The face of the rootwad fan must intersect the incoming velocity vectors at a 90-deg angle, but can be rotated as much as 15 deg toward the stream channel (away from the streambank). The rootwad fan should not be rotated towards the streambank or extend straight out into the channel or excessive bank erosion and failure may result (Sylte and Fischenich, 2000). The ends of the project should be tied into the banks (Heaton et al., 2002).

Generally, the rootwad fan should extend from the lowest scour elevation to the annual high water elevation. Prepare subgrade to a depth below the streambed that will protect against anticipated scour (at least one (1) and one-half the footer log diameter.) Design rootwads to withstand rootwad buoyancy, drag force, and frictional resisting forces. Methods can be found in Saldi-Caromile et al. (2004). Rootwads are often embedded in an excavated trench in the bank line. The excavated material placed on top of the rootwad trunk serves as ballast to offset buoyancy and drag forces on the rootwads.

Excavate log trenches parallel to stream flow. Footer logs should be spaced 4 to 6 ft apart. The footer logs should be keyed into the streambank (Heaton et. al., 2002). Excavate rootwad trunks perpendicular to footer logs. The rootwad should be above low flow elevations and the root fan should be above bank full elevations. Backfill trenches with native bank material (Heaton et al, 2002). Install high stage deflector logs in tight behind the rootwads and parallel to footer logs. Backfill with large field stones (Heaton et al., 2002).

Use boulders to anchor the footer log against floatation. Excavate trenches into the bank to accommodate the rootwad boles (eight (8) to 12 ft). Orient the trenches to allow placement of the root mass in a way that faces slightly towards the direction of flow and to allow the brace roots to be flush with the streambank. Backfill and combine the vegetative plantings and soil bioengineering systems behind and above the rootwads. These can include live stakes and dormant post plantings in the openings of the slope below bankfull stage. Place live stakes, fascines and other treatments above the revetment. If bankfill material will be subject to erosion flows, then treatment should be used to stabilize the banks above the revetment.

The face of the excavated trench along the bank line is subject to fluvial erosion during high river flows. This is especially a concern when the soil material is sandy and does not have binding clay or larger erosion resistant particles. Erosion of the face of the trench can be overcome by placing small riprap or cobble bed material to armor the bank, or back filling with large rock material. Placement of the rootwads on the inside of bends or in transition reaches will also reduce the erosion potential of the trench face.

From the rootwad fan, the bole is securely embedded into the stream bank. As described in the previous section, the three quarters of the length should remain securely embedded after erosion. When stream width is less than 15 ft, bole length can be as short as 10 ft An embedment length of 20 ft or more may be needed for larger streams (Sylte and Fischenich, 2000).

The footer log is positioned roughly parallel to the stream bank and is also securely embedded. Primarily, the footer log retains a more vertical stream bank and provides support for live transplants in the eddy zone area. It also provides additional lateral and vertical support for the rootwad bole and helps prevent minor settling and lateral movement. The angle between the footer log and rootwad bole is roughly parallel with the streambank, but can deviate provided that function is maintained. The rootwad is placed on the streamside of the footer log and the bole rests upon the footer log. The footer log will extend beyond the rootwad fan for a length sufficient to support the vegetation revetment. Vertically the footer log should be close to the scour depth.

Step 6. Determine spacing. As a general rule, a spacing of 3 to 4 times the projected length of the rootwad is adequate. As the radius of the bendway decreases, so should the rootwad spacing. For a radius of curvature divided by the width is less than about 2.5, the rootwads no longer deflect the flow and must effectively overlap to armor the bank.

Step 7. Design bank above the rootwads or revetment. Vegetation of the bank is crucial to the long-term success of this installation (McCullah and Gray, 2005). Vegetative efforts should be focused on those areas just upstream of the rootwad fan, and the area between the rootwad trunk and the footer log, where eddying may occur (NCHRP, 1975). Backfill and combine vegetative plantings and soil bioengineering systems behind and above the rootwads. These can include live stakes and dormant post plantings in the openings of the slope below bankfull stage. Place live stakes, fascines and other treatments above the revetment. If bankfill material will be subject to erosion flows, then treatment should be used to stabilize the banks above the revetment. Bio-engineering within and above the structure provides the best erosion protection as part of the revetment.

Step 8. Review constructability. Construction usually starts at the downstream end of the project area. Schedule installation for times which will least interfere with fishery and other instream functions.

10.2.2 Discussion and Recommendations

Native material revetments are intended to provide bank erosion control using naturally occurring materials including vegetation. They resist erosive flows and armor the bank. Care must be exercised that these structures are installed where there are stable upstream and downstream bends to prevent flanking, and where the river bed elevation and slope are stable. It is recommended that extensive bioengineering be incorporated into the revetment to provide erosion control after the logs and rootwads decompose.

Risk and Failure. The most prevalent method of failure of native material revetments is flanking. Flanking occurs when the stream moves around the structure and is caused by stream instability. Prevention of flanking can be achieved by avoiding areas of instability in the upstream or downstream direction (Sylte and Fischenich, 2000). At least one stable meander or straight reach should be upstream, and another downstream, of the project reach to ensure a consistent entrance and exit flow condition (Sylte and Fischenich, 2000).

Another method of rootwad failure is undercutting. Undercutting will occur if the roodwad is placed too high in the channel and flow scours the underlying soils. Other causes of undercutting include inadequate embedment (Sylte and Fischenich, 2000). A good way to prevent undercutting is to construct the rootwads to the scour depth, and provide a continuous footer log.

Where the banks of the river are comprised of uniform sands, rootwads have limited application (Sylte and Fischenich, 2000) due to bank instability.

Monitoring and Maintenance. The native material and rootwad revetment should be inspected on a yearly basis and after flood events. The structure should be examined for signs of undercutting or flanking, vegetation survival, and animal damage. (Sylte and Fischenich, 2000).

10.3 Engineered Log Jams (ELJs)

Traditional engineering approaches can be solved with non-traditional approaches including ELJs. ELJs are versatile for habitat and river engineering applications. A complete assessment of the reach and site as described in sections 3, 4, and 5 will aid the development of an ELJ design (Abbe et al., 2003).

Large woody debris commonly placed in streams can be categorized into three types: whole trees, logs, and root wads. A whole tree is a tree cut off at the stump with all or most of the limbs attached, including terminal branches. Logs are sections of the bole with all limbs removed referred to as stems. Root wads consist of the root portion of the tree and a section of the bole.

Abbe et al (2003) identified six types of ELJs: step jams or valley jams for channel control, bank stabilization, bench or flow deflection jams, flow diversion structures, bench or meander jams.

10.3.1 Design Procedure

A design procedure for ELJs is presented below and is an integration of recommendations from D'aoust and Millar (2000), Abbe et al. (2003), Abbe et al., (2005), and NRCS (2007c). General components of an ELJ include: structures built in the channel, logs (tree boles or stems), material for back fill, and principal structural members. The height, density and anchoring criteria also need to be specified.

Step 1. Watershed and reach analysis. This analysis includes investigations of the hydrology, sediment supply, channel dynamics and future land use. The hydrologic condition for design is commonly a 2-yr flow event. The channel dynamics investigation should determine existing and historical geomorphic conditions. Determine physical boundary conditions through a reach analysis at a spatial scale that is appropriate for the number of ELJ's in the design. See section 5.7 for details on the upstream and downstream limits of the engineered log jam features.

Step 2. Construct 1D hydraulic model with measured cross-sections. Determine water surface elevations and velocities for a range of flows.

Step 3. Determine type, number location and size of ELJs. In general, spacing of ELJ's should be 3-4 times the length of the ELJ structure (Sylte and Fischenich, 2000).

Step 4. Design foot print elevation and height of first ELJ. The design includes estimating the foot print and height (elevation) of the log jam. Common failure methods include undercutting with scour. This would involve placing ELJ base or footer at an elevation below maximum scour (Abbe et al., 2005). Empirical or analytical models can be used to predict scour below dikes and groins and the same equations can be used to predict scour below an ELJ. Additional scour estimation equations are described in Abbe et al. (2005). The elevation of the base or footer members would be the maximum scour subtracted from the lowest channel elevation in the thalweg.

The upper elevation of the ELJ should extend to the elevation of the lowest floodplain surface or to the bank full or channel-forming discharge. The length of groin type of ELJs would be the length of key member plus the pile of racked members, which extends upstream of the key member rood wads. The pile of racked logs should be of sufficient size and density to deflect flow around the ELJ and prevent scour from reaching key parts of the ELJ. The width of a groin-type of ELJ is used to specify length of orthogonal stacked members. The length of revetment type of ELJs will be related to the length of bank to be protected (Abbe et al., 2005). The width of an ELJ revetment would be the extent that stacked members extend into the bank and far logs and root wads of key members extend into the channel. The length will be the length of channel that is necessary for treatment. Extra design considerations should be given to upstream and downstream ends of revetments. Revetments should be close to the channel margins of the migration zone and be oriented parallel to the valley axis. Any ELJ structure should be evaluated for all possible future channel changes (Abbe et al., 2005).

Step 5. Design of key members. Rootwads and stems are critical to the design (Abbe et al., 2005). Project specifications should clearly show minimum dimensions and condition of trees. The largest members in diameter and length, and most symmetric rootwads should be used for key members (Abbe et al., 2005). In groin type of ELJs, key members are placed so the root wads of adjacent members are next to each other (Abbe et al., 2005).

Key members size could be estimated from similar sites and determining availability of trees to for key members. The average rootwad length and average fan (diameter of the root section), tree stem average diameter and length are necessary to determine ELJ stability. These dimensions are used to calculate the stability of key and other members based on drag, buoyancy, shear stress and friction shown in step 9 below.

Step 6. Design of footer log. Key footer logs in the correct position directly beneath and perpendicular to key member boles. Key footers should have a rootwad that is at least twice the base diameter of the outer key members of the ELJ (Abbe et al., 2005). The longest rootwad radius is oriented upward. This prevents the key member from sliding or rolling out of the ELJ. Key members are placed between the key footer and the stacked members. The length of the key footer is usually the width of the key member and also identical to stacked members that are oriented orthogonal to flow. Generally, the key footer bole should be no less than ¾ of the key member bole (Abbe et al., 2005).

Step 7. Design of stacked members. The number of stacked members should be increased to supplement key members in situations where key members do not have the necessary diameter or smaller than necessary to maintain ELJ stability. The general guideline is that stacked logs should have a basal bole diameter that is equal or greater than 80 percent of the corresponding key member diameter (Abbe et al., 2005). Stack member size is based on force balance analysis, then a decay model. Stacked members that are placed parallel to flow should have large symmetrical root wads. The number of stacked members should be large enough to create a large wall that extends from the floodplain to the maximum scour elevation below the channel (Abbe et al., 2005). Stacked members oriented normal to flow must have root wads that exceed bole diameters of logs above and below them. If no root wad logs are available, then notched logs could be used (Abbe et al., 2005).

Step 8. The layout design. Determine the placement of key footer members, key members, and stacked members. Typical ELJ flow deflector design examples are shown in Figures 10–4 and 10–5. The number of structural members (key footer and key members) as shown in Figure 10–5 is also important (Abbe et al., 2005). The maximum root wad diameter of stacked key members should equal or exceed average bankfull depth of the channel. The sum of the maximum rood wad diameters of key and flow parallel stacked members should equal or exceed twice the vertical dimensions of the ELJ in order to account for overlap (Abbe et al., 2005). For groin-type of ELJs, key members for a single layer underlain by 1 or 2 perpendicular key footers positioned downstream of the key member root wads. When key members do not meet specifications, then supplement with additional stacked members (Abbe et al., 2005). For groin types of ELJs, a sufficient number of stacked members are important to form a tightly packed pile.

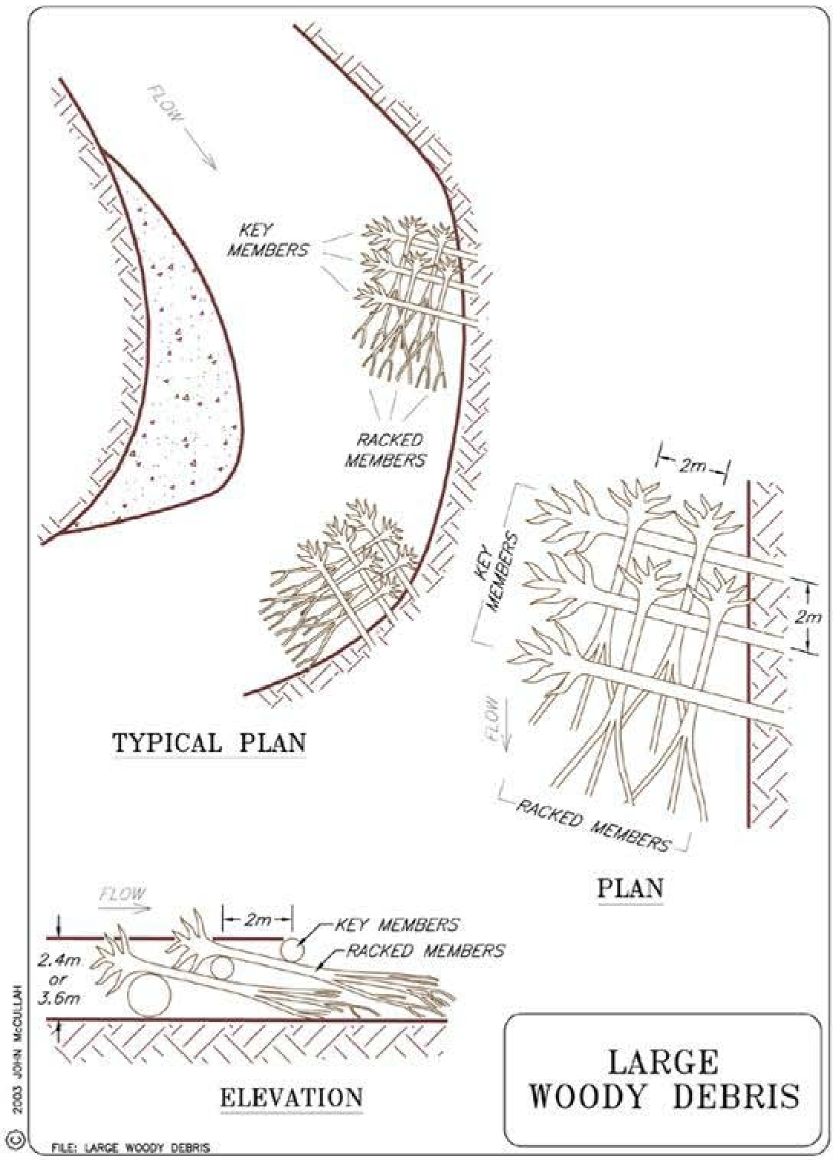


Figure 10–4. Large Woody Debris design details. Used by permission from Transportation Research Board (Report #544), and Salix Applied Earthcare (NCHRP 2005).

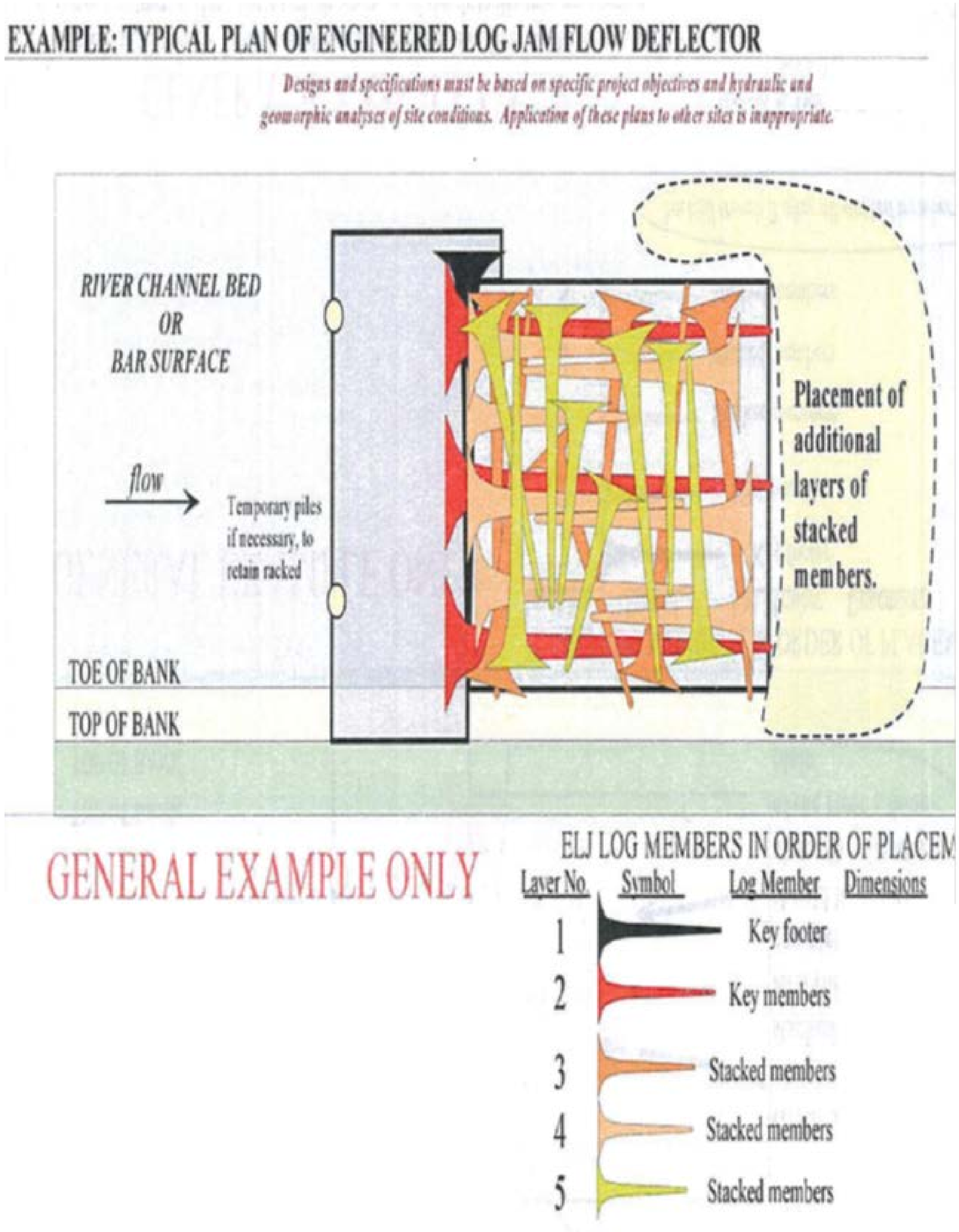


Figure 10–5. Engineered log jam (Abbe et al., 2005).

Step 9. Determine ELJ stability. Perform a force balance and determine the number and type of anchors, key, middle and top wood members.

The principal forces are as follows (D'aoust and Millar, 2000):

- F_{BL}—Net buoyancy force acting on the ELJ and transferred to the ELJ anchor (anchors can be log members, boulder (s), piles or earth anchors)
- F_{DL}—Horizontal drag force acting on the LWD and transferred to the ELJ anchor
- F_{DB}—Horizontal drag force acting directly on the ELJ anchor.
- F_{LB}—Vertical lift force acting directly on ELJ anchors.
- W'—Immersed weight of the ELJ anchors
- F_F—Frictional force at the base of the ELJ that resists sliding

Both buoyancy and sliding force analysis is needed for design. The net buoyancy force F_{BL} of key, stacked middle and top members is (D'Aoust and Millar, 2000):

$$F_{BL} = \left(\frac{\pi D_{TS}^2 L_{TS}}{4} + \frac{\pi D_{RW}^2 L_{RW}}{4} \cdot (1-p)\right) \rho_w g (1 - S_L) \cdot N_L \tag{10.1}$$

Where:

N_L = Number of pieces of wood (including key, stacked middle and top members).
D_{TS} = Tree stem average diameter (ft)
L_{TS} = Tree stem average length (ft)
D_{RW} = Average rootwad diameter (ft)
L_{RW} = Average rootwad length (ft)
p = Proportion of voids in rootwad, equal to about 0.2 (Wright, 2003)
g = Gravity acceleration (32.2 ft/s^2)
ρ_w = Density of water (1.94 slugs/ft^3)
S_L = Specific gravity of large woody debris (LWD).

S_L values are dependent upon specific species and moisture content. S_L Typical values for S_L would range from 0.3 for cotton wood (Wright, 2003) to 0.5 for coniferous species such as Douglas fir (D'Aoust and Millar, 2000). Considering members subjected to long term submergence would become waterlogged, S_L values may be as high as 0.8 and 0.9. S_L values of 0.3 to 0.5 would represent LWD at the time of placement prior to submergence and should be used in design representing the lower end of the probable range (D'Aoust and Millar, 2000).

The immersed weight of an anchor boulder is:

$$W' = \frac{\pi D_B^3}{6} \rho_w g (S_s - 1) \tag{10.2}$$

Where:

S_s = Specific gravity of anchor boulders ~2.65

D_B = Anchor boulder diameter

The magnitude of the lift force acting on anchor boulders is calculated using:

$$F_{LB} = C_{LB}\rho_w \frac{V^2}{2}\frac{\pi D_B^2}{4} \quad (10.3)$$

Where:

C_{LB} =Lift coefficient, equal to 0.17 (Cheng and Clyde, 1972).
V =Mean cross sectional velocity (m/s)

The factor of safety with respect to buoyancy is equal to:

$$FS_B = \frac{\sum W'}{\sum F_{BL}+\sum F_{LB}} \quad (10.4)$$

Where:

FS_B = Buoyancy factor of safety which should be 1.5 minimum and 2.0 or greater (Wright, 2003; and D'Aoust and Millar, 2000) for the design to be considered safe.

The horizontal sliding force is based on drag and frictional force at the base that resists sliding. The horizontal drag force on an ELJ (Wright, 2003) is:

$$F_{DB} = C_D^{app} A_{ELJ}\frac{V^2}{2}\rho_w \quad (10.5)$$

Where:

C_D^{app} =Drag coefficient (approximately 1.2 (D'Aoust and Millar, 2000))
A_{ELJ} =Projected area (perpendicular to the main flow direction) obstructed by the ELJ

The horizontal streambed friction resistance on an ELJ can be calculated using:

$$F_F = (W' - F_{BL} - F_{LB})f \quad (10.6)$$

Where,

f = friction factor of logs on the streambed (approximately 0.78 (Wright, 2003))

The factor of safety from sliding equals:

$$FS_s = \frac{\sum F_F}{\sum F_{DB}} \tag{10.7}$$

Where:

FS_s = Sliding factor of safety which should be 1.5 minimum and 2.0 or greater (Wright, 2003; and D'Aoust and Millar, 2000) for the design to be considered safe.

Note: Soil ballast was not considered in this analysis and represents an additional factor of safety.

Step 10. Compute Bed scour. Scour can be determined Julian (2002) using:

$$d_s = 1.1\left(\frac{L_e}{d_1}\right)^{0.4} Fr^{0.33} d_1 \tag{10.8}$$

Where:

d_s = Depth of scour below channel thalweg (ft)
L_e = Effective length of log jam protruding into flow (ft)
d_1 = Average upstream flow depth in channel (ft)
Fr = Froude number upstream of ELJ (HEC-RAS Output)

Step 11. Re-design and re-check forces. Re-design the placement and layout of key and stacked members and anchor logs (or other anchoring) if needed, and re-calculate the force balance until the forces on each ELJ are balanced with the resisting forces of the ELJ plus a factor of safety.

Step 12. Design backfill and grading plan.
Bank areas excavated for placement of key ELJ members should be backfilled and compacted in lifts applying moisture to maximize consolidation which will provide greater key member stability. Banks should be graded to as near as native grade as practical.

Step 13. Develop upper slope vegetation plan. Re-vegetation should be with native species representing the pre-project condition. Depending upon climate conditions and weather patterns, irrigation may be required, along with re-planting. If beavers or cattle are known to use the area, fencing trees or the re-vegetation area may be required.

Step 14. Investigate log acquisition. Native source trees that are indigenous to the local area will be imported to the site. These trees should be decay resistant. A list of trees is shown in Table 10–2, with information on decay rates:

Table 10–2. Tree Species and Desirability

Source: NRCS Technical Supplement 14J, 2007.

Species	Durability (assuming wetting and drying)	Source of information[1]
Cottonwood (*Populus* spp.)	Poor	Johnson and Stypula (1993)
Alder (*Alnus* spp.)	Poor	Johnson and Stypula (1993)
Maple (*Acer* spp.)	Fair (will survive 5 to 10 yr)	Johnson and Stypula (1993)
Hemlock (*Tsuga* spp.)	Least durable of conifers	Johnson and Stypula (1993)
Sitka spruce (*Picea sitchensis*)	Excellent	Johnson and Stypula (1993)
Douglas-fir (*Pseudotsuga* spp.)	Excellent (will survive 25 to 60 yr) 32–56 yr	Johnson and Stypula 1993); Harmon et al. (1986)
Western red cedar (*Thuja plicata*)	Most desirable (will survive 50 to 100 yr)	Johnson and Stypula (1993)
Yellow-poplar (*Liriodendron tulipifera*)	0.4 yr	Harmon et al. (1986)
Aspen (*P. tremuloides*)	5 yr	Harmon et al. (1986)
White fir (*A. concolor*)	4 yr	Harmon et al. (1986)
Norway spruce *(Picea abies*)	~30 yr	Kruys, Jonsson, and Stahl (2002)
Conifers (*P. sitchensis, T. heterophylla, P. menziesii, T. plicata*)	Half-life of ~20 yr	Hyatt and Naiman (2001)
Black locust, red mulberry, Osage orange, Pacific yew	Exceptionally high heartwood decay resistance	Simpson and TenWolde (1999)
Old growth baldcypress, catalpa, cedars, black cherry, chestnut, Arizona cypress, junipers, honeylocust, mesquite, old growth redwood, sassafras, black walnut	Resistant or very resistant to heart wood decay	Simpson and TenWolde (1999)
Young growth baldcypress, Douglas-fir, western larch, longleaf old growth pine, old growth slash pine, young growth redwood, tamarack, old growth eastern white pine	Moderately resistant to heartwood decay	Simpson and TenWolde (1999)
Red alder, ashes, aspens, beech, birches, buckeye, butternut, cottonwood, elms, basswood, true firs, hackberry, hemlocks, hickories, magnolia, maples, pines, spruces, sweetgum, sycamore, tanoak, willows, yellow-poplar	Slightly or nonresistant to heartwood decay	Simpson and TenWolde (1999)

[1] Information from Johnson and Stypula (1993) is qualitative and unsubstantiated. Evidently, these comments pertain to the region of King County, Washington. Harmon et al. (1986) provide a review of scientific literature dealing with decomposition rates of snags and logs in forest ecosystems. The times from Harmon et al. (1986) represent the time required for 20 percent decomposition (mineralization) of a log based on exponential decay constants obtained from the literature. Fragmentation of logs in streams due to mechanical abrasion would accelerate the decay process, as would more frequent wetting and drying. Kruys, Jonsson, and Stahl (2002) provide data on decay of fallen and standing dead trees in a forest in mid-northern Sweden. Hyatt and Naiman (2001) provide data on residence time of large wood in Queets River, Washington. Simpson and TenWolde (1999) provide data for evaluating wood products, not whole trees.

Step 15. Evaluate site dewatering and other constructability issues. Constructability issues are very site specific. Often dewatering is required using cofferdams to excavate the bed and banks and place all of the ELJ members. Access routes need to be determined which are suitable for the type of equipment and trucks needed to haul LWD members to the stream bank unless than are available locally, in which case equipment will still be required for placement.

Step 16. Complete final design plans and prepare specs. Careful preparation of the final design drawings and specifications is required to ensure constructed product meets functional requirements.

10.3.2 Discussion and Recommendations

Logs used for constructing ELJ's decay over time. The longevity of the tress species used in ELJ construction should be accounted for in long term project planning. Sylte and Fischenich (2000) indicate that tree species have an effective life span in streams between 5 and 15 years that is general dependent on species but is also impacted by wetting and drying cycles (Table 10–2). Depending upon the longevity of log species, additional measures may be needed at a later time to maintain bank stabilization benefits.

Risk and Failure. Failure of ELJ's is primarily due to flanking and scour. Engineered Log Jams can have a high rate of failure (Frissell and Nawa, 1992). Larger structures can cause accelerated downstream bank erosion unless properly spaced and positioned.

The modes of failure of structures examined in southwestern Oregon and Washington (Frissell and Nawa, 1992) included failure of cables and anchoring devices and also through bank erosion and bed load deposition (these Guidelines recommend cables should not be used as a river construction material). There was an average of 40 percent failure rate in southwest Oregon and 6 percent in southwest Washington. Rates of damage were high in streams that were larger and wider.

The most extensive structural damage trend was found to occur in in wide low-gradient reaches in alluvial valleys and fans (Frissell and Nawa, 1992). There were multiple failure modes but mostly the structures were damaged by watershed changes, morphologic changes, upstream bank erosion and increased sediment supply from road failures. Notable adverse ELJ effects included anchor log erosion or dislodgement.

Monitoring and Maintenance. Monitoring is either periodic or event oriented monitoring on all types of hydraulic structures (Abbe et al., 2005). Generally, all structures have a monitoring plan. The routine monitoring schedule might include a schedule: 1) six months after completion, 2) end of year one, two, three etc. The structure should also be checked after the first bankfull discharge (every year for the first five years) (Abbe et al., 2005).

A separate inspection should occur after every major hydrologic event such as the 10 year or greater frequency. Performance monitoring after five years would only occur for any critical conditions (Abbe et al., 2005).

General performance monitoring should include some of the following factors (Abbe et al., 2005):

- Has structure changed size?
- Has structure increased in height and size by adding material?
- Is there any vertical settling?
- What is extent of water flow through the structure?
- Has stream orientation changed or is there any danger of structure abandonment or flanking?
- Has bank erosion occurred?

10.4 Boulder Clusters

Boulder groupings can be used in conjunction with vegetated banks and bank hardening stabilization methods to provide habitat elements and flow diversity. They can also be used at the toe of bank slopes to disrupt secondary flow currents and reduce erosive that is undercutting the banks. Scoured pools can also form downstream of these features, providing fishery habitat, but large scour formations in locations of high velocity can contribute to boulder movement. The use of boulders should be compatible with local features since their presence may be odd in areas that do not have rocky features. The occurrence of this error may be limited by the high costs of transporting boulders to the site from another region.

10.4.1 Design Procedure

Each boulder cluster should be designed by applying a force balance procedure.

Step 1. Develop a hydraulic and sediment model for the project reach. When selecting locations for boulder clusters, consider local hydraulic changes in velocity and water surface elevations, and also sediment loads (WDFW, 2004) when determining the locations for boulder clusters. A 2D flow model will provide more detail on non-uniform velocity distributions, while a 1D model can provide cross-sectional, average velocity information over a greater longitudinal distance.

Step 2. Locate boulder cluster sites. The stream should be evaluated during low flow, normal and high flow conditions to determine the thalweg, existing habitat and best location and configuration for boulder placement. Each reach should be classified into pool, run, or riffle, and assessed for:

- Length of pool, run, or riffle.
- Mean depth of each habitat class.

- Percent instream protruding boulders.
- Percent instream logs and debris.
- Percent overhead cover > 3 ft from the surface.
- Local and cross section average velocity.
- Substrate composition (% by class or gradation).

Also inspect the reach for fish and invertebrate samples. The design should only be utilized in an area where limited cover is present and should include as few boulders as possible. Boulders should occupy less than 10 percent of the flow area at bank-full flow (Fischenich and Seal, 1999), or less than one-fifth of the channel width (Barton and Cron, 1979). Avoid placing boulders in low velocity regions like pools and slow runs. Velocity should exceed 1.2 m/s (4 ft/s) during events that fill the base flow channel. The clusters are not recommended for sand bed streams, braided river, or unstable sections. Boulders should be sized for stability at bank full flow and should not be placed at the upper end of riffles.

Step 3. Determine boulder size using force and moment stability analyses. A boulder that is immersed in flowing water will experience the hydrostatic forces of pressure, and the body forces of weight and buoyancy (F_w) and (F_b), and the additional hydrodynamic forces of pressure and viscous shear forces tangent to the body surface (Fischenich and Seal, 2000). The normal and tangential forces can be resolved in the drag force (F_d) and the lift force (F_l). The boulder will remain at rest as long as the active forces of drag, lift and buoyancy are less that the resistive forces of weight and friction (Figure 10–6).

<u>Drag and Lift Force Analysis</u>. Drag and lift forces are functions of the approach velocity raised to the second power. Velocity or shear stress are sometimes used to analyze the stability of a boulder. Critical shear stress and velocity values for boulders, cobbles and gravel are shown in Table 10–3 (Fischenich and Seal, 2000).

For a submerged boulder with turbulent flow over a rough horizontal surface, incipient motion occurs when:

$$d_s = \frac{(18yS_f)}{(G-1)} \tag{10.9}$$

Where

d_s = minimum boulder diameter (ft)
S_f = friction slope (ft)
y = water depth (ft)
G = specific gravity of the boulder (approximately 2.65)

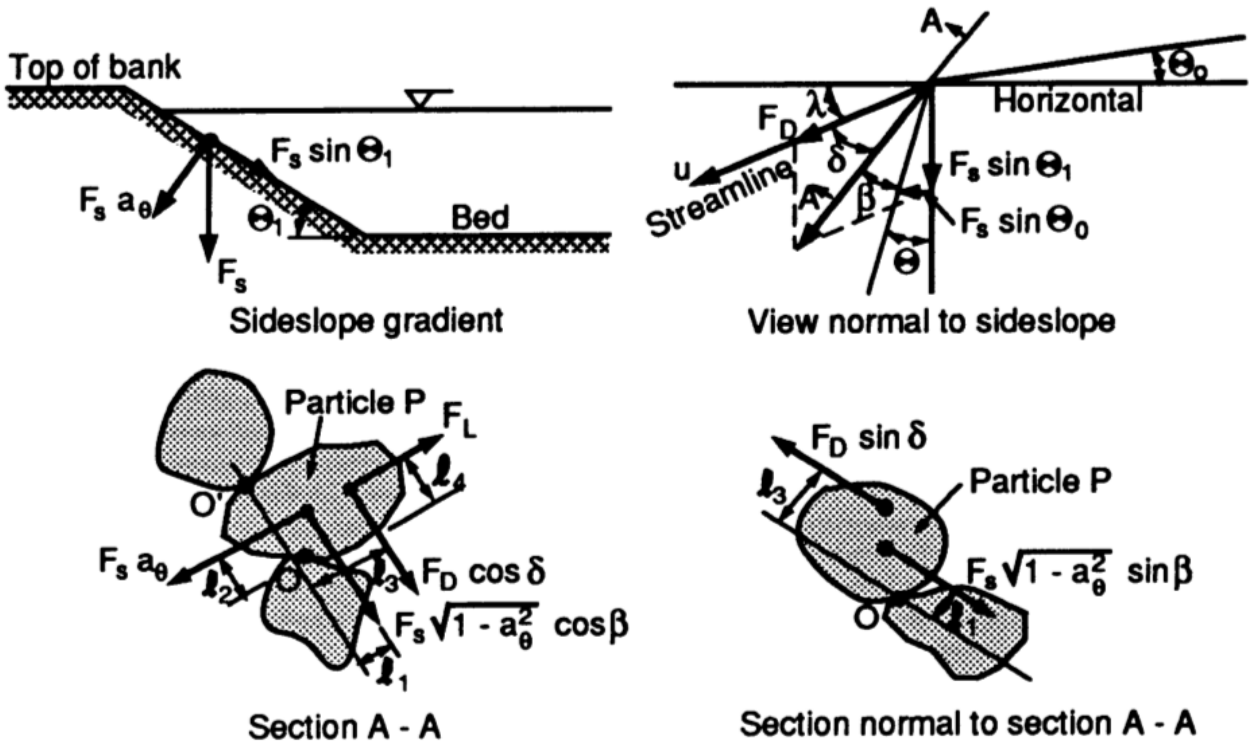


Figure 10–6. Drag and lift force analysis of boulders/particles (Fischenich and Seal, 2000).

Table 10–3. Threshold Conditions for Boulder Movement

Source: Fischenich and Seal (2000), Julien (1995)

Class name	Ds (in)	ϕ (deg)	τ_{*c}	τ_c (lb/sf)	V_c (ft/s)
Boulder					
Very large	>80	42	0.054	37.4	25
Large	>40	42	0.054	18.7	19
Medium	>20	42	0.054	9.3	14
Small	>10	42	0.054	4.7	10
Cobble					
Large	>5	42	0.054	2.3	7
Small	>2.5	41	0.052	1.1	5
Gravel					
Very coarse	>1.25	40	0.050	0.54	3
Coarse	>0.63	38	0.047	0.25	2.5

Moment Force Analysis. Table 10–2 and Equation 10.9 can be used for a preliminary analysis to determine approximate dimensions of a stable boulder. However, analyses that are more detailed are necessary. In a moment stability analysis, a single boulder is evaluated based on the ratio of moments resisting overturning to the moments causing overturning of the particle about the point of contact. The ratio of moments resisting overturning M_r , to those causing overturning represents the:

$$Safety\ Factor\ SF = \Sigma M_r / \Sigma M_p \tag{10.10}$$

Ratios greater than one indicate stable riprap (Fischenich and Seal, 2000, Julien, 1995).

A generalized moment stability analysis is also presented for analysis of the stability of boulders placed on side slopes and including streamlines not parallel to the channel (Fischenich and Seal, 2000; Julien, 1995). Figure 10–7 shows the forces acting on the boulder resting on the bed or bank of a stream an inclination angle θ_2 (Fischenich and Seal, 2000; Julien, 1995). For water slope less than 0.1, the buoyancy forces and be deducted from the boulder weight to get a submerged weight of the boulder. This would be shown as $F_s = F_w - F_b$ The other forces are defined above. The streamline is allowed to deviate from the horizontal by an angle λ to account for secondary currents. In a straight section $\lambda = 0$. The direction the boulder could move if it becomes unstable is described by the angle β. Based on simple geometric relations, the following equation is defined (Fischenich and Seal, 2000; Julien, 1995):

$$a_\theta = \sqrt{cos^2\theta_1 - sin^2\theta_0} \tag{10.11}$$

$$tan\theta = \frac{sin\theta_0}{sin\,\theta_1} \tag{10.12}$$

Using these two relations and given the angle of repose for the boulder ϕ, ($\phi = 42$), the moment arms can be determined from (Fischenich and Seal, 2000, Julien, 1995):

$$A = \left(\frac{l_4}{l_2}\right)\left(\frac{F_l}{F_S}\right) and\ B = \left(\frac{l_3}{l_4}\right)\left(\frac{F_D}{F_S}\right) \tag{10.13}$$

The moment arms are defined in the figure above. Using these relations the following four equations can be used to determine the safety factor: (Fischenich and Seal, 2000), Julien, 1995)

$$SF = \frac{a_\theta tan_\phi}{\eta_1 tan_\phi + \sqrt{1 - a_\theta^2 cos\beta}} \tag{10.14}$$

$$\eta_{1} = \eta_2 \left[\frac{\left(\frac{A}{B}\right) + sin(\lambda+\beta+\theta)}{1+\left(\frac{A}{B}\right)} \right] \tag{10.15}$$

$$\eta_0 \cong \frac{18_\tau}{(\gamma_{s-}\gamma_w)d_s} \tag{10.16}$$

$$\beta = \ tan^{-1} \frac{cos(\lambda+\theta)}{\left[\frac{(A+B)\sqrt{1-a_\theta^2}}{B\eta_0 tan\phi} + sin(\lambda+\theta) \right]} \tag{10.17}$$

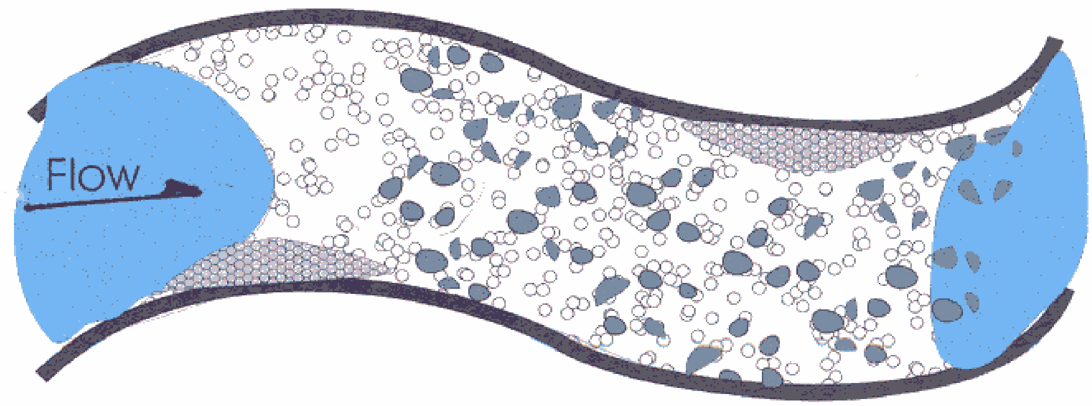


Figure 10–7. Plan view showing boulder placement between riffles (WDFW (2004).

The assumption is that $A = B$ because equations are not sensitive to this ratio. These equations are only valid when λ is less than zero. If the boulders are placed in a bend and secondary currents occur up the bank, a different equation is required (Fischenich and Seal, 2000).

Step 4. Design boulder configuration and spacing. Avoid placing boulder groups near the upper end of riffles. Concentrate boulders in or near the channel thalweg to ensure habitat availability during low flow, but place them well away from either bank. Avoid placements that will deflect flows toward erodible banks.

Boulder clusters can be configured to trap or pass large woody debris as desired. If trapping debris is desirable, boulders should be located downstream from source areas, should protrude above the water surface, and gaps between boulders should be narrower than the length of debris to be trapped. Conversely, more streamlined, lower configurations may be used if debris passage is desirable.

Boulders can be placed in entirely random configurations, in diamond-shaped clusters of four, or in such a way as to create step-pool sequences in steep channels. In larger streams, boulders are placed in riffles in clusters of five with the largest boulder set at the head of a cluster. Optimal spacing between boulders within a cluster is 1.5-3 ft, with 9 ft between clusters. The optimum configuration of clusters is a “staggered” pattern, as shown in Figure 10–8. Placement of boulder clusters in the uppermost riffle zone, as depicted in Figure 10–9, is avoided because these boulders tend to frequently in-fill with bed-load gravels, eliminating their effectiveness (Fischenich and Seal, 2000).

Step 5. Investigate boulder acquisition. It can be difficult and challenging to find boulders of sufficient mass to withstand high flow events. The quality of the boulder is also important and mass should not degrade over time. Trucking boulders is expensive, making the ideal option to locate boulders onsite or within the immediate area.

Step 6. Assess constructability. Specific placement of large boulders in a stream may require heavy machinery with a thumb. Determine if machinery has access to the site. Check windows of operation for the equipment in and around the stream. Locate potential stock pile sites, and re-visit the main issue of acquiring boulders in the immediate area.

10.4.2 Discussion and Recommendations

Boulders that move or dislodge a few ft are not causing stability problems. If however, the flow event that shifted the boulders is less than the design flow event, and there is significant movement, this is a more substantial concern that should be inspected and assessed. Boulders placement should be assessed during design to ensure the configuration is not causing constriction scour and increased erosive forces that also cause more boulder movement. A change in the channel alignment that bypasses the boulders should also be inspected and assessed at that time to determine if it is feasible and necessary to do maintenance in the form of moving boulders.

Risk and Failure. Boulders pose a low risk to existing habitat, but temporary losses of habitat value may occur with the rearrangement of gravels. When boulders are placed low in the channel profile, little risk is posed to infrastructure and property. Boulders may pose a risk to boaters. Risks of boulder designs are great because of the uncertainty in boulder placement. Boulder placements should also consider local hydraulic changes in velocity and water surface elevations and also sediment loads (WDFW, 2004).

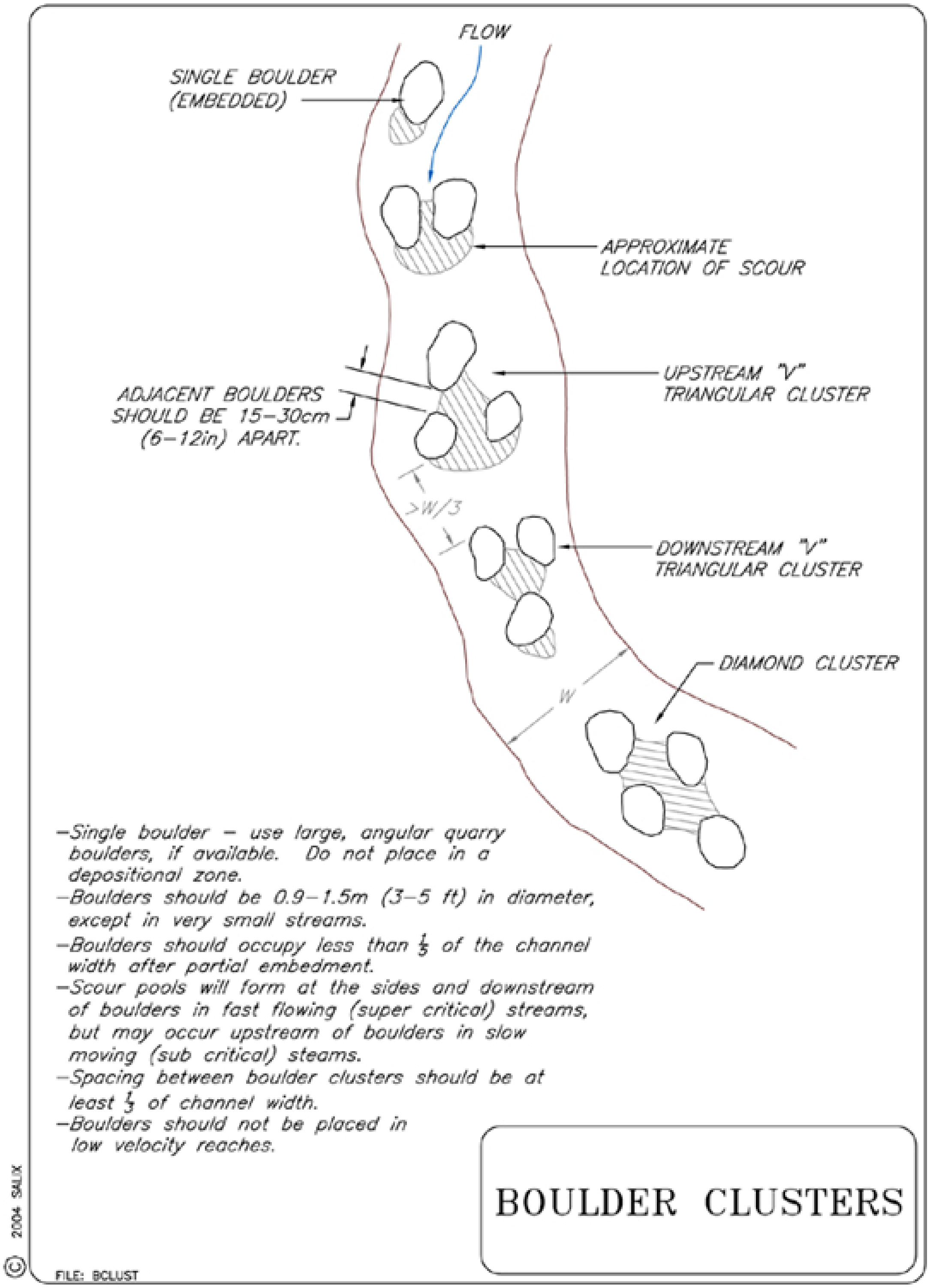


Figure 10–8. Boulder clusters. Used by permission from Transportation Research Board (Report #544), and Salix Applied Earthcare (NCHRP 2005). Velocity should exceed 1.2 m/s (4 ft/s) during events that fill the base flow channel (low flow velocity is less than 1.2 m/s (<4 ft/sec).

Monitoring and Maintenance. Monitoring should address the following questions (WDFW, 2004):

- Is maintenance required?
- Did the treatment help fish production? Do the boulder clusters favor fish habitat?
- How has habitat changed since addition of the boulders? Has pocket pool habitat been established? Have any of the boulders migrated downstream? If so do any need to be-positioned to provide pocket pool habitat?
- Is the project low or high risk?

If the project is low risk than an annual site visit and a documentation of qualitative observations regarding scour, deposition, fish use, and boulder stability may be all that is required. If the project is high maintenance then, projects that pose a relatively high risk to infrastructure, property, or habitat may require frequent quantitative physical and biological surveys to be conducted (WDFW, 2004). Such surveys may include photos and detailed pre- and post-construction surveys of boulder locations; bed and bank topography to document changes over time; pre- and post-construction snorkeling of the site; and a reference reach to document fish use.

Regular maintenance should be completed after high flow events. Maintenance may include replacement of boulders (WDFW, 2004).

11 Channel Relocations/Construction

It is often beneficial to design a relocated channel as close to the final stable configuration as can be determined by the designer. This will minimize natural channel adjustments that occur immediately following construction, although the channel can still be responsive in later years to adjustments in the flow or sediment regime. To this end, the designer should identify ongoing channel processes, and the point of balance that is stability for the reach. When planning habitat restoration projects on streams where habitat has degraded, the designer should identify the reasons for the instabilities (Kondolf, 1990). With the cause of channel instability in hand, the next steps are to evaluate, design and implement channel restoration/stabilization methods.

Relocating channels is often a means of re-establishing or expanding floodplain by locating the channel in a less congested area. The relocation may provide more access and conveyance area for overbank flow, and/or area for wider channels with laid-back bank slopes and higher width-to-depth ratios. With a well-considered design, there should be a minimal transition period for the channel to stabilize. This period is considerably shorter than a project which relies on multiple high-flow events to carve out a stable configuration. Channel relocations can be a cost effective means of stabilizing banks and eliminating long-term maintenance costs.

If there is sufficient land available, channel relocations can also increase bank stability by restoring meander bends. Extending the channel length in a restored meander bend can result in increased hydraulic roughness, decreased bed load transport rates, decreased slope, and general increase in channel stability (Brookes, 1996; Brookes et al., 1996; McCullah and Gray, 2005).

Re-establishing a river channel in historic oxbows, especially those disconnected as a result of past channelization, can have the same benefits as channel relocation to restore meander bends. In reaches where there is active incision/degradation due to sediment supply being less than the transport capacity, a longer channel length can reduce or eliminate continued incision. Reestablishing oxbows are discussed in this chapter as another method of channel relocation. Reactivated oxbows are also useful for creating and maintaining side channels as presented in section 8.3.

Channel processes are re-established where new areas of bank erosion and deposition occur naturally, depending upon the channel curvature, local soil types, geology, and geomorphology (McCullah and Gray, 2005); thus, new areas where site-specific bank erosion issues will occur are also possible. Relocated channels can be stable provided that the sediment transport capacity is in balance with supply. If the relocated channel significantly changes the hydraulics upstream of the relocated reach, then there should be an investigation into the effect of this change. Baird (2015) provided recommendations on the width of relocated channels and design considerations.

Increasing meanders can also rehabilitate ecological functions and increase visual diversity (McCullah and Gray, 2005). Habitat complexity can be increased by creating connected flood plain/wetted areas for fishery egg entrainment and larval development, and variable depth and velocity habitat is created for nursery or rearing habitat. Environmental benefits can be realized as a result of the formation of a new channel, while the abandoned channel can become a wetland, a backwater area, or a high flow secondary channel.

Three approaches to determining relocated meander channel characteristics are outlined in Table 11–1. Channel characteristics for anabranched or braided channels will be distinct from meander channels as introduced in Chapter 4, Hydraulic Assessment of Energy, River Form and Shear Forces. One source of characteristic values for meander, annabranched and meander river forms is The Federal Interagency Stream Restoration Working Group (1998). Characteristic values for the new channel can be selected during the development of the new channel design.

11.1 Design Procedure

Project design criteria, ecological factors, hydrology, geomorphic factors and general hydraulic and scour factors may have been assembled previously to aid in selecting a suitable bank stabilization method. Important aspects of design criteria, hydrology, and permitting are described in chapter 2. Habitat (chapter 2) and floodplain mapping should also be available (WDFW, 2003) from the initial investigation. Assess the watershed for changes such as urbanization, fires, agricultural impacts, and historic changes (WDFW, 2003) prior to initiating design of relocated channel. Determine the geomorphic factors including sediment continuity issues that may impact channel stability as discussed in chapter 3. Define channel form and energy level, the general hydraulics for the study area including energy (chapter 4) and the potential for scour (chapter 5) that may influence the channel and the extent of the floodplain.

Table 11–1. Three Approaches to Achieving Final Design (from Shields 1996)

Approach A		Approach B (Hey 1994)		Approach C (Fogg 1995)	
Task	**Tools**	**Task**	**Tools**	**Task**	**Tools**
Determine meander geometry and channel alignment.[1]	Empirical formulas for meander wavelength, and adaptation of measurements from pre-disturbed conditions or nearly undisturbed reaches.	Determine bed material discharge to be carried by design channel at design discharge. Compute bed material sediment concentration.	Analyze measured data or use appropriate sediment transport function[2] and hydraulic properties of reach upstream from design reach.	Compute mean flow, width at design discharge.[4]	Regime or hydraulic geometry formulas with regional coefficients.
Compute sinuosity, channel length, and slope.	Channel length = sinuosity × valley length. Channel slope = valley slope/ sinuosity.	Compute mean flow, width, depth, and slope at design discharge.[4]	Regime or hydraulic geometry formulas with regional coefficients, or analytical methods (e.g. White et.al. 1982 or Copeland 1994).[3]	Compute or estimate flow resistance coefficient at design discharge.	Appropriate relationship between depth, bed sediment size, and resistance coefficient, modified based on expected sinuosity and bank/berm vegetation.
Compute mean flow width and depth at design discharge.[4]	Regime or hydraulic geometry formulas with regional coefficients, and resistance equations or analytical methods (e.g. tractive stress, Ikeda and Izumi 1990 or Chang 1988).	Compute sinuosity and channel length.	Sinuosity = valley slope/channel slope. Channel length = sinuosity × valley length.	Determine allowable velocity or boundary sheer stress at design discharge.	Allowable velocity or shear stress criteria based on channel boundary materials.
Compute riffle spacing (if gravel bed), and add detail to design.	Empirical formulas, observation of similar streams, habitat criteria.	Determine meander geometry and channel alignment.	Lay out a piece of string scaled to channel length on a map (or equivalent procedure) such that meander arc lengths vary from 4 to 9 channel widths.	Compute mean channel slope and depth required to pass design discharge.	Uniform flow equation (e.g. Manning, Chezy), continuity equation, and design channel cross-sectional shape. Numerical water surface profile models may be used instead of uniform flow equation.
Check channel stability and reiterate as needed.	Check stability.	Compute riffle spacing (if gravel bed), and add detail to design.	Empirical formulas, observation of similar streams, habitat criteria.	Compute sinuosity and channel length.	Sinuosity = valley slope/ channel slope. Channel length = sinuosity × valley length.
		Check channel stability and reiterate as needed.	Check stability.	Compute sinuosity and channel length.	Lay out a piece of string scaled to channel length on a map (or equivalent procedure) such that meander arc lengths vary from 4 to 9 channel widths.
				Check channel stability and reiterate as needed.	Check stability.

[1] Assumes meandering planform would be stable. Sinuosity and arc-length are known.
[2] Computation of sediment transport without calibration against measured data may give highly unreliable results for a specific channel (USACE 1994, Kuhnle et al. 1989).
[3] The two methods listed assume a straight channel. Adjustments would be needed to allow for effects of bends.
[4] Mean flow width and depth at design discharge will give channel dimensions since design discharge is bankfull. In some situations channel may be increased to allow for freeboard. Regime and hydraulic geometry formulas should be examined to determine if they are mean width or top width.

Step 1. Investigate stable channel slope, stable meander bend widths, and/or stable anabranched (multi-thread) channels. One area of the initial investigation to be revisited is stream gradient. An essential investigation in establishing a new reach of channel should include field data collection of stream gradient in the project reach, and upstream and downstream of the site (WDFW, 2003). Stream gradient is a major factor in determining the river form, energy level and sediment transport of a river reach and in defining the attributes of the new channel. Channel cross-sections, bedload and bed material sizes, streambank stratigraphy, channel mapping of meander widths, wavelengths and radius of curvature and sinuosity are other necessary data for design (WDFW, 2003). Understanding the factors that define a stable and unconfined meander width is also needed to assess if there is sufficient space for a new channel corridor. Classification schemes are not advocated as a basis of the design, but may be useful in helping to identify ongoing channel processes: Schumm (1981), Harvey and Watson (1986), Rosgen (1994) and U.S. Army Corps of Engineers (1994).

Step 2. Investigate land acquisition and creative partnering. Another element of the investigation is to explore means of acquiring land or agreements to reestablish the channel and flood plain of a re-meandered channel. Creative partnering with current or potential stakeholders is an effective and feasible means of bank stabilization. The design will require enough land area for relocated or re-meandered channel to adjust in length and location to a new dynamic equilibrium.

Step 3. Define the hydrologic regime. Design of specific features of the floodplain and channel may be based on individual flow events but a single design flow is not sufficient for development of all the features. Instead, the designer considers the hydrologic regime for developing channel elements and floodplain. Estimate the high-flow events and frequencies that will occur over a period of 25, 50 or 100 years and base the design on the average assortment of flow events in a period. The hydrologic design may also be driven by life stage requirements of specific species.

Identify abnormally long flow durations for the drainage area. Flows with a longer duration can increase the effect of a peak flow event. The impact of long duration flows however, may also be affected by the sediment supply and transport, complicating this relation. Also identify specific climate and local factors that may impact the project hydrology. In addition to more common factors of flow diversion and flow management, note if rain-on-snow events, freeze up and breakup, sediment plugs, dam break or other infrequent occurrences could influence the hydrologic regime.

Frequently used design flows for the main channel and a floodplain (two-stage channel) are mean annual peak flow, 2-, 10-, and 25- year peak flow events. The design flow events depend upon existing or proposed channel terrace or floodplain elevation, and existing or proposed channel capacity. Also important to the design is the frequency when flows reach an elevation where they begin to flow overbank into the second stage channel.

Step 4. Identify processes in the project reach. Based on information from the geomorphic and sediment transport review, the form of the existing channel, instabilities of existing channel, and the energy level of the proposed channel, determine an appropriate channel planform (low-energy, meander, multi-channel/complex channel, anastomosed, braided) and develop a preliminary estimate of the section form (width-to-depth ratio, low-flow channel) to match the channel planform.

Channel slope is a main driver of channel form. Determine the elevation difference available for the stream gradient from the start to the end of the relocated channel. The basic design would include field data collection of stream gradient in project reach and upstream and downstream (WDFW, 2003). In low to mid-energy channels there is often a need to maximize slope in the design, and in medium to high-energy streams, the goal is often to minimize the channel slope to establish a stable channel. If the channel form is consistent throughout the project reach, the channel slope should also be set as a consistent gradient. Breaks in channel slope can produce instability in the channel.

Evaluation of the watershed and channel characteristics should not be limited to classification schemes; it is more important to investigate and develop an understanding of watershed and channel processes affecting channel dynamic equilibrium. Ask the questions: Are there dominant or unique flow processes? What are the mechanisms for erosion or scour and what are the dominant factors?

Sediment transport is a major concern of the channel design. A channel with consistent sediment continuity is stable and sustainable. Determine existing instability in the channel and its causes (Kondolf and Sale, 1985). Check for bars and the condition of islands. Are they building or eroding? What is the condition of the banks? Is the bed incising and what are the causes? Are channel widths narrowing, widening, or stable? Is vegetation coverage increasing, decreasing or changing locally? Is the species makeup of the vegetation coverage changing? Are there any changes in the number and location of invasive species and does their structure impact flow processes and channel form? Check the bed gradation and armor layer and determine if it is consistent with the reconstructed picture of ongoing channel processes.

Step 5. Develop a numerical model to aid development of the design. The model can be a 1D or 2D, and can be a flow model (SRH-1D, SRH-2D or HEC-RAS), a flow and sediment transport model, or a flow, sediment and vegetation model (SRH-1DV, SRH-2DV). The model can be used here to help develop preliminary sizing of the main channel cross section, and the floodplain cross section and for many successive steps including sizing the low flow channel, analyzing sediment transport modeling and channel sustainability, and to evaluate the floodplain grading plan and revegetation plan for erosion and sustainability.

Step 6. Design a stable cross section for the main channel. Bankfull flow and channel slope are used to size the main channel. Bankfull flow can be flow with a 2- to 5-year return interval, and sometimes more in the arid southwest. If there is a goal of preventing vegetation establishment in the channel, select a more frequent return interval of 2 years, but if vegetation in the channel is desirable to help stabilize banks, look at a higher value, perhaps a 4-year or less frequent return interval. If the goal is for a healthy stand of vegetation in the overbank area, more frequent overtopping is desirable. The channel will adjust its section form to the frequency of flows. If the bankfull flow is undersized for the hydrologic regime, the channel cross section area will increase over time with high-flow events. Slope may still be an estimate at this point, and will not be finalized until the alignment is defined. Steps 6, 7 and 8 can be an iterative process to finalize the slope, channel cross section, and alignment. The order of steps 6 and 7 (cross-section design), and step 8 (alignment layout), can be reversed depending on the data that is fixed or defined by the project site.

Bankfull flow is used to determine the cross section area, and the width-to-depth ratio of the main channel. This ratio is interdependent with river plan form. A braided river has wide and shallow channels that can transport large sediment loads, while a meander channel will be more narrow and deeper. Match the width-to-depth ratios of existing conditions with stable channels that have similar sediment loads (Figure 4–1). The final width of the channel is designed based on the river form, channel length and slope, incoming hydrology, and sediment supply and bed material size.

One method to determine the stable channel design is to use analytical equations for stable design for cross-section, width, depth and slope. However, flow resistance and sediment transport are indeterminate without further process based equations that relate cross-section to slope. The procedure is to use a width-hydraulic geometry equation to make the channel determined for a range of depths and slopes. Stable slope values could be picked for the reach. Copeland (1991) used a one-dimensional trapezoidal cross-section with steady uniform flow in conjunction with the process-based equations of Brownlie. The Brownlie equations account for grain and form roughness. Soar and Thorne (2001) also detail changes and enhancements to the method that can be used to determine a stable channel for channel relocation in a sand bed river.

<u>Constructing width-to-depth ratios</u>. There is a common tendency to design channels to a smaller than ideal width to depth ratios, to make than more constructible for large machinery. Ditches are common examples. Less precision is required since there is more elevation difference to work with. Coarse channel linings also complicate the construction process due to the irregularities of the material. Designers may measure to the center of a boulder but during

construction the easier measure is to the top elevation of the mix. Field inspect the channel depth during construction to help the contractor understand the product that is needed. Watch for a minimum width requirement when designing a low-flow channel within a main channel to allow for finding a small blade to cut out the channel. Using an overly small width-to-depth ratio for the main channel has a large negative impact: the overly narrow and deep channel can concentrate flow within the channel and disconnect the channel from the floodplain, instigating the channel incision processes.

Step 7. Design the low-flow channel within the main channel. Select a flow rate, possibly average daily low-flow, to define conveyance for the low-flow channel. Slope is also a factor in sizing the low-flow channel. The low-flow channel helps to convey sediment, contributing to channel stability, and increase stream flow depths, an ecosystem benefit, during the more frequent daily flows.

Step 8. Finalize the channel alignment. Confirm the correct river form and energy level of the channel, which is related to the available slope. Is the proposed river form compatible with upstream and downstream river form (low-energy, meander, complex or anastomosed, braided)? Are the upstream and downstream sediment transport conditions a match for the relocated reach? A mismatch in form and sediment continuity will produce channel instability following construction, and sediment erosion and/or deposition will continue until the channel evolves to a stable condition. The channel alignment also helps define the channel slope. Steps 6, 7, and 8 can be an iterative process to determine slope and design the channel cross section. In some cases, the channel alignment may be determined before the channel cross section.

In most cases, the new channel alignment should not result in a shorter, steeper path for river flows. Countermeasures such as constructed riffles are one means of preventing upstream channel bed lowering but they also remove energy from the system, which is undesirable if the system requires higher energy to remain braided, or in the case of meander rivers, maintain bend pools for fish habitat. A shorter, steeper path is especially undesirable in situations when the upstream channel is already incised and disconnected from the historical flood plain.

<u>Designing alignments</u>. Relocated channel alignment can be determined using the methods in table 11–1 and can be based on the alignment of typical meanders in the adjoining river reaches if natural undisturbed channel exists, or can be based on empirical findings from rivers with similar factors. Empirical guidelines for determining a stable channel alignment can be found in the Federal Interagency Stream Restoration Working Group (1998).

<u>Check compatibility of features with river form</u>. The alignment of meandering rivers is defined by meander bend radius and wavelength, and stable values are a function of factors including flow rate, slope and soils. Braided rivers tend to be straight and have more gradual bends. Low-energy rivers with limited sediment

transport can be placed into any configuration but sediment continuity is required at high flows when material is transported. Sustainable bend pools in meander bends require some energy in the system at higher flows. If the channel is devoid of energy at higher flows, the bed forms and channel will not be able to maintain pool-riffle systems and other desirable features for fisheries habitat. If it is appropriate to the project, side channels can be added for a complex channel form. Side channel design can be based on the geometry guidelines for the appropriate flow rate and soils.

Using geomorphic science, confirm that side channels and other features are compatible with the desired river form. As an example on the Platte River, a braided plan form is the historical form that provided habitat for current threatened and endangered species. The occurrence of braided river, along with the target species, is now declining and much of the river is a complex, anastomosed channel. River energy appears to be at a threshold in many remaining locations of braided river (Murphy et al., 2004). In an independent attempt to increase bird habitat, stake holders in the years 2005 to 2010 constructed side channels on remaining reaches of braided river that decreased flow energy in the main channel. The construction of side channels was in conflict with the main program effort to maintain remaining braided river plan form, and the associated desired habitat.

Abandoned meanders and oxbows. On rivers where meander or complex river form is desirable, abandoned meander bends or oxbows can be used for relocating channels. A detailed geomorphic analysis can help in the selection of suitable features. The sediment transport capacity should be in balance with sediment supply for relocated channels to be geomorphically stable (see section 8.2.4.2). A channel plug may be needed to block the previous channel to divert flow into the relocated channel. An objective is to only implement features that do not require structural aids or maintenance. Consider instead if terrain adjustments are needed at the relocated channel entrance to prevent recapture of the river in the old channel entrance during high flows. The inlet and outlet alignment should make for a smooth transition between the existing channel and the relocated channel. An overly sharp exit angle could induce downstream bank erosion.

Step 9. Check sediment continuity and design channel lining gradation. Inadequate sediment transport can cause excessive sediment deposition or incision, i.e. a channel instability that disturbs flow and sediment conveyance and can require maintenance actions. Sustainability can be checked by comparing the sediment transport capacity to sediment supply for a range of discharges above and below the design discharge (Shields, 1996) for the river reach. For larger projects use sediment transport models such as SRH-1D to evaluate bed stability. The new channel should transport the available sediment supply without channel degradation or aggradation. Sediment transport capacity for the new alignment, and resulting channel slope, should ensure transport capacity and sediment supply are in balance (McCullah and Gray, 2005). In certain circumstances, it can also be desirable for the re-aligned channel to have an increased sediment transport capacity and cause the upstream bed elevation to decrease.

Soar and Thorne (2001) have developed an analytical design procedure for gravel bed rivers. For gravel bed rivers it would be the simultaneous calculation of a flow resistance equation like Kuelegan, Limerinos, or Bathurst, and use of a gravel bed sediment transport equation like Meyer-Peter-Mueller or similar. Only use a d_{50} for the Meyer-Peter-Mueller equation. For sediment mixtures, use a Parker or Wilcock equation.

Sizing Channel Lining. Sizing requirements and gradations for bed material/channel lining is distinct from sizing riprap. Bed material should be transported downstream during higher flows, but is replenished by the upstream sediment supply. Select the recurrence event for bedload transport and using this flow and incipient motion equations, determine the D_{50} bed material size for the channel. This variable can be difficult to ascertain if the channel is bimodal (mixtures of sand and gravel).

If the bed material is over-sized like riprap, and cannot transport at high flows, the stream loses function since the cross section form and bed features cannot adjust. Conversely, some coarser armoring may be beneficial at river locations where there is not a sufficient sediment supply naturally supplied from upstream. Standard methods can be used to characterize bed material if a well-developed armor layer is present. In some instances it may be necessary to use a higher percentile than the median bed material size (D_{75}) (Shields, 1996). If more than 5 percent gravel- or cobble-sized materials are in the new bed of the pilot channel, these sizes will influence the new dynamic equilibrium width, bed slope, and substrate size. These sizes can lead to bed armoring.

Channel lining material should be rounded. Although angular material is preferred for riprap, angular channel lining rock introduces an irregularity in the river system material and will also be visually odd as non-natural stream bed material.

Filters. Geotextile should NOT be used as a filter underneath the channel lining. If a filter is needed, granular filters can be used. Granular filters may not function as well in the short term but are more adapting and compatible with future channel adjustments. Even stable creeks or rivers will shift, and banklines and bedforms adjust in response to the different flow events. These processes eventually expose edges of the geotextile, and flow currents can produce a flapping motion in the material that quickly rips out segments of the geotextile and the bank. This accelerated erosion causes bank erosion and leaves exposed geotextile, like trash, abandoned in the creek. As a rule, geotextile is undesirable in rivers and can only be used for the construction of fixed river infrastructure that will have routine inspection and maintenance (dams, flow diversions).

Step 10. Develop the floodplain grading plan. Check and if appropriate to the project, modify the floodplain to ensure that adequate conveyance is available.

Using a numerical model developed in step 3 and a project design-flow event check the function of the floodplain. At a peak flow event there should be no significant damage to infrastructure in the floodplain, or no significant erosion on the floodplain. Check depth of flow and shear forces on the floodplain, and compare to erosion resistance of anticipated vegetation on the floodplain (Table 9–2) to ensure the vegetation will be effective in preventing general erosion. In the case of braided rivers, the model can also be used to check that there is a sufficient level of erosion to remove vegetation and access the sediment supply in the banks and floodplain to maintain a braided condition.

Revise the grading plan if the erosive forces on the flood plain are too high for the vegetation coverage. Consider temporal impacts and how to prevent the erosion of newly seeded or planted areas of the floodplain, or allow for a second seeding or planting period in the wake of a high-flow event or drought in the 2-3 years of vegetation establishment.

<u>Grading the Overbank Area</u>. Flat floodplains appear odd although they are occasionally seen in nature in very large lowland drainages. Waste material from construction can be used on-site to add additional features and contours to the floodplain when there is a sufficiently wide floodplain. Placing material is not for aesthetics alone. Material contoured to resemble natural terraces and located at sharp bends in the river can help slow migration of meander bends. Raised contours in meanders can discourage the development of cross channel avulsions. Erosion of the contoured material can be a sediment source and contour material should be compatible with sediment transport of the stream. For example, coarse materials can have negative impacts to fine-grained sand bed rivers. Excess material eroding into a high energy meander bend is a means of augmenting sediment during high flows.

Avoid over-restricting the floodplain width. Limit placement of material to avoid increasing floodplain erosive forces beyond the shear force resistance of the vegetation, with the exception that steeper slopes can be used adjacent to the river if sediment augmentation is desired. If the drainage is also a water delivery system, steeper slopes can be used to promote quicker drainage returns after overbank flooding. Floodplain grading can be used to recharge groundwater, store to pools for habitat, or delay return flow to reduce flow peaks. Check erosive force on the floodplain by using a numerical flow model with modified cross sections or terrain to represent proposed grading contours. Accuracy of grading can be defined in the specifications to provide the contractor with some leeway for onsite conditions.

<u>Habitat Enhancement</u>. Flatter contour slopes can be used to promote seeding or planting of the area (steeper slopes require seed drilling) following construction, or contours can be influenced by a vegetation plan. A vegetation plan may require specific percentages of vegetation coverage divided between wetlands, riparian zones or more dry upland areas. Depending on environmental needs, avoid pooled

areas in the floodplain to prevent stranding of fish during the declining limb of a hydrograph, or add pooled areas adjacent to the channel to promote the establishment and survival of new cottonwood plants. Low lying areas reduce the distance seedling roots have to grow to reach the water table and can reduce irrigation requirements for plant establishment in addition to supporting more mature vegetation through dry periods. Contoured ground can also be designed to introduce habitat complexity into the overbank area. Varying elevations and features in the floodplain will create more niches and more diversity in the riparian plant species on the banks and floodplain.

Low lying areas can also be attractive to salt cedar (*Tamarix sp*.) plants in warmer climates, and the disturbed area in general can promote the spread of invasive vegetation. Include an invasive plant control program for several years following construction.

Step 11. Develop recovery plans for the period following construction. This may include planting and seeding plans, a monitoring plan, and flow release requirements for the next 3 years during plant establishment. Planting vegetation along a newly excavated channel alignment is frequently part of the design (McCullah and Gray, 2005). All designs should include vegetation growth on the banks, bank stabilization, erosion resistance of bed or bank material, minimal change in channel length, preservations of original vegetation, and having both low flow and high flow channels (Brice, 1981). Determine if planting plans require irrigation and develop an irrigation plan. When designing the planting plan, check the schedule and determine when the nursery must begin growing plants so the plants will be ready to go in the ground following construction.

Step 12. Check constructability. Review the plan and designs for construction issues. Check for accessibility and roads that can support heavy machinery required for the project. Consider nearby neighborhoods and the level of tolerance for truck traffic, noise and dust. What are sediment permitting requirements? Is there tolerance for higher sediment loads as the channel adjusts form and temporarily increases sediment transport? Consider potential sources of channel lining material and transport distances.

11.2 Pilot Channels

Pilot channels are a low-cost approach to channel reconstruction. Pilot channels are excavated channels along a desired alignment. This method can be a good option when there is no time restriction on the period allowed for a stable channel to develop, and when there are no concerns about excess sediment being transported downstream for multiple years.

Pilot channels are excavated to a narrower width than the current main channel to reduce construction costs and reduce the size of sediment disposal requirements. By constructing a narrower channel than exists in the reach, higher flow events will erode the channel banks to a stable cross section form. Excavated sediment can be added to banks as a sediment source, in an incising reach of river. Sediment piles may need to be repositioned over time to be eroded fully and removed by high flows. At suitable sites, vegetation may also be cleared to aid the process of channel widening. Bank lowering can also aid establishment of the new channel width. Bank lowering could include creating a compound channel section and widening the channel.

Any changes in channel length or slope should be evaluated to ensure capacity, sediment continuity, and desired river pattern is maintained. Flattening the channel gradient can affect planform and fisheries habitat. Channel relocation using pilot channels or pilot cuts has been documented in the literature, mainly for channel shortening projects. Historically, thousands of channels have been straightened to accommodate roadways, bridges, and other transportation facilities. These types of projects have been shown to have an adverse effect upon the river channel, flood plain, and riparian zone (Brookes, 1988; Parker and Andrus, 1976; Piest et al., 1977; and others). When a channel is shortened, slope, channel velocity, and sediment transport increase. This can lead to upstream channel incision, narrowing, and reduced flood plain connectivity. Downstream, the effects can be aggradation, channel widening, and loss of flood capacity.

One of the primary goals of the channel narrowing and limited straightening on the Middle Rio Grande was to promote increased sediment transport, reduce channel aggradation, and increase flood carrying capacity. This method has been used with some success on the Middle Rio Grande at Santa Ana Pueblo and the Bernalillo site.

In some cases it may be wise to excavate a new channel alignment, but allow several years for vegetation to develop before diverting flows into the new channel. The disadvantage to this approach is that riparian vegetation cannot establish without flows so irrigation may be required if there is not sufficient rainfall during this period of vegetation establishment. The same may be true for the overbank area; vegetation establishment could be dependent on spring overbank flows, or irrigation may be needed to help with re-vegetation in the immediate overbank area. The initial period of vegetation establishment is approximately 2 to 3 years and is dependent on climate and flow conditions with wetter periods commonly increasing establishment and growth rates of riparian plants.

11.3 Discussion and Recommendations

Risk and Failure. The intent of a channel design is to construct a stable system. If the channel is designed as a stable system, there should be minimal to no maintenance required and the transport capacity of the constructed reach should transition from the sediment transport of the upstream reach to transport in the downstream reach. If it is not possible to design a stable channel for the reach, some maintenance may still be required. One example of stabilities that cannot be addressed through the project design is sediment plugs that occur on the Rio Grande as a result of general system aggradation. Sediment plugs cannot be eliminated within the extent of short channel relocations and some maintenance may still be required in the form of sediment plug removal.

Sediment deposition in an abandoned channel can reduce the effective lifespan of remnant channel habitat. A second mode of failure can be when the channel plug or dike constructed at the junction of the existing channel and the relocated channel is washed away, leading to flow being recaptured by the previous channel (McDullah and Gray, 2005).

Monitoring and Maintenance. Relocated channels can be monitored visually in the field and/or by using cross-section and thalweg surveys to ensure that the relocated channel is stable and is not threatened by erosion or sediment deposition. Maintenance requirements include possible repositioning of the sediment disposal piles if needed. In addition, selective sediment removal may be required as part of the river adjustments to a lengthened channel to ensure that connectivity is maintained to the relocated features and abandoned oxbows.

Depending upon the extent of channel response, monitoring may need to extend upstream or downstream. Additional monitoring may include physical aquatic habitat, fish presence or absence, and macroinvertebrates.

12 Transverse or Indirect Methods

Transverse features are bank protection structures that extend into the stream channel and redirect flow so that the eroding velocity and shear stresses are reduced along the bankline. The transverse features included in this guide are bendway weirs, vanes or barbs, spur dikes, and, J-hooks. The methods are also called indirect because they do not provide increased bank erosion resistance directly on the bank. They are also called re-directive methods because they re-direct flow patterns along the bank.

Bendway weirs and vanes (or barbs) are similar but have a different crest slope and effect on flow patterns. Bendway weirs have a flat top positioned above the low water surface and below the bank-full water surface elevation; thus, there is a weir effect at low flows. The flow that is captured by the weir is redirected to the center of the channel (Figure 12–1). At high flows, bendway weirs redirect the secondary currents, which helps reduce the near bank high-flow velocity. Bank line currents above the weirs are not re-directed during peak flow events. Vanes (or barbs) have a crest that slopes upward from near riverbed to generally the top of bank or to the mean annual peak flow water surface elevation. The tip is inundated for most flows. This sloping top redirects the flow away from the bank in the near-bank region. Flows are redirected up to the elevation of the vane crest at the bankline at high flows. J-hooks are vanes with a downstream pointing “J” configuration of partially embedded large stones. Near the bank the J-hook has the same effects on flow as vanes. The partially embedded “J” configuration causes a scour hole to form for habitat. Spur dikes have horizontal crests at the elevation of the top of the bank or the mean annual peak flow water surface elevation, and deflect flow away from the bank, reducing near-bank velocity to prevent bank erosion.

There are potentially several advantages to using transverse features for bank protection. The construction benefits include that bank preparation is typically much less than other methods. This reduces costs and reduces impacts to the riparian environments and usually allows overbank drainage and inundation levels to remain unchanged. In addition, these structures generally use less riprap material to protect the same amount of bankline as compared to the direct method of riprap lining (revetment). Existing channel alignment and/or geometry can be modified if needed to establish a channel which is re-directed away from infrastructure. Transverse methods usually increase geotechnical bank stability by inducing sediment deposition near the bank, although this process may take a number of years to occur. Sediment deposition between transverse structures can increase bank stability and provide surfaces for vegetation growth. In some cases though, this deposition may adversely affect aquatic habitat. In these cases, the deposition may be mitigated by constructing notches (lowered areas in the transverse feature near the bank) (Shields, 1983a). Transverse features create variable depth and velocity and bank edge habitat important for many aquatic species.

Transverse features deflect flow away from the bank, thereby altering the secondary currents and flow fields in the bendway. Therefore, they have different local impacts than do bank armoring techniques. The amount of flow that is deflected away from the bank varies with each method and local site conditions. It generally is assumed that the length of effects is “about one-half meander wavelength up and downstream for meandering streams and about four channel widths downstream and one to two widths upstream for a braided stream” (Fischenich, 2001).

These features are intended to move the thalweg away from the bankline (Johnson et al., 2001). With the thalweg being moved out away from the bankline and interruption of the secondary currents, the maximum velocity location is more towards the center of the channel. This could increase the velocity in the center of the crossing or riffle and result in mining bed material from the crossing or riffle. (This is a conceptual hypothesis without supporting data.)

Transverse features have the potential to raise the water surface because of the reduced cross sectional area and increased form roughness. Generally, the rise in water surface elevation is small but should be evaluated. Transverse features can be represented as blocked obstructions in HEC-RAS (Sclafani, 2010). Encroaching upon the channel width with transverse features often results in deepening of the main channel and local scour. These methods induce additional local scour at the toe of the structure and downstream for a distance of “about two to three times the scour depth” (Fischenich, 2000). Transverse features generally decrease the cross sectional area often resulting in the cross sectional mean velocity increasing. In addition to disrupting secondary currents, these methods generate eddies, increase turbulence, and create velocity shear zones.

These structures can also significantly alter the scour and deposition patterns. The scour and depositional patterns depend on many factors including orientation, planform, channel type, bed material size and sediment load in the river. The scour and deposition, and zones of high and low velocity in close proximity, create variable depth and velocity habitat. This creates greater environmental benefits than riprap revetments or longitudinal stone toe (Shields et al., 1995) and greater edge or shoreline length is also created (McCullah and Gray, 2005).

Bank scalloping between transverse features is a common occurrence and can lead to failure which can be overcome by constructing sufficiently long bankline keys. Some bank retreat between structures is likely to occur before a stable bank is achieved by vegetation growth and/or sediment deposition. When bendway weirs, vanes and J-hooks are used, long term bank stability is best achieved if significant sediment deposition occurs and subsequent vegetation growth forms a dense riparian zone (McCullah and Gray, 2005). Long term bank stability is also best achieved when there is low potential for upstream channel migration. The upstream channel migration potential should be evaluated to determine the liklehood of the angle of the approaching flow changing over time. The angle of approach for multiple stages of flow should be evaluated and included in the evaluation of transverse feature structure geometry and placement locations along the bend.

Like all bank stabilization projects, fixing the bank location with transverse features can diminish lateral and down valley migration and have the same upstream and downstream morphological effect as bankline features described in Section 4. In channel conditions where the sediment supply from bank erosion is a significant part of the total sediment load, there could be increased or accelerated bed and bank erosion downstream (Fischenich, 2001). Reduced down valley migration can reduce the potential for establishment of multiple age classes of vegetation.

12.1 Bendway Weirs

Bendway weirs are features that extend from the bank out into the flow with horizontal crests that are submerged at high flows and angled upstream (Figure 12–1). Bendway weirs are designed to control and redirect currents though a bend and immediately downstream of the bend. Their purpose is thalweg management (to move, realign, or relocate the river thalweg through the weir field and downstream). They reduce near-bank velocity by redirecting the current and adding form roughness along the bank. Some bank scalloping may occur between weirs; it is difficult to predict. Field observations have shown that taller bendway weir crest elevations appear to be associated with increased bank scalloping.

Specifically, bendway weirs are intended to (1) deflect high velocity near-bed flow away from the outer bank, (2) inhibit helical secondary current motion in the bend, and (3) redistribute momentum near the outer bank (Derrick, 1997). They also increase flow resistance near the bed and base of the outer bank and redirect high-flow currents towards the center of the channel and away from the bank, thereby reducing or eliminating bank-line erosion (Derrick, 1997b). However, bendway weirs have also been shown to cause increased high boundary shear stress over the weir crest as flow converges, and on the bed in the lee of the weirs as flow expands downstream of the weir (Abad et al., 2008; Scurlock et al., 2014).

Bendway weirs differ from spurs and vanes (barbs) in that they capture the flow field and redirect flows away from the bank (Derrick, 1997b). This is accomplished throughout the bendway usually with a minimum of five structures in the weir field. Bendway weirs are intended to function when overtopped; vanes however deflect flow and are meant to be overtopped near the tips but not near the bankline. Spur dikes are designed to be above the water surface during the design high flow event.

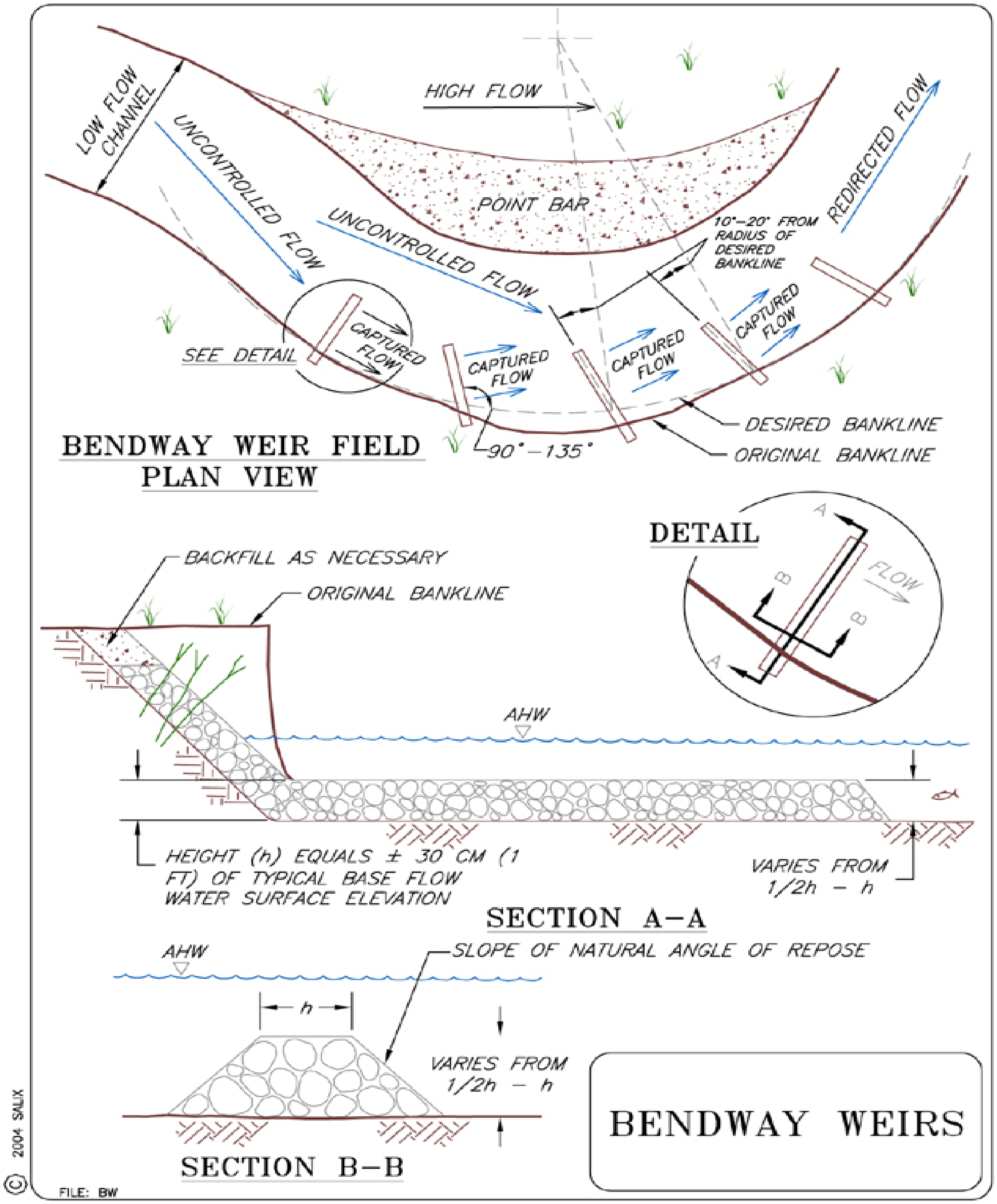


Figure 12–1. Bendway weirs. Used by Permission of the Transportation Research Board (Report No. 544) and Salix Applied Earthcare (NCHRP, 2005).

Bendway weirs were originally developed by the USACE to improve the navigation channel and increase channel depths to reduce dredging costs in the Mississippi River (Biedenharn et al., 1997). Since the original development, bendway weirs have been applied to small streams as a streambank protection measure. Bendway weirs can create variable depth and velocity habitat, can lead to sediment deposition along the bankline, and can be constructed for less cost than riprap revetments or a longitudinal stone toe with bio-engineering.

Weirs can be installed after other methods of bank protection are installed (such as longitudinal features) to improve their performance and habitat value by creating more variable velocity and depth conditions. In addition, woody debris can accumulate on weirs, adding fish cover (McCullah and Gray 2005). There is also a tendency for scallops to develop between structures, which can create nursery habitat.

12.1.1 Examples of Application

The number and range of bendway weir application is very large, from small streams where the main purpose is bank protection, up to large rivers, such as the Mississippi River, where the main purpose was creating a wider and deeper shipping channel (Biedenharn et al., 1997). Lagasse et al. (2009) report that bendway weirs are used for bank protection at highway bridge crossings on braided or meandering rivers with small to medium radius bends and for channels up to 150 ft wide in the States of Colorado, Idaho, Illinois, Missouri, Montana, Oregon, and Washington. They also note that there is “limited but successful field experience using bendway weirs/stream barbs as stream instability countermeasures.”

12.1.2 Design Procedure

Key design variables include crest angle, crest elevation, spacing and length. Design also includes plan layout when structures are used in series along eroding banklines, riprap material sizing, evaluating the need for filter placement, key length and scour protection. The following design procedure reflects guidance provided by NRCS (2007), LaGrone (1996), Saele (1994), Derrick (1994, 1996), and Lagasse, et al., 2009), with some modifications. In some instances McCullah and Gray (2005) have a different approach which will be noted.

No complete systematic quantitative design method currently exists for crest length, angle, width, and spacing that accounts for various channel conditions (Abad, et al., 2008; Lyn and Cunningham, 2010). Current guidelines do not address approach velocity, which can influence the effectiveness of bank erosion control with bendway weirs (Lyn and Cunningham, 2010).

Because of this and because the weirs act primarily as flow deflection structures and not bank reinforcement structures, the existing bend condition, geometry, planform, stages, discharges, and sediment transport capacity must be carefully evaluated (McCullah and Gray, 2005). The direction and velocity of flow entering the bend proposed for bendway weirs must be measured and evaluated (McCullah and Gray, 2005), including for both low flow conditions and the design flow.

The below design procedure for bendway weirs presents recommended steps after a geomorphic analysis has been completed and has determined that bank protection using transverse features is an appropriate approach.

Step 1: Determine design flows and hydraulic conditions

The hydraulic conditions during the mean annual flow, the annual low flow, the mean annual high flow level, and the flow corresponding to the green line should be computed. The hydraulic numerical model should be developed using methods described in Section 4.4 to determine the flow hydraulics. Bendway weir design guidance is based upon the flows listed in this section.

Step 2: Determine desired Bank Line and Thalweg Location

First sketch the desired thalweg location (flow alignment) with a curve and with upstream and downstream transitions. Consider that the thalweg may shift locations depending on the flow magnitude and bed material size. In channels with gravel or cobble sized bed material the high and low flow thalweg location may be the same. In sand bed channels that are highly mobile and have an erodible point bar, the thalweg will tend to move based on the alignment of flow lines at high or low discharges. The desired flow alignment may be based on preventing further erosion of the outside bank, or reverse erosion of the bank to alter the flow alignment. The curve representing the desired flow alignment should form a smooth upstream and downstream transition. After bendway weir installations, the thalweg is typically shifted near the alignment of the weir tip. Therefore, the stream ends of the bendway weirs should be at or just short of the desired thalweg location.

Next draw an arc that represents the desired bankline location if it is different from the current eroding bankline. The desired bankline could be the existing concave bank or a new bankline that reverses past erosion. Note that the flow alignment and bankline locations may need to be drawn several times after both are first sketched to provide for smooth transitions and proper thalweg location.

Bendway weirs should be constructed throughout the bendway because the success of the weirs depends on the flow conditions at the upstream end of the bend. The length of bank protection can extend between the upstream and downstream bend inflection points. The core of maximum velocity follows the thalweg at low flows but it can shorten its path by cutting across the point bar at high flows. At a bend, the region of maximum scour and bank erosion is observed to be located in the downstream part of the bend. Therefore, the protection may not need to extend to the upstream inflection point between bends but may need to be extended beyond the downstream inflection point. Transverse features should be constructed sufficiently far downstream to direct flows toward the center of the downstream channel.

Step 3. Determine Weir Length

A range of weir lengths are recommended. Weir lengths ranging from one-third to one-half of the active channel width, with some lengths being 20 percent of the active channel width is recommended by McCullah and Gray (2005). Lagasse et al. (2009) recommends weir length between one-tenth and one-third of the active channel width. Lagasse et al. (2009) observed that weirs longer than one-third of the stream width can alter channel patterns and, possibly, cause erosion on the opposite bank. Weir length can also be based on how far from the eroding bank the thalweg should be moved and how erodible the point bar is (Derrick 1997a).

As an initial estimate of weir length, it is suggested that the weir length equal one-fourth the bankfull top width. The top width is the approach-channel top width from the one-dimensional model results usually using the mean annual peak flow. The width for the Q_2 and the width between the riparian vegetation may also be useful. One should confirm that this weir length crosses the thalweg throughout the bend in order to move the thalweg toward the channel center and away from the outer bank. Note that the desired bankline and thalweg locations determined using the procedure in Step 2 may need to be altered to meet these criteria. The selected weir length should result from a synthesis of the desired thalweg and bankline location with the recommendations based on channel width. In addition, both the length and angle may vary through the bend of the river to better capture, control and direct the flow.

Step 4: Determine weir crest elevation

Using the hydraulic model results, the preliminary elevation of each weir would be the water-surface elevation minus 0.5 to 0.7 of the mean flow depth using the mean annual high flow. An alternative preliminary elevation is shown in Figure 12–1. Check this preliminary elevation with the criteria that weir elevation should be between the annual mean flow and the annual low flow water-surface elevations. If the preliminary elevation weir height is above the annual mean flow then it should be adjusted to be between the annual mean flow and annual low flow water-surface. Likewise if the alternative preliminary elevation is below the mean annual low flow water surface elevation it should be adjusted also to be between the annual mean and annual low flow water surface elevation. Bendway weirs constructed taller than these guidelines are not frequently overtopped leading to increased bank scalloping potential. The strongest downstream eddies form when the flow does not over top the weirs at higher discharges. A final consideration is the McCullah and Gray (2005) recommendation that the weir crest elevation should be within one foot of the typical base flow elevation. Base flow is defined as the average of the mean daily flows, excluding typical high flow seasonal periods.

Step 5. Calculate weir spacing

A key variable in determining the weir spacing is the bend centerline radius (R_c). This is the radius of a circular arc which best approximates bend curvature between the upstream and downstream bend crossing or inflection points. The centerline is located using one-half the top width (as determined in Step 4) of the approach channel along the bend axis.

Calculate spacing based upon these equations Lagasse et al. (2009):

$$S = 1.5L\left(\frac{R_c}{W}\right)^{0.8}\left(\frac{L}{W}\right)^{0.3} \tag{12.1}$$

$$S = (4\ to\ 5)L \tag{12.2}$$

$$S_{max} = R_c\left(1-\left(1-\frac{L}{R_c}\right)^2\right)^{0.5} \tag{12.3}$$

Where S = spacing
L = bendway weir axis length
W = Channel top width
S_{max} = maximum spacing

Spacing should fall between the results of equations 12.1 and 12.2, depending upon flow alignment and bend geometry. The spacing should not exceed the value of equation 12.3. Results from these equations should be further investigated to determine if the weir spacing, length, and angle would redirect the flow based upon the criteria that the angle φ, between the approaching streamline and the streamline perpendicular to the weir, be no more than 30° at high flows and not less than 15° at low flows. If the approaching stream line is essentially parallel with the flow line perpendicular to the weir crest, then the weir will be ineffective and will direct flow into the bankline, possibly causing local scalloping. The weir angle and spacing should be such that the line perpendicular from the midpoint of an upstream weir points to the midpoint of the following downstream weir. Streamlines entering the bend, and exiting the weirs should be analyzed and drawn in planform.

Step 6: Determine weir positions through out the bend

Determine the position of the first weir from the aerial photographs or drawings of the bend. This is accomplished by drawing a line parallel to the upstream high flow approach channel and extending to the bankline in the bend.. This would be the position of the second weir. A shorter weir should be placed upstream of the second weir at the same spacing as weirs throughout the bend. Position each weir on the aerial photograph or drawing of the bend using either the chord length between weirs or the arc length. Using the arc length would be slightly more conservative.

Step 7: Determine weir orientation angle

Each positioned weir should be drawn on an aerial photograph or drawing at the location and length determined from the above steps. Draw the projection line through the center and parallel with the approach channel bankline.
The angle between the projection line through the center and parallel with the approach channel bankline and the weir should be between 15° and 30°. Weirs should be oriented upstream with an upstream orientation angle between 60^o and 80^o away from the bankline tangent . Smaller angles provide a larger amount of flow re-direction. For bank protection applications 60^o is recommended. The weir orientation angle criteria takes precedence over the projection line through the approach channel and weir angle.

Step 8: Determine weir key length

The key length (LK) is the length of the weir embedded in the existing bankline (Figure 12–2). Bendway weirs should be keyed (rooted) into the bank to prevent flanking. The design criteria for the key length is as follows (Lagasse et al., 2009). When $R_c > 5W$ and $S > L/\tan(20°)$ use:

$$LK = S \cdot \tan(20°) - L \tag{12.4}$$

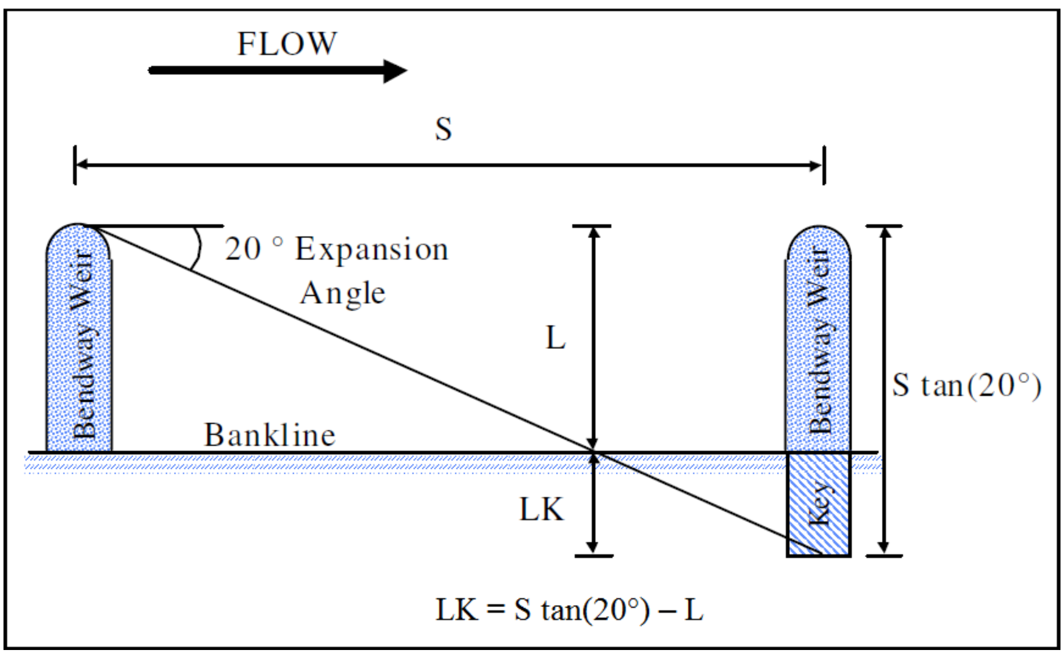


Figure 12–2. Bendway weir key length (after Lagasse et al. 2009).

When the channel radius of curvature is small $R < 5W$ and $S < L/\tan(20°)$ use Lagasse et al. (2009):

$$LK = \frac{L}{2}\left(\frac{W}{R_c}\right)^{0.3}\left(\frac{S}{R_c}\right)^{0.5} \tag{12.5}$$

The NRCS (2005) guideline for length of key (LK) for short weirs or barbs (NRCS, 2007) is to key the barb into the bank a minimum distance of 8 ft (2.4 m) or 1.5 times the bank height, whichever is greater. NRCS (2007) suggests a minimum root length for small streams of 8 ft or 4(D_{100}) whichever is greater.

If the eroding bend is close to valuable infrastructure, the maximum protection would be provided by having a longer key. Depending on the site location and project goals, the key may be extended into the bank at the same elevation as the weir crest, or sloped up along the bank to the design flow elevation or the top of bank at a 1.5H:1V maximum slope. Using the weir crest height for the key elevation maintains the continuity of the bendway weir structure if bankline erosion continues. Raising the key elevation at the bank reduces the excavation volume and also provides a countermeasure against locally increased bankline velocities at the weir crest.

Step 9: <u>Determine rock size</u>

By re-directing flow transverse features create zones of higher velocity and shear stress, especially at their tips, where riprap is often eroded. Often larger material is necessary to stabilize bendway weirs than riprapped banks at the same site.

First, determine riprap size using equations for turbulent flow for bankline riprap revetments. Size the well graded riprap according to bankline sizing methods in Section 13.2. Riprap size is usually based on between a 10 and 25 year return flow period peak flow. The NRCS (2007) method for stream barbs is recommended for bendway weirs. Their method uses longitudinal bankline methods (Chapter 13) for sizing riprap modified with the following:

- D_{50}, bendway weir = $2 \times D_{50}$, as determined for streambank riprap
- D_{100}, bendway weir = $2 \times D_{50}$, bendway weir
- $D_{minimum} = 0.75 \times D_{50}$, as determined for streambank riprap

The stone material specifications from Lagasse et al. (2009) are as follows:

- Stone should be angular, and not more than 30 percent of the stone should have a length exceeding 2.5 times its thickness.
- No stone should be longer than 3.5 times its thickness.
- Stone should be well graded, but only a limited amount of material should be less than half the median stone size. Since the stone will most often

be placed in moving water, the smaller stone will be subject to displacement by the flow during installation.

- Construction material should be quarry run or blasted/fractured stone. High-quality material is recommended for long-term performance.

As an alternative to well-graded riprap, bendway weirs can also be constructed from large stone elements when the site is dewatered, constructed using equipment in the river, and large stone is available.

Step 10: Determine bendway weir top width

The top width should not be smaller than 2 to 3 times D_{100} and typically is between 3 and 12 ft Weirs that are too long to be constructed by bank-based equipment will need to be wide enough (e.g., 12 to 15 ft) for equipment to safely drive out onto them. The weir top width may be determined by the volume of riprap material available for launching into the tip and downstream scour hole to provide for weir stability.

Step 11: Estimate scour and launch riprap volume

By this stage of the design a scour evaluation will have already occurred (Section 5). By re-directing flow around the tip and over the top of bendway weirs flow acceleration occurs which creates local scour. Important local scour estimation variables include design flow discharge per unit width, sediment size and structure type. Scour equations based upon bendway weir length, height, horizontal angle and bend geometry are not available. Laboratory and some field testing has been done for bridge abutment and pier scour (Melville and Coleman, 2000), and for spur dikes on sand bedded channels (Garde et al., 1961; Gill, 1972). Comprehensive evaluation of scour for bendway weirs with overtopping flow are not available. Independent analysis of the limited scour data of Cox (2005) showed that the Blench (1969) methodology calculated measured scour more accurately than nine other equations for transverse structures with overtopping flow. The depth of scour (Blench, 1969) is estimated using:

$$d_{fo} = \frac{q_f^{2/3}}{F_{bo}^{1/3}} \tag{12.6}$$

Where:

d_{fo} = Depth for zero bed sediment transport (ft)
q_f = Design flood discharge per unit width ft^3/s per ft
F_{bo} = Blench's "zero bed factor" in ft/s^2

Based upon bed sediment median diameter, F_{bo} is estimated using Figure 12–3.

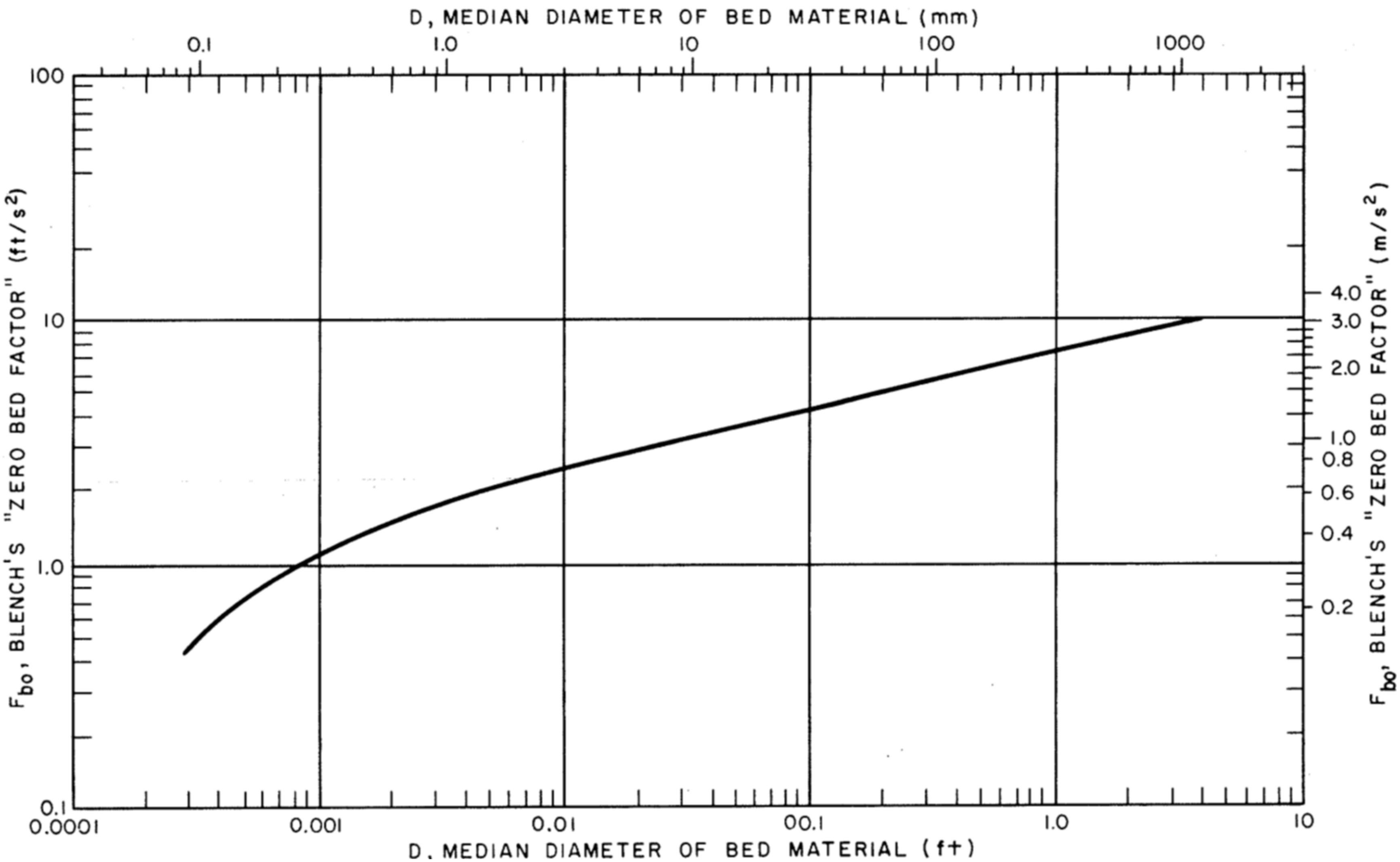


Figure 12–3. Graph to Estimate Blench Zero Bed Factor (Pemberton and Lara, 1984).

This equation is adjusted by empirical multiplying factors Z. The depth of scour below the streambed is:

$$d_s = Z\ d_{fo} \tag{12.7}$$

Using the limited laboratory scour data from Cox (2005) a value of $Z \sim 0.5$ was found to best match measurements. This Z value is within the range reported by Pemberton and Lara (1984) for the nose of piers. As a factor of safety, for high value infrastructure it is suggested that d_s be multiplied by 2.

The launchable riprap volume should be estimated using a 1V:1.5H slope to the scour depth with a thickness of at least D_{100} plus 30 percent for rock dislodgment during launching. The volume is determined based upon structure and cross section geometry. For bendway weirs constructed in flowing water the volume should be increased by 20 percent or more to account for rock erosion during placement.

Step 12: Check Constructability

In some rivers turbidity can be a significant problem for fish communities. Turbidity can be minimized by using tracked excavators, working during low flow periods, dewatering, stream diversion, or isolation. Working from the top of the bank, excavating the key, filling it with stone, and then constructing the transverse feature will also reduce turbidity.

When the weir is too long to be constructed from the bank or cannot be constructed in the river channel, the crest width should be wide enough and a flat enough slope for equipment to drive on the crest. An end dump and dozer can be used to place riprap sequentially out into the river until the design length is achieved. Then the transverse feature can be reshaped to the design elevation and crest slope with a hydraulic excavator from the tip to the bank until the final configuration is achieved. The key would need to have a temporarily flatter slope for driving than the final design configuration. Riprap can be placed at the drivable slope then removed after the design length has been constructed.

If the bendway weirs are too narrow, equipment in the stream will likely be necessary. In such cases stream diversion or isolation techniques could be used to prevent turbidity impacts. Access and staging areas should be carefully planned to protect the existing riparian zone.

12.1.3 Discussion and Recommendations

12.1.3.1 Risk and Failure

Bendway weirs are not recommended in narrow streams (less than 50 ft wide). They have not been used in high-velocity and high-gradient streams; however, the exact definition of high velocity and high gradient relative to channel morphology has not been defined.

The re-directive effects of bendway weirs on the flow field may be limited in cobble and gravel bed streams due to the erosion resistance of the bed material (McCullah and Gray, 2005). Weir stones would tend to launch into the downstream scour hole much more readily in a sand bed channel than in cobble or gravel bed streams. Bendway weirs are susceptible to flanking from upstream bend migration which changes the upstream approach angle.

Bendway weirs can be undermined in sand and gravel bed channels when the substrate upon which the rock is founded upon is mobilized and the rock falls into the hole that was created. This situation can be addressed by placing a gravel filter underneath the stone weir and placing self-launching riprap material to stabilize the weir toe as scour holes form. However, if there is reach scale degradation, bendway weirs structures may not function well (Lagasse et al., 2009).

Derrick (1997b) has documented weir conditions that can lead to additional bank erosion problems. If the weirs are angled incorrectly and are too long and high near the downstream end of the bend, there can be a large amount of flow over the point bar that could impinge against the downstream bank. Since the success of the weir field depends on entrance angle, the entrance conditions in relationship to the stream reach must be stable. If the flow direction changes due to upstream channel migration, the flow patterns may not be suitable to reduce the near-bank velocity and shear stress, and the bendway weir and bank will erode away (Chester Watson, 2007, personal communication, Biedenharn Group, LLC, Vicksburg, MS).

Bendway weirs have been shown to reduce bank erosion due to toe scour. However, the velocity and boundary shear stress increases locally over the top of the weirs, which can promote bank retreat, potentially creating a “shelf” along the outer bank as shown in figure 12–4 (Abad et al., 2008). Bank erosion above weir crests was also observed in a laboratory study of bendway weirs (Lyn and Cunningham, 2010). Bio-engineering could be used as a counter measure to prevent the formation of a bank shelf above the bendway weirs. However, additional studies and research are needed to quantify the effects of bendway weirs on upper bank erosion control.

Bank scalloping is a risk. There is no methodology to estimate the amount of scalloping. Longer keys are advisable near valuable infrastructure where bendway weirs are deemed appropriate. When bendway weirs are constructed near valuable riverside infrastructure, placement of small riprap or native river gravel or cobble size material is recommended along the toe up to the elevation of the weir height. There must be sufficient space between the bank structures that need to be protected and the weirs so that, as bank scalloping occurs, the bank structures will remain protected (Derrick, 2002). As is the case with other hard features placed within the stream channel, the weirs can create scour pools that could be used by native or exotic fish species.

Figure 12–4. Erosion of bank face at a meander bend with bendway weirs, Big Creek, Clark County, Ill. Retreat of bank has produced a platform at the elevation of the tops of the weirs and intervening stone toe protection. Rocks in the foreground represent exposed part of the weir that was originally embedded in the bank to key the weir into the bank. A weir and its exposed key can be seen in the background. Photo used by permission from American Society of Civil Engineers. After Abad et al. (2008).

12.1.3.2 Monitoring and Maintenance

Due to the hydrodynamic forces on the weir tips and scour generated by weirs, riprap erosion is common. All transverse structures should be routinely inspected at least once a year and after all high flow events with return periods greater than 2 years. The inspection/maintenance staff should determine:

- Is the structure intact?
- Are flows being redirected where expected?
- Is there any unintended scour?
- Is there deposition on the upstream side of the structures?

- Has the structure series created or exacerbated erosion or lateral instability downstream of the structure?
- Is the outer bank between weirs stable and becoming vegetated. (recognizing that some scalloping or a shelf may have developed but is stable and vegetating)?
- Is sediment being deposited on the outer bank after high flow events?
- Is there bank material at the toe of the previously eroding bank?
- Has the point bar scoured? This would typically leave a near vertical inner bank slope usually about mid-bend to the downstream end of the bend.

Common transverse feature maintenance includes:

1. Adding riprap to the weir tip and crest.
2. Increasing the length of the key or root.
3. Adding additional bank protection if scalloping or formation of a shelf (bendway weirs) threatens the integrity of either the transverse features or riverside infrastructure. Additional bank protection could be achieved through bio-engineering, reducing the slope in scalloped areas between structures, or adding armoring such as small riprap or native river cobble or gravel material. Using native cobble or gravel with colors and shapes consistent with the local environment makes the resulting structure more visually esthetic.

12.2 Vanes or Barbs

Vanes, also known as barbs, are discontinuous, transverse structures angled into the flow. Instream tips of vanes are usually low enough to be overtopped by nearly all flows; the crest slopes upward generally to the bank-full stage elevation at the bank. They are angled upstream to redirect overtopping flows away from the protected bank (Biedenharn et al., 1997).

They can be used for bank protection, as well as for providing variable depth and velocity that can benefit aquatic organisms. Vanes redirect flow, provide toe protection and reduce or arrests local bank erosion, and result in bed scour downstream of the axis of the vane and near their tips. Flow redirection reduces the velocity and shear stress along the bank, and creates a secondary circulation cell that transfers energy to the center of the channel (Fischenich, 2000), creating a new thalweg location. Vanes can require less rock than riprap for a similar length of bank, and require bank disturbance only where keys are placed.

Some sediment deposition may occur upstream and downstream of the structures, resulting from the redirected flows. In cases where sediment deposition occurs between structures, additional bank protection can develop over time and can be provided more cheaply than through use of riprap revetments. Vanes can be used on rivers where establishing riparian vegetation is a high priority. Once the sediment deposits occur between vanes, natural vegetation begins to grow there, and this could be supplemented by plantings later.

12.2.1 Examples of Application

Lagasse et al. (2009) report that vanes (barbs) are used for bank protection at highway bridge crossings on braided or meandering rivers with small to medium radius bends and for channels up to 150 ft wide in the States of Colorado, Idaho, Illinois, Missouri, Montana, Oregon, and Washington. They also note that there is “limited but successful field experience using bendway weirs/stream barbs as stream instability countermeasures.” NRCS (2007) reports vanes being used at many rivers in the United States. The Washington State Department of Transportation has used vanes for bank protection at highway crossings and in some cases to improve habitat especially in “shallow gravel bed streams” (Brebbia, 2009).

12.2.2 Design Procedure

No systematic quantitative design method for crest length, angle, width, spacing, orientation, or vertical angle for various channel conditions had been available prior to 2012. As noted below, equations have been developed that empirically relate channel-bend velocity to the maximum permissible velocity of the channel bank material. The maximum permissible velocity is the largest velocity at which the bank material does not erode. The equations are based upon the bankline velocity reduction that results from transverse features. The geometric elements used include the length, orientation angle, spacing, bank vane elevation and crest slope of the transverse features, the channel width and depth, and the centerline radius of curvature for the bend. The bankline velocity reduction equations are based on trapezoidal channel physical model results. Additional testing and equation development is planned for the future based on native topography physical and numerical model results.

Two design procedures are presented in the sections below using geometry and bankline velocity reduction, respectively. The key design/construction elements of vanes are length, crest orientation angle, crest elevation and slope, channel width and depth, channel centerline radius of curvature for the bend, local scour, rock size, placement of appropriate footer or foundation rocks, and vane spacing.

12.2.2.1 Geometric Based Design Procedure

This design procedure was developed from experience, including field observations and data, and from Lagasse et al. (2009), McCullah and Gray (2005), and NRCS (2005, 2007d).

Step 1: Determine design flows and hydraulic conditions

The hydraulic conditions during the mean annual flow, the annual low flow, the mean annual high flow level, and the discharge corresponding to the green line should be computed. The hydraulic numerical model should be developed using methods described in Section 4.4 to determine the flow hydraulics. Determine the design top width (T_w) from the upstream approach channel using the hydraulic model results for the mean annual peak flow, or Q_2. Vane design guidance is based upon the flows listed in this section.

Step 2: Determine desired bank line and thalweg location

First sketch the desired thalweg location (flow alignment) with a curve and with upstream and downstream transitions. Consider that the thalweg may shift locations depending on the flow magnitude and bed material size. In channels with gravel or cobble sized bed material the high and low flow thalweg location may be the same. In sand bed channels that are highly mobile and have an erodible point bar, the thalweg will tend to move based on the alignment of flow lines at high or low discharges. The desired flow alignment may be based on preventing further erosion of the outside bank, or reverse erosion of the bank to alter the flow alignment. The curve representing the desired flow alignment should form a smooth upstream and downstream transition. After bendway weir installations, the thalweg is typically shifted near the alignment of the weir tip. Therefore, the stream ends of the bendway weirs should be at or just short of the desired thalweg location.

Next draw an arc that represents the desired bankline location if it is different from the current eroding bankline. The desired bankline could be the existing concave bank or a new bankline that reverses past erosion. Note that the flow alignment and bankline locations may need to be drawn several times after both are first sketched to provide for smooth transitions and proper thalweg location. After vane installation, the thalweg is typically shifted near the alignment of the vane tips.

Step 3: Determine vane length

The effective (perpendicular to bank line tangent) vane length (L_e) should cross the thalweg (dependent upon horizontal angle), and generally range between $T_w/4$ and $T_w/3$ (Johnson et al., 2001; Maryland, 2000). .

The total vane length, L_w, can be calculated as: $L_w = L_e / \sin \theta$, where θ is the horizontal angle between the bank-line tangent and the axis of the vane crest, and L_w is the horizontal length along the axis of the crest from the bankline to the vane tip. The location of the desired thalweg and bankline may need to be altered based upon these spacing criteria.

Rock vanes extending one-third of the bank-full width into the channel and oriented upstream between 20 and 30 degrees, from the bankline tangent, have been shown to move the thalweg an average of 20 percent of the bank-full width away from the eroding bank (Johnson et al., 2001).

Step 4: Determine bank vane elevation

At the bankline, vane height can be between the (a) annual mean high water, or 2-year return period design water surface elevation and (b) the elevation of the lower limit suitable for the establishment of woody riparian vegetation (NRCS, 2007; McCullah and Gray, 2005). For systems which are slightly incised or, where greater protection is desired, designing the bank vane elevation using the Q_2 water surface elevation is recommended. The elevation below which vegetation does not grow will provide the least amount of protection.

For incised channels a water surface elevation greater than Q_2 can be used for greater protection, as long as the other design criteria are met (crest slope between 2 and 8°, and length crosses the thalweg with tip height either D_{100} or embedded into the bed).

Step 5: Determine positions throughout the bend

Determine the position of the first vane from the aerial photographs or drawings of the bend. This is accomplished by drawing a line parallel to the upstream approach channel and extending it to the bankline in the bend. This would be the position of the second vane. A shorter vane should be placed upstream of this location at the same spacing used for vanes throughout the bend. Vanes downstream of the second vane are positioned based upon the vane spacing in Step 6 below.

Step 6: Calculate vane spacing

NRCS (2007b) recommends vane spacing be determined by extending a line downstream from the tip of the second vane (see step 5) parallel to vane two bank line tangent. Where this line intersects the downstream bankline would be the position of the third downstream vane (NRCS, 2007b). Spacing measured along the chord between two adjacent vanes should not exceed $5T_w$.

Step 7: Determine vane orientation angle

The recommended orientation angle is 20 to 30 degrees from the bank-line tangent (Johnson et al., 2001). When R_c/T_w is less than 3 the orientation angle "probably should be less than 20 degrees" (NRCS, 2007). When sinuosity is less than 3, the orientation angle should not exceed about 25 degrees or the vane "can capture too high a proportion of the cross-stream flow, a conditions that results in strong back eddies upstream of the structure" (NRCS, 2005). If necessary, vane orientation angles of up to 45 degrees may also result in acceptable designs (Lagasse et al., 2009), but are not generally recommended.

A smaller orientation angle will result in a longer vane with more bank protection.

Step 8: Determine vane crest slope

Ideally, the vane crest slope should be between 2° and 8° (NRCS, 2005; McCullah and Gray, 2005). The height of the vane at the bankline, the length, the orientation angle, and the location of the thalweg all affect the vertical vane angle.

The crest slope is oriented so that the tip will be inundated at most flows, with the crest sloping up to the bank-full stage elevation at the bankline. Table 12–1 shows the ranges of conditions which must be satisfied. Several iterations may be necessary to determine the optimal vane height, length, orientation angle, and vertical slope while crossing the thalweg with each structure.

Table 12–1. Range of Conditions to Satisfy for Vane Geometric Design

	Vane Height at Bankline	Effective Length ($L_e = L_w \times \sin\theta$)	Orientation Angle	Vertical Slope
Acceptable Range	Lowest elevation at which vegetation grows up to bankfull or design discharge water surface elevation	$T_w/4$ to $T_w/3$	20° to 45° with 20° to 30° recommended, when $R_c / T_w < 3$ no greater than 25°	2–8%

Step 9: Determine vane key length

The key length can be designed using key length of a bendway weir (see previous section).

Step 10: Determine riprap size

The rock material can be designed using riprap sizing for bendway weir (see previous section).

Step 11: Determine vane top width

The top width should not be smaller than (1 to 3) x D100 and typically is between 3 ft and 12 ft. Vanes that are too long to be constructed by bank-based equipment will need to be wide enough (e.g., 12–15 ft) for equipment to safely drive out onto them. The vane top width may be determined by the volume of riprap material needed for launching into the tip and downstream scour hole to provide for vane stability.

Step 12: Estimate scour and launch riprap volume

Scour depth and launch riprap volume can be determined using the procedures for bendway weirs (see previous section). The "*Z*" value for vanes is the same as bendway weirs (~0.5).

Step 13: Check constructability

The constructability issues are very similar to those of bendway weirs discussed in the previous section.

12.2.2.2 Bankline Velocity Reduction Based Design Procedure

As an alternative to the above geometric approach, design procedures were developed using the statistical regression equation from Scurlock et al. (2012), Scurlock et al. (2015), and previously unpublished regressions based upon physical model measurements as described below. This method was developed because no systematic quantitative design methods have existed for calculating crest length, angle, width, and transverse feature spacing for various channel conditions (Abad et al., 2008; Lyn and Cunningham, 2010). Past design guidelines do not address approach velocity, which can influence the effectiveness of bank erosion control with bendway weirs (Lyn and Cunningham, 2010).

As a result of the lack of quantitative methods, Colorado State University researchers built and studied a physical model (funded by Reclamation) of a trapezoidal channel to determine the effects of crest length, planview angle, crest slope, radius of curvature, channel width, and spacing on the high velocity on the concave (outside) bank of eroding river bends. They measured the reduction in bankline velocity and shear stress as a result of transverse features for a variety of geometries. They incorporated their laboratory measurements into a backward linear regression statistical method to develop design equations based upon the significant dimensionless variables for river bends and transverse features (Scurlock et al., 2012). The results of the first phase of this work, which are based upon trapezoidal channel measurements, are presented here. Results from the second phase of this work, which will include the effects of bed topography, will be added after completion.

Using the ratio of maximum velocity within a transverse structure field and channel bed as compared to average velocity under baseline channel conditions (pre-structure), allows one to estimate the bank or bed stability based upon transverse structure and channel hydraulic conditions. The maximum velocity ratio (MVR_o) is the maximum velocity along the outer (concave) bankline, within the transverse structure field divided by the baseline bend averaged velocity ($MVR_o = V_{bank} / V_{baseline}$). The average velocity ratio can also be used for design, defined as $AVR_o = AV_{bank} / V_{baseline}$, where AV_{bank} is the average velocity along the outside of the bend. The maximum and average velocity ratios can also be used for the centerline and inside of bend as an indication of potential for bed material transport as a result of transverse feature installation.

The baseline bend average velocity can be determined from a one-dimensional backwater model such as HEC-RAS, using the mean velocity for each model cross section. Between the upstream and downstream crossing or riffle, each cross sectional mean velocity is averaged by weighting the distance between each pair of adjacent cross sections.

The velocity along the bankline for design purposes is the greatest velocity possible without eroding bank material (permissible velocity). Permissible velocities are found in Tables 4–2 and 4–3.

MVR and AVR can be estimated (Scurlock et al., 2012) using:

$$MVR, AVR = a_1 + a_2(A^*)^{a_3}\left(\frac{L_{ARC}}{T_W}\right)^{a_4}\left(\frac{R_C}{T_W}\right)^{a_5}\left(\frac{L_{W-PROJ}}{T_W}\right)^{a_6}\left(\frac{D_B}{D_B-\Delta z}\right)^{a_7}\left(\frac{2\theta}{\pi}\right)^{a_8} \tag{12.8}$$

where:

- $L_{W\text{-}PROJ}$ = projected length of structure into channel [L];
- L_{ARC} = arc length between centerline of structures [L];
- R_C = radius of curvature of channel bend centerline [L];
- T_W = averaged top width of channel measured at baseline in bend [L];
- D_B = averaged maximum cross-section baseline flow depth in bend [L];
- Δz = elevation difference between water surface and structure crest at the tip [L];
- θ = structure plan angle [radians];
- A^* = percentage of projected cross-sectional weir area to baseline cross-sectional flow area at design flow and is function of $L_{W\text{-}PROJ}$, D_B, Δz, θ and bed topography.
- $a_1,\ldots,a_8$ = regression coefficients, a_1 is an offset term to create an upper envelope to include all laboratory data two standard deviations away from the central tendency regression line for regression coefficients a_2 through a_8 .

Moving left to right in Equation 12.8, dimensionless terms can be described as a structure spacing ratio, a curvature ratio, a length ratio, a depth ratio, and a measure of the angle of the structure into the channel. Figure 12–5 provides a plan view of installed structures in a channel bend and the definition of variables in Equation 12.8. Figure 12–6 shows a cross section schematic of vanes along with spur-dikes.

Values of the regression coefficients are given in Table 12–2. Where *MVR* or *AVR* do not depend upon that dimensionless variable the exponent regression coefficient is zero. The subscripts *o*, *c*, and *i* denote the velocity ratio for the outside bank of the bend, the channel centerline, and the inner bank, respectively. Equation 12.8 applies only for the range values of dimensionless variables tested in the trapezoidal physical model shown in Table 12–3. Note that the laboratory vane testing covered plan view (horizontal) vane crest angles between 60^0 and 90^0 while the NRCS (2005, 2007) design criteria ranges from 20^0 to 45^0. For each method the horizontal angles should be in their respective ranges. The second (future) phase of the work to develop design methodology and design equations using native bed topography channels will include the NRCS (2005, 2007) design criteria as part of the test range.

Equation 12.8 applies to vanes that have a bankline elevation equal to the annual mean peak flow, or Q_2. These equations do not apply for cases in which vane structures along the bankline are significantly submerged, such as where the bankline vane elevation corresponds to the lower limit suitable for the establishment of woody riparian vegetation (NRCS, 2007; McCullah and Gray, 2005) or to one-half or one-third of the bank-full flow depth (NRCS, 2007).

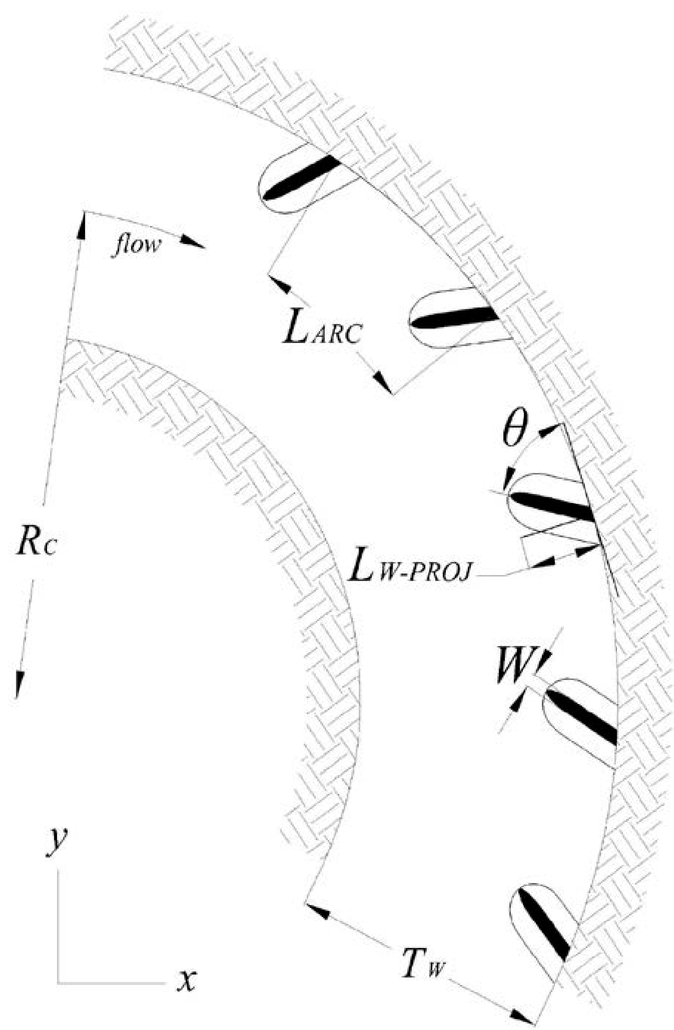


Figure 12–5. Plan schematic of transverse feature parameters (Scurlock et al. 2012).

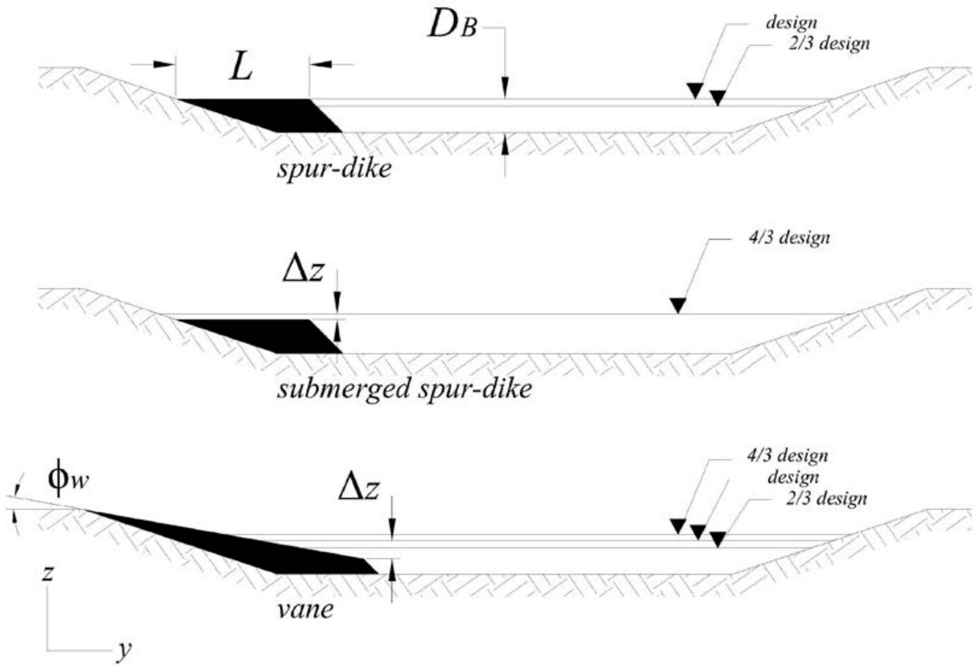


Figure 12–6. Cross section and structure profile view schematic of evaluated structures in trapezoidal model (Scurlock et al. 2012).

Table 12–2. Regression Coefficients for Equation 12.8, for Vanes

	a_1	a_2	a_3	a_4	a_5	a_6	a_7	a_8
MVRo	0.237	0.015	0.000	0.794	0.0000	–2.000	0.614	0.350
MVRc	0.197	0.620	0.182	0.000	0.191	0.000	–0.025	0.105
MVRi	0.176	0.696	0.213	–0.116	0.093	0.000	–0.020	0.151
AVRo	0.058	0.061	0.000	0.268	0.000	–0.593	0.237	0.264
AVRc	0.166	0.538	0.212	–0.101	0.188	0.000	–0.046	0.140
AVRi	0.131	0.132	0.489	–0.131	0.177	–0.410	0.117	0.127

Table 12–3. Range of Dimensionless Variables to Use in Equation 12.8

A^*		L_{ARC}/T_W		R_C/T_W		L_{WPROJ}/T_W		Depth Ratio		$2\theta/\pi$ (radians) (60 to 90°)	
Max	*Min*	*Max*	*Min*	*Max*	*Min*	*Max*	*Min*	*Max*	*Min*	*Max*	*Min*
27.00	10.75	3.085	0.547	6.862	2.479	0.373	0.140	6.984	0.768	1.000	0.667

Sensitivity analysis (Figure 12–7) was completed for each of the dimensionless variables in Equation 12.8 and the regression coefficients in Table 12–2. The purpose of this analysis was to show how each dimensionless variable affects *MVR* or *AVR* through and past laboratory ranges. When the velocity ratio is greater than 1 for *MVRo* the design results in higher bankline velocity than bend average velocity without the transverse features. The lower and upper bound of the laboratory range of data is designated by the vertical red lines in Figure 12–7. The *MVRo* as a result of changes in $L_{W\text{-}PROJ}/T_W$ over the range of laboratory testing shows a large rate of change and should be kept greater than about 0.2 to provide velocity reduction. D_B should be in the range of about 1 to 2.5, and R_c/T_W greater than 2.5 for suitable velocity reduction.

Table 12–4 shows the median value of each dimensionless variable and the upper and lower limit of the sensitivity analysis. The sensitivity analysis consisted of successively changing one dimensionless variable at a time while holding all other dimensionless variables constant as possible. It was not always possible to keep each dimensionless variable constant while sequentially changing because of inter-dependence between variables. For example, in the case where R_c/T_w is varied by sequentially changing T_w, $L_{w\text{-}proj}$ remained the same and the dimensionless ratio $L_{w\text{-}proj}/T_w$ is also sequentially changed.

Sensitivity was completed using R_c = 180 ft, T_w = 790 ft, and D_b = 9.55 and an average depth of 9.55 ft to calculate the dimensionless ratio values and sequentially alter each ratio Figure 12–7). Some of the dimensionless ratios exceed unity throughout a large portion of the laboratory testing ranges. These graphs are intended to show how MVR/AVR change inside and outside the laboratory parameters rather than be values suitable for design. There was not a laboratory test for the case where median values were used for all dimensionless variables.

Use of this design method requires the bank material and bed material median (D_{50}) size and a classification of the bank material sufficient to determine the permissible velocity. Steps 3, 4, and 5 are initial estimates of length and horizontal orientation angle, spacing, and vertical angle respectively. Several iterations may be necessary to determine the optimal, length, orientation angle, and Δz (vertical slope) for each structure crossing the thalweg, so that all dimensionless variables are within the limits in Table 12–4.

Step 1. <u>Determine bend hydraulics and design water surface elevations</u>

Determine the 2-, 10, and 25 year return period discharge peaks for riprap and scour design. See Section 4.4 for more information about hydrology and bend hydraulics. Additional guidance is given here for bankline velocity reduction designs.

Determine the baseline bend average velocity. In cases where there is overbank flow at the design discharge (mean annual peak flow, or Q_2), the main channel baseline bend average velocity should be used. This is the hydraulic condition the transverse features would be exposed to. The baseline bend average velocity is found from a one-dimensional backwater model such as HEC-RAS, using the mean velocity for each model cross section. Between the upstream and downstream crossing or riffle, each cross sectional mean velocity is distance weighted to compute the average bend velocity.

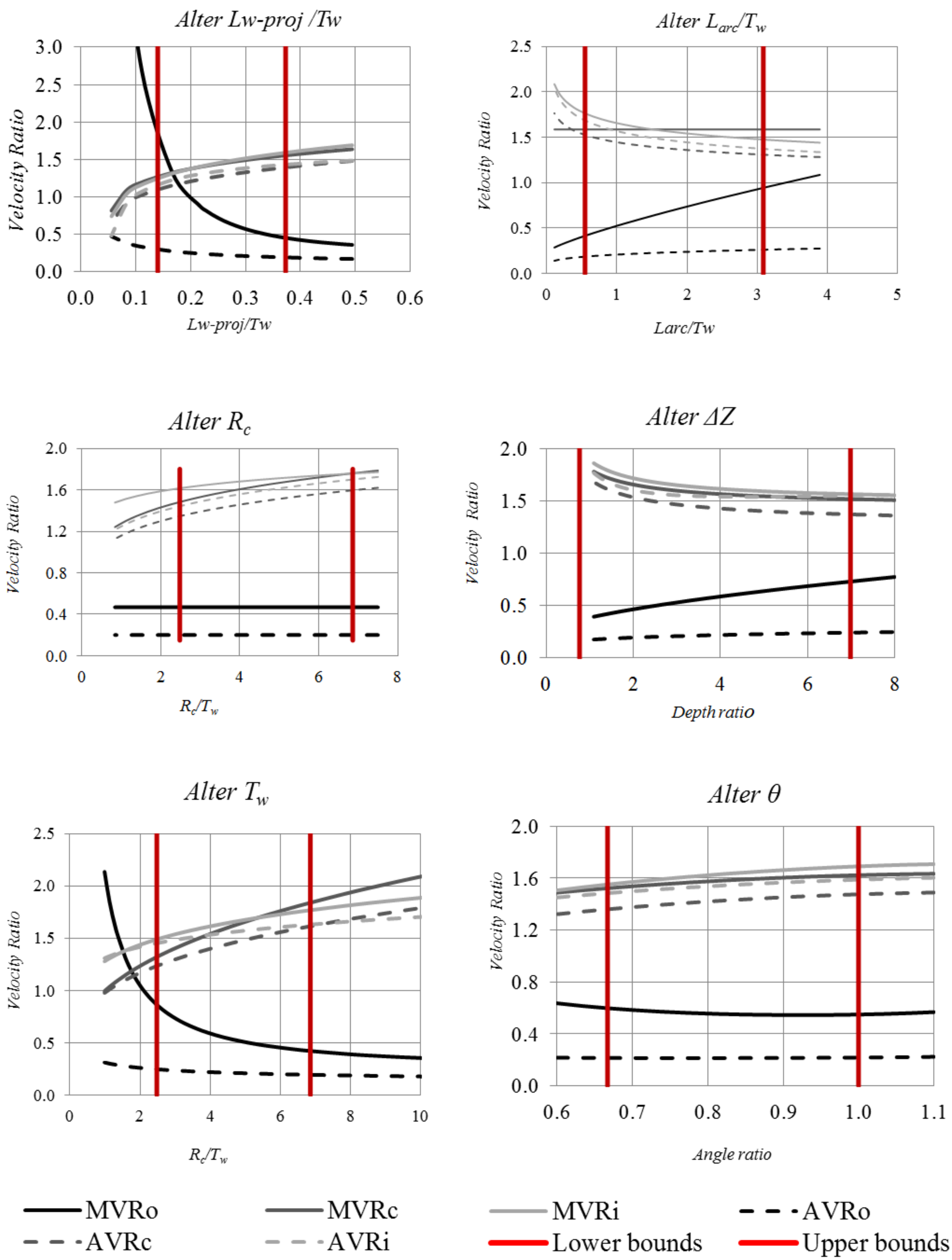


Figure 12–7. Vane velocity equation parameter response.

Table 12–4. Range of Laboratory and Sensitivity Dimensionless Variable Testing

Dimensionless Variable	Upper Limit of Laboratory Testing	Lower Limit of Laboratory Testing	Median of Laboratory Testing	Median Value Used in Sensitivity Analysis median	Upper Limit of Sensitivity Testing	Lower Limit of Sensitivity Testing
A^*	27	10.75	18.87	13.53	29	2.3
L_{arc}/T_w	3.08	0.55	1.82	1.82	4	0
R_c/T_w	6.86	2.48	4.67	4.67	10	1.5
L_{wproj}/T	0.37	0.14	0.28	0.26	0.56	0.04
$D_B/(D_B\text{-}\Delta Z)$	6.98	0.77	3.88	3.88	10	0.1
$2\theta/\pi$ (Radians)	1	0.67	0.83	0.83	1.1 (99^o)	0.06 (51^o)

Determine the baseline bend averaged maximum flow depth (D_B). This is the distance-weighted average of the maximum design flow depth from each cross section in the bend excluding the upstream and downstream crossing or riffle sections.

Determine the baseline bend averaged design flow channel cross sectional area. This is the distance-weighted average of the flow area for each cross section in the bend excluding the upstream and downstream crossing or riffle sections. Determine the design top width from the upstream approach channel using the HEC-RAS results for the mean annual peak flow, or Q_2.

Step 2: Determine bank vane elevation.

Use either the mean annual peak flow water surface elevation (Lagasse et al., 2009) or the elevation below which vegetation does not grow (McCullah and Gray, 2005). For channels which are slightly incised or where greater protection is desired, matching the bank vane elevation to the Q_2 water surface elevation would be best. The elevation below which vegetation does not grow will provide the least amount of protection. For bends where bank erosion control is necessary but for which the riverside infrastructure is of medium to lower value, the elevation of the mean annual peak flow water surface elevation should be used. This elevation is preliminary and may need to be adjusted during later steps.

Step 3: First estimate of vane length, angle, spacing and slope

The first approximation of vane length, spacing, angle, and slope should be accomplished using the procedure used for the geometric design procedure. Vane geometry may need to be adjusted in design Step 5, so that the maximum permissible velocity for protection of the bank material is not exceeded.

Step 4: <u>Initial calculation of A^* (projected percent of the flow cross sectional area occupied by the vanes</u>)

Calculate the projected cross sectional area of the vane (A_v) initially by assuming a flat bottom at the maximum flow depth and the average bank angle along the outside of the bend. Note that the estimate may be refined using the cross section geometry nearest each vane or upon distance-weighted cross section geometry. The projected percent of the flow cross sectional area occupied by the vanes is the vane $A_{v\text{-}proj} = A_v \times \sin\theta$, where θ is the angle between the bankline tangent and the axis of the vane crest (ranging from 60° to 90°).

From the vane tip to the channel bed the area is determined by the angle of repose of the riprap material. A side slope of 1V to 1.5H or 1V to 1.2H is generally used for this calculation. The projected vane length is used. The percent of the flow cross sectional area occupied by the vanes is the projected vane area divided by the baseline average bend channel cross sectional area (Step 1), multiplied by 100.

The projected area of the vane will likely need to be adjusted in design Step 5 so that it does not cause the flow to exceed the maximum permissible velocity for protection of the bank material.

Step 5: <u>Initial and iterative calculation of velocity reduction</u>

The primary design variable from which to estimate the stability of the eroding outside bank of the bend is $MVR_{o.}$, while AVR_o shows the bend average. Secondary design variables are the velocity ratios for the centerline and inner bank. These can be used as an indicator of the erodibility of the bed and inner bank. When large stone elements are used instead of well graded riprap to construct the vane, the MVR/AVR_c could be used to size the elements
Use Equation 12.8 with the appropriate regression coefficients and input variables to calculate $MVR_{o.}$ The bankline velocity predicted by Equation 12.8 is $V_{bank} = MVR_o \times V_{baseline}$. While the main design variable is MVR_o a non-conservative approach would be to use the AVR_o relationship because this will show less velocity reduction: $V_{bank} = AVR_o \times V_{baseline}$. Use the centerline and inner bank MVR/AVR regression coefficients to estimate the velocity at these locations. Increasing the inner channel (centerline) velocity will generally result in local channel deepening and possibly reduce point bar deposition. Inner bank velocities may be used to determine the size of material on the point bar which may help produce suitable substrate for fish species of interest.

If V_{bank} is greater than the permissible velocity (Table 4–2 and 4–3) for protection of the bank material, then the $L_{W\text{-}PROJ}$ should be incrementally increased and L_{ARC} and Δz incrementally decreased until V_{bank} equals or is less than the permissible velocity based upon MVR_o. The most economical vane design will generally use the least amount of riprap to achieve the desired velocity reduction.

Step 6: Determine vane positions throughout the bend

Determine the position of the first vane from the aerial photographs or drawings of the bend. This is accomplished by drawing a line through the approach channel centerline and extending to the bankline. This would be the position of the second vane. A shorter vane should be placed upstream of this location at the same spacing as used for vanes throughout the bend.

Position the location of each vane along the bankline on the aerial photograph or drawing of the bend using either the chord length between vanes or the arc length. The arc length would be slightly more conservative.

Step 7: Determine final vane length ($L_{W\text{-}PROJ}$), spacing (L_{ARC})and depth to vane tip (Δz)

Using the existing cross section and vane locations throughout the bend, interpolate between cross sections if needed to determine the actual percentage of the projected cross-sectional vane area to average cross-sectional area of the baseline bend (A^*). All vane areas should be averaged together since Equation 12.8 is based upon bend average values with a coefficient to account for measured data within two standard deviations of the central tendency line predicted by the regression equation.

Step 8: Determine the final centerline and inside-of-bend velocity changes

Compute $MVR/AVR_{c,,}$ MVR/AVR_i , and compare with the permissible velocities for protection of the bed material and the inner bank material. Centerline and inner bank velocity should increase following construction, since some of the cross sectional area will be blocked by the vane structures, there will be added roughness, and the outer bankline velocity is reduced. Comparing centerline and inner bank velocities to permissible velocities will give an indication of the stability of the bed and inner bank.

Step 9: Determine vane key length, riprap size, vane top width, scour, and launch riprap volume

The procedures for the vane key length, riprap size, vane top width, scour, and launch riprap volume are identical to the geometric design procedure for vanes.

Step 10: Review Constructability

The constructability issues are essentially the same as those of the geometric design procedure for vanes.

12.2.2.3 Discussion and Recommendations

Vanes can be combined with other biotechnical soil stabilization measures for areas between the vanes. If significant scalloping is anticipated then an armoring layer of cobbles or gravel or a longitudinal stone toe can be added in between vanes (McCullah and Gray, 2005). Cobbles and gravel could be native material from the river.

Risk and Failure

The most common causes of vane failure are undermining due to scour and flanking by the river (bank erosion around and behind the structure). The scour results from the fact that the hydrodynamic forces on vane tips are large and the rock volume near the tips of the vanes is small. Countermeasures for these risks can be launching riprap and active monitoring and maintenance. Providing additional downstream footers also can be used as a countermeasure. There should be some distance between the protected bankline and riverside infrastructure to allow for bank scalloping that often occurs. A common long-term maintenance activity for vanes is adding riprap to the vane tip. Flanking can be prevented by keying the structure into the bank adequately.

A secondary cause of vane failure is the entrance angle changing due to upstream channel migration, resulting in a larger flow entrance angle with concurrent increases in velocity and scour. A countermeasure for this situation can be upstream bend stabilization and adding more rock as a safety factor, to launch into the scour hole. It may also be necessary to accept that upstream bend migration may occur and that the vanes may need to be repositioned in the flow.

Bank scalloping between vanes is a common occurrence and can be excessive if the vane spacing is too large. Long-term bank protection is best provided if sediment deposition and subsequent vegetation are sufficient to create a dense riparian zone (McCullah and Gray, 2005).

Because the risk of scour and bank scalloping increases as the width-to-depth ratio decreases, it is suggested that vanes only be used in channels with a width-to-depth ratio of 12 or greater (Maryland, 2000).

Improper vane angle and height can redirect flow into unintended places, creating further bank erosion downstream from the structures (Johnson et al., 2001).

Some failures are related to seepage under and around the structures. Proper footer placement and, in some cases, additional deeper footer rocks are a countermeasure for this failure mode. Gravel or fabric filters can also help reduce seepage failure.

Monitoring and Maintenance

Regular monitoring and maintenance are required for the vanes to remain functioning during their service life. Due to the hydrodynamic forces on the vane tips and scour generated by vanes, riprap erosion is common. The amount of riprap erosion should be monitored to ensure that it does not exceed the amount accounted for in the design.

12.3 Spur Dikes (Groins, Hard Points, L Dikes, and T Dikes)

Spur dikes are a series of individual structures that are placed transverse to the flow, projecting from the riverbank with a horizontal crest, usually at the elevation of the top of the bank. Earth core spur dikes are constructed with a soil core armored by a layer of riprap stone and have an embedded stone toe on the upstream side. Spur dikes can be capped with a prism of earth reinforced with live fascines or other types of riparian woody vegetation. The earth prism can be constructed with alternating layers of live fascines and earth.

Spurs deflect flow away from the bank, reducing the near-bank velocity and, thus, preventing erosion of the bank in critical areas. Spurs can also be used to reduce the channel width and induce sediment deposition along the bankline between individual spur dikes. Spur dikes can create variable depth and velocity habitat, can lead to sediment deposition along the bankline and can generally be constructed for less cost than riprap revetments or longitudinal stone toes with bio-engineering.

Spur dikes are generally constructed with a downstream angle or perpendicular to the bankline tangent for bank protection purposes (Lagasse et al., 2009). Spurs oriented in the upstream direction generally protect less bank length downstream of the spur tip for the same spur length, have greater scour depth at the tip and increase hydraulic roughness. Spurs oriented 90^o forces the main flow current farther from the concave bank than spurs oriented either upstream or downstream. Thus, more bank protecting flow re-direction is achieved with spurs oriented approximately normal to the channel bank. Spurs oriented upstream cause greater scour than when normal to the bank and downstream oriented spurs cause less scour. Therefore, spur dikes oriented 90^o results in the greatest benefit for their length and are recommended to reduce tip scour.

Spur dikes are generally used to halt meander migration. They can also be used to narrow channels that are overly wide and create a deeper main channel. Increased bank protection can occur over time if sediment accumulates downstream of spur dikes.

There are several variations on spur dikes. Hard points are short spur dikes (Biedenharn et al., 1997) that add roughness and localized bank stability. An L-head, (hockey stick), or a T-head added to the spur tip can move scour away from the dike (Biedenharn et al., 1997). In cases of tight river bends, the spur dike can connect to a longitudinal dike, in an L or T shape, off of the spur dike tip to effectively reduce the number of dikes required. Some types of spurs are permeable, such as fence type, jacks, or tetrahedron spurs, and others are impermeable, such as transverse dikes constructed of riprap. This guideline applies to impermeable transverse spur dikes. Design guidance for permeable spur dikes can be found in Lagasse et al. (2009).

12.3.1 Examples of Application

The most common use of spur dikes is in shallow, wide streams with moderate to high suspended sediment loads. The shallow depth reduces the height of the structures needed, while the wide channel provides opportunity for the channel alignment and geometry to adjust to the presence of the spur dikes (Biedenharn et al., 1997). Spur dikes are often used on large rivers to increase channel depth and improve channel alignment and stabilization of the banks (McCullah and Gray, 2005). Spur dikes can be used on smaller, higher gradient streams with slightly tighter spacing than on lower gradient streams (WDFW, 2003).

Spur dikes can be used on rivers where establishing riparian vegetation is a high priority. As soon as the sediment deposits form between the spur dikes, natural vegetation commences, which could be supplemented by plantings later. Channel capacity at high flow can decrease initially, depending on the level of flow restriction caused by the spurs. The channel usually adjusts by forming a deeper, narrower cross section (McCullah and Gray, 2005). The extent of the local adjustment cannot be reliably predicted (Biedenharn et al., 1997).

Spur dikes are commonly used at highway bridge crossings for braided or meandering rivers with long radius bends that are wide or moderately wide (Lagasse et al., 2009). Spur dikes had success for relatively long radius bends with a wide range of outer bank shear intensities in eight streams in Mississippi, where R_c/W values were greater than about 3 (Thorne et al., 2003).

12.3.2 Design Procedures

No systematic quantitative design method has existed for crest length, angle, width, and spur spacing for various channel conditions (Abad et al., 2008; Lyn and Cunningham, 2010). Past transverse feature design methods failed to address approach velocity, which can influence the effectiveness of bank erosion control (Lyn and Cunningham, 2010). Design methods have been based upon channel and structure geometry (Lagasse et al., 2009; NRCS, 2007) without consideration for channel velocity and bank material. Equations have been developed that empirically relate channel-bend velocity, to the maximum permissible velocity for protection of the channel bank material. The equations (12.9) are based upon the bankline velocity reduction that results from transverse features. The geometric elements used include the length, orientation angle, spacing, and vertical angle of the transverse features, the channel width and depth, and the centerline radius of curvature for the bend.

Two design procedures are presented in Sections 6.4.4.5 and 6.4.4.6 using geometry and bankline velocity reduction, respectively. Important design variables and considerations include: limits of protection, spur length and spacing, crest angle and crest elevation channel width and depth, channel centerline radius of curvature for the bend, local scour, riprap sizing, and key length.

12.3.2.1 Geometric Based Design Procedure

Design procedures were developed from experience including field observations and data, and available publications.

Step 1: <u>Determine design flows and hydraulic conditions</u>

The hydraulic conditions during the mean annual flow, the annual low flow, the mean annual high flow level, and the flow above which significant riparian vegetation grows should be computed. The hydraulic numerical model should be developed using methods described in Section 4.4 to determine the flow hydraulics. Determine the design top width (T_w) from the upstream approach channel using the hydraulic model results for the mean annual peak flow, or Q_2

Step 2: <u>Determine desired bank line</u>

The desired flow alignment may be based on preventing further erosion of the outside bankline, or reversing erosion of the bank to alter the flow alignment. The curve representing the desired flow alignment should form a smooth upstream and downstream transition. Draw the desired thalweg and bankline locations according to the procedure in the bendway section.

Step 3: <u>Determine spur dike elevation</u>

Spur dikes are usually constructed at an elevation equal to the bank height, regardless of whether the design flow is higher or lower than that elevation. If the design water surface elevation is higher, constructing the spurs at the bank height will prevent overtopping flows from being diverted around the spur dikes along the bankline, which could cause flanking (Brown, 1985). If the bank height is such that constructing spurs to that elevation is expensive then the elevation of the mean annual flow , or Q_2, could be used provided that a transition to the bank elevation is included so that overtopping the spur dike near the bank will not occur (Lagasse et. al, 2009).

Step 4: Determine spur dike length

Lengths typically range from less than 15 percent to about 30 percent of the bank-full or design flow channel width (Brown, 1985; Lagasse et al., 2009). Spur dikes in laboratory tests show diminishing returns when spur length is greater than 20 percent of the channel width. Analysis of the physical model data from the Colorado State University (Scurlock et al., 2012) shows that when length is below about 15% of the bank-full or design width the bankline velocity can increase above the mean cross sectional velocity. This is due to flow expansion downstream of the spur dike tip. When the desired length is less than 15% of the bank-full or design width, spacing spur dikes closer together can prevent scalloping. Use Figure 12-2 to place the next downstream spur dike upstream of where the 20 degree flow expansion angle intersects the bank, instead of using this figure for determining the key length. However, field installations have been successful with lengths from 3 to 30 percent of channel width (Lagasse et al., 2009). These lengths are for cases in which the project objective is to prevent further erosion. If the objective is to reverse past erosion and alter flow alignment, the spur dike lengths will be based upon achieving the desired change in flow path. For cases where flow constriction is an objective, the spur dike length will be based upon the degree of constriction needed. At locations where excessive constriction is expected, excavation of the inside of the bend may be required. However, the constricted flows may also erode the inner bank.

Step 5: Determine slope of spur dike

The crest profile should be nearly level with a slight downward slope towards the tip, because it is difficult to construct and maintain a level spur. Use of a sloping spur avoids the possibility of overtopping at a low point in the spur profile, which could cause riprap erosion or damage to the streambank (Lagasse et al., 2009).

Step 6: Determine spur dike key length

The key length can be determined in a similar manner to the methods used for bendway weirs. For spur dikes less than 15% of the design channel width, Equation 12.5 should be used.

Step 7: Determine spur dike spacing

Spacing is dependent upon spur length, spur angle, and the degree of bend curvature. The angle at which flow expands (flow expansion angle) toward the bank downstream of the spur is most often near 17° (Brown, 1985). Spur spacing can be determined by first drawing an arc representing the desired alignment of the flow thalweg. The desired flow alignment may be based on preventing further erosion of the outside bankline, or reverse erosion of the bank to alter the flow alignment. The curve representing the desired flow alignment should form a smooth upstream and downstream transition. The next step is to draw an arc that represents the desired bankline location. The desired bankline could be the existing concave bank or a new bankline that reverses past erosion. Estimate the length of the spurs along the desired bankline location. Spur length, along with the expansion angle fixes the spacing between spurs (Lagasse et al., 2009). Flow leaves the tip of the spur dike and diverges toward the bank at the expansion angle, and the next downstream spur dike should be positioned to intercept this redirected flow before it reaches the bank: ($L_{ARC} = L_w \cot \varphi$), where φ is the downstream flow expansion angle. In many cases this corresponds to a spacing of about 3.3 times the length of the spur dikes (Washington, 2002). Others suggest spacing should be 1.5 times the spur length (McCullough and Gray, 2005). The actual spacing should be determined based upon the desired flow alignment, the new bank location (if needed), and the bank curvature. Spacing decreases as the bend radius decreases; therefore, the cost of spur dikes in low radius bends is so high that a longitudinal stone toe with bio-engineering may be more economical.

The most upstream spur should be angled downstream to provide a smooth transition for flow approaching the spur field. The upstream spurs should have conservative key lengths to prevent upstream flanking.

Step 8: <u>Determine spur dike angle</u>

There is no consensus regarding the orientation of spur dikes (Lagasse et al., 2009). However, there is some agreement that spur dikes pointing upstream do not protect as great a length of channel bank downstream, result in greater tip scour depth, and have a greater tendency to accumulate ice and debris (Lagasse et al., 2009). Spurs oriented downstream have less scour than either spurs pointing upstream or those perpendicular to the bankline tangent (Brown, 1985). Spur orientation of approximately 90° forces the main flow current and thalweg away from the eroding bank with the shortest length and cost. Crest orientation angle generally varies from perpendicular to the approach flow to being angled downstream at an angle 60 degrees from the bankline tangent. When the farthest upstream spur is angled downstream, the transition of flow lines is smoother near the bank and tip scour is reduced. Subsequent downstream spurs are recommended to be normal to the bankline tangent and to the flow direction to minimize construction cost. Ranges of entrance angles are not always reported in the literature for successful installations. More review of the literature is needed to determine the entrance angles of successful installations.

Step 9: <u>Determine riprap size</u>

The key length can be determined in a similar manner to the methods used for bendway weirs.

Step 10: <u>Determine spur dike top width</u>

The top width should not be smaller than (1 to 3) x D_{100} and typically is between 3 ft and 12 ft. Spur dikes that are too long to be constructed by bank-based equipment will need to be wide enough (e.g., 12–15 ft) for equipment to safely drive out onto them. The dike top width may be determined by the volume of riprap material needed for launching into the tip and downstream scour hole to provide for dike stability.

Crest width usually varies from 2 or 3 times D_{100} to wider if more launchable rock is desired (McCullah and Gray, 2005). If the spur dikes cannot be constructed from the bankline, then the width will need to be sufficient for safe equipment access.

Step 11: Estimate scour and launch riprap volume

By this stage of the design a scour evaluation (Chapter 5 and Appendix A) will have already occurred. By re-directing flow around the tip of the spur dike flow acceleration causes local scour to occur. While studies have been done to estimate spur dike scour (Garde et al., 1961; Gill, 1972) there is no universally accepted scour estimating method. Some researchers consider spur dike scour to be essentially the same as bridge pier scour (Melville and Coleman, 2000). It is recommended that the Blench (1969) method be used as found in the section on bendway weirs. Based upon experience and evaluation of the limited data by Cox (2005) the Z in Equation 12.7, should range between 1 and 1.75. This Z value is in the range reported by Pemberton and Lara (1984) for nose of guide banks. As a factor of safety, for high value infrastructure it is suggested that d_s be multiplied by 2.

The launchable riprap volume should be estimated using a 1V:1H slope to the scour depth with a thickness of at least D_{100} plus 30 percent for rock dislodgment during launching. The volume is determined based upon structure and cross section geometry. For spur dikes constructed in flowing water the volume should be increased by at least 20% or more to account for rock erosion during placement.

Step 12: Review Constructability

The constructability issues are similar to those of the construction of bendway weirs.

12.3.2.2 Bankline Velocity Reduction Based Design Procedure

No systematic quantitative design methods have existed for crest length, angle, width, and transverse feature spacing for various channel conditions (Abad et al., 2008; Lyn and Cunningham, 2010). Past design guidelines were independent of approach velocity, which can influence the effectiveness of bank erosion control with transverse features (Lyn and Cunningham, 2010). As a result of the lack of quantitative methods, Colorado State University researchers built and studied a physical model (funded by Reclamation) of a trapezoidal channel to determine the effects of crest length, planview and vertical angles, channel width, and spacing on the high velocity on the concave (outside) bank of eroding river bends. They measured the reduction in bankline velocity and shear stress as a result of transverse features for a variety of geometries. They have incorporated their laboratory measurements into a backward linear regression statistical method to develop design equations based upon the significant dimensionless variables for river bends and transverse features (Scurlock et al. 2012; Scurlock et al. 2015). The results of the first phase of this work, based upon trapezoidal channel measurements, are presented here.

Using the ratio of maximum velocity within a transverse structure field and channel bed as compared to average velocity under baseline channel conditions (pre-structure), allows one to estimate the bank or bed stability based upon transverse structure and channel conditions. The maximum velocity ratio (MVR_o) is the velocity along the outer (concave) bankline, within the transverse structure field divided by the baseline reach averaged velocity ($MVR_o = V_{bank} / V_{baseline}$). The average velocity ratio can also be used for design, defined as $AVR_o = AV_{bank} / V_{baseline}$, where AV_{bank} is the average of the bankline. Maximum and average velocity ratios can also be used for the centerline and inside of bend as an indication of the potential for bed material transport that would result from transverse feature installation.

The baseline reach average velocity is determined from a one-dimensional backwater model such as HEC-RAS, using the mean velocity for each model cross section. Between the upstream and downstream crossing or riffle, each cross sectional mean velocity is averaged by weighting the distance between each pair of adjacent cross sections. The velocity along the bankline for design purposes is the greatest velocity possible without eroding bank material (permissible velocity).

MVR and *AVR* can be estimated for spur dikes (Scurlock et al. 2012a) using:

$$MVR, AVR = a_1 + a_2 (A^*)^{a_3} \left(\frac{L_{ARC}}{T_W}\right)^{a_4} \left(\frac{R_C}{T_W}\right)^{a_5} \left(\frac{L_{W-PROJ}}{T_W}\right)^{a_6} \left(\frac{2\theta}{\pi}\right)^{a_7} \tag{12.9}$$

where:

$L_{W\text{-}PROJ}$ = projected length of structure into channel [L];

L_{ARC} = arc length between centerline of structures [L];

R_C = radius of curvature of channel bend centerline [L];

T_W = averaged top width of channel measured at baseline in bend [L];

θ = structure plan angle [radians];

A^* = percentage of projected cross-sectional weir area to baseline cross-sectional flow area at design flow and is function of $L_{W\text{-}PROJ}$, D_B, *and* θ. Where D_B is the averaged maximum cross-section baseline flow depth in bend [L]

$a_1,\ldots,a_7$ = regression coefficients, a_1 is an offset term to create an upper envelope to include all laboratory data two standard deviations away from the central tendency regression line for regression coefficients a_2 through a_8 .

Equation 12.9 is similar to equation 12.8 but without the depth ratio term $\left(\frac{D_B}{D_B-\Delta z}\right)$ since for spur dikes Δz is zero, hence the depth ratio term is equal to 1. Moving left to right in Equation 12.9, dimensionless terms can be described as a structure spacing ratio, a curvature ratio, a length ratio, and a measure of the angle of the structure into the channel. Values of the regression coefficients are given in Table 12–5.

Table 12–5. Regression Coefficients for Equation (12.9), for Spur-Dikes (depth ratio = 1)

	a_1	a_2	a_3	a_4	a_5	a_6	a_7
MVRo	0.238	0.024	-0.222	1.018	0.000	-2.000	0.539
MVRc	0.197	1.677	0.000	-0.068	0.249	0.355	0.187
MVRi	0.176	2.077	0.000	-0.041	0.000	0.247	0.230
AVRo	0.058	0.061	0.000	0.268	0.000	-.593	0.264
AVRc	0.166	1.653	0.000	-0.082	0.153	0.316	0.198
AVRi	0.195	1.736	0.000	-0.084	0.093	0.300	0.180

Where *MVR* or *AVR* does not depend upon that dimensionless variable the exponent regression coefficient is zero. The subscripts *o, c, and i* denote the velocity ratio for the outside bank of the bend, the channel centerline, and the inner bank respectively. Equation 12.9 applies only for the range of dimensionless variables tested in the trapezoidal physical model (Table 12–6). For bends with R_c/T_w less than 3 the value of A^* should not be greater than 19 percent (Scurlock et al., 2012b), unless the value of MVR_o is greater than 0.4 and/or AVR_o is greater than 0.2.

Table 12–6. Range of Dimensionless Variables to Use in Equation 12.9

A^*		L_{ARC}/T_W		R_C/T_W		L_{WPROJ}/T_W		**2θ/π (radians) (60 to 90°)**	
Max	*Min*	*Max*	*Min*	*Max*	*Min*	*Max*	*Min*	*Max*	*Min*
27.00	10.75	3.09	0.55	6.86	2.48	0.37	0.14	1.00	0.67

Equation 12.9 applies to spur dikes that have a bankline elevation equal to the design water surface elevation, or bank-full flow. These equations do not apply for cases in which spur dike structures along the bankline are deeply submerged, such as where the bankline vane elevation corresponds to the lower limit suitable for the establishment of woody riparian vegetation (NRCS, 2007; McCullah and Gray, 2005) or to one-half or one-third of the bank-full flow depth (NRCS, 2007).

Sensitivity analysis (Figure 12–8) was completed for each of the dimensionless variables in Equation 12.9 and the regression coefficients in Table 12–5. The purpose of this analysis was to show how each dimensionless variable affects *MVR* or *AVR* through and past laboratory ranges. Table 12–7 shows the median value of each dimensionless variable and the upper and lower limit of the sensitivity analysis.

The sensitivity analysis consisted of successively changing one dimensionless variable at a time while holding all other dimensionless variables constant as possible. It was not always possible to keep each dimensionless variable constant while sequentially changing because of inter-dependence between variables. For example, in the case where R_c/T_w is varied by sequentially changing T_w, $L_{w\text{-}proj}$ remained the same, and the dimensionless ratio $L_{w\text{-}proj}/T_w$ is also sequentially changed. When the velocity ratio is greater than 1 for *MVRo* the design results in higher bankline velocity than the average velocity without the transverse features. The lower and upper bounds of the laboratory range of data are designated by the vertical red lines in Figure 12–8. The *MVRo* as a result of changes in $L_{W\text{-}PROJ}/T_W$ over the range of laboratory testing shows a large rate of change and should be greater than about 0.13 to provide velocity reduction. R_c/T_W greater than 2.5 for suitable velocity reduction.

Sensitivity was completed using R_c = 180 ft, T_w = 790 ft, and D_b = 9.55 and average depth of 9.55 ft to calculate the dimensionless ratio values and sequentially alter each ratio (Table 12–57). Some of the dimensionless ratios exceed unity throughout a large portion of the laboratory testing ranges. These graphs are intended to show how MVR/AVR changes inside and outside the laboratory parameters and not to present values suitable for design. There was no laboratory test in which median values were used for all dimensionless variables.

Design procedures were developed using the statistical regression equation from Scurlock et al. (2012a), and Scurlock et al. (2015). Use of this design method requires the bank material and bed material median (D_{50}) size and a classification of the bank material sufficient to determine the permissible velocity (see Tables 4–2 and 4–3). Step 4 is the initial estimate of length and horizontal orientation angle, spacing, and vertical angle respectively. Several iterations may be necessary to determine the optimal length, spacing, and orientation angle, so that all dimensionless variables are within the permissible velocity.

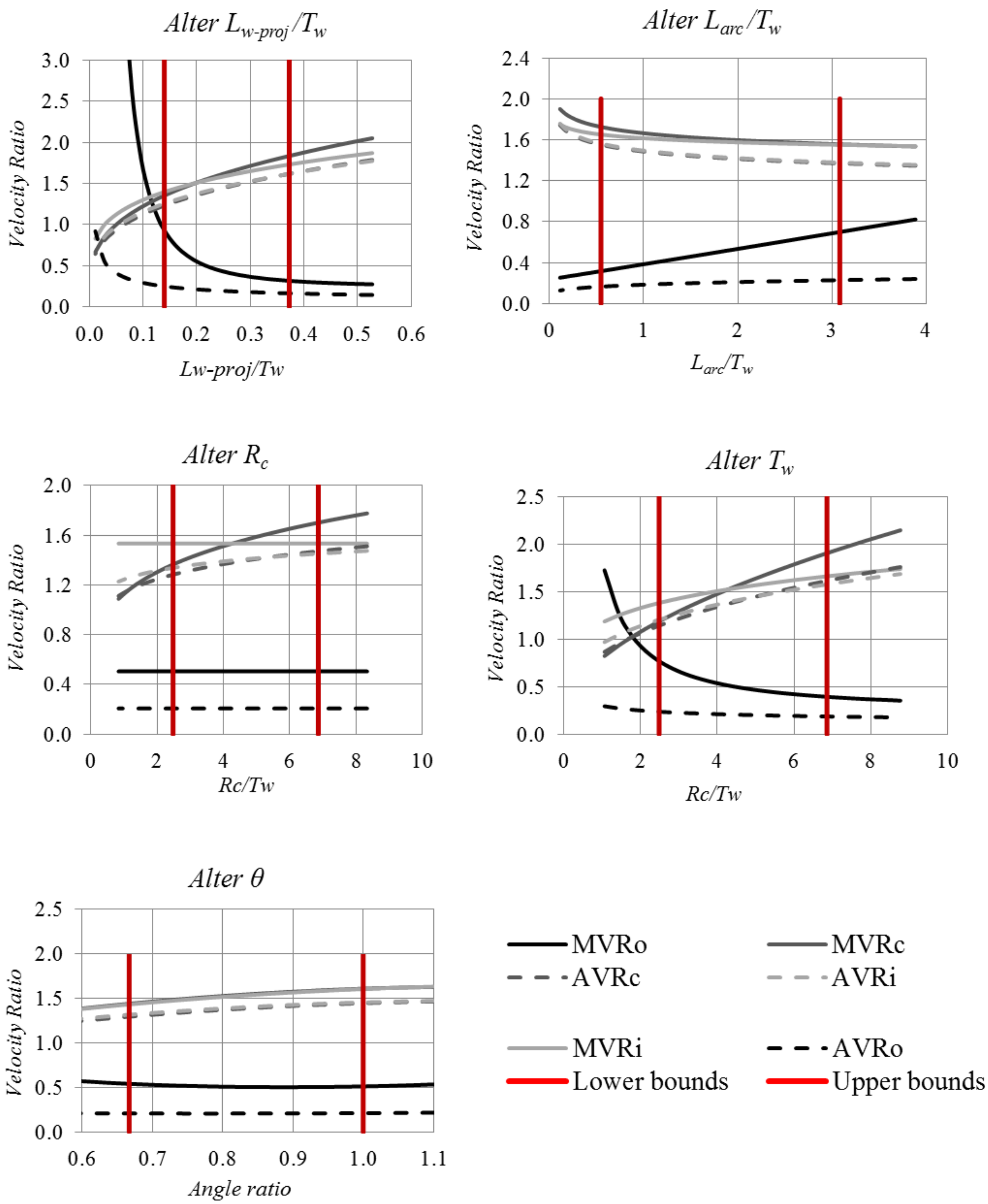


Figure 12–8. Spur-dike velocity equation parameter response.

Table 12–7. Range of Laboratory and Sensitivity Dimensionless Variable Testing

Dimensionless Variable	Upper Limit of Laboratory Testing	Lower Limit of Laboratory Testing	Median of Laboratory Testing	Median Value Used in Sensitivity Analysis median	Upper Limit of Sensitivity Testing	Lower Limit of Sensitivity Testing
A^*	27	10.75	18.88	13.53	29	2.3
L_{arc}/T_w	3.08	0.55	1.82	1.82	4	0
R_c/T_w	6.86	2.48	4.67	4.67	10	1.5
L_{wproj}/T	0.37	0.14	0.28	0.26	0.56	0.04
$2\theta/\pi$ *(Radians)*	1	0.67	0.83	0.83	1.1 (99^o)	0.06 (51^o)

Step 1: Determine design flows and hydraulic conditions

The hydraulic conditions during the mean annual flow, the annual low flow, the mean annual high flow level, and the flow above which significant riparian vegetation grows should be computed. The hydraulic numerical model should be developed using methods described in Section 4.4 to determine the flow hydraulics. Determine the design top width (T_w) from the upstream approach channel using the hydraulic model results for the mean annual peak flow, or Q_2.

Step 2: Determine desired bank line

The desired flow alignment may be based on preventing further erosion of the outside bankline, or reversing erosion of the bank to alter the flow alignment. The curve representing the desired flow alignment should form a smooth upstream and downstream transition. Draw the desired thalweg and bankline locations according to the procedure in the bendway weir section.

Step 3: Determine spur dike elevation

Spur dikes are usually constructed at an elevation equal to the bank height, regardless of whether the design flow is higher or lower than that elevation. If the design water surface elevation is higher, constructing the spurs at the bank height will prevent overtopping flows from being diverted, which could cause flanking (Brown, 1985). If the bank height is such that constructing spurs to that elevation is expensive then the elevation of the mean annual flow , or Q_2, could be used provided that a transition to the bank elevation is included so that overtopping the spur dike near the bank will not occur (Lagasse et al, 2009).

The first approximation of spur dike length, spacing, angle, and slope and spacing should be accomplished using the procedure used for the geometric design procedure. Spur dike geometry may need to be adjusted in design Step 5, so that the maximum permissible velocity for protection of the bank material is not exceeded.

Step 4: First estimate of spur dike length, angle, spacing and slope

The first approximation of spur dike length, spacing, angle, and slope and spacing should be accomplished using the procedure used for the geometric design procedure. Spur dike geometry may need to be adjusted in design Step 5, so that the maximum permissible velocity for protection of the bank material is not exceeded.

Step 5: Determine permissible velocity, channel centerline radius

Using the bank material and bed material median size (D_{50}) and a classification of the bank material, determine the permissible velocity (Tables 4–2 and 4–3). For cases where there are differing values based upon classification and particle size the classification's permissible velocity is used, because the values based on particle size are for uniform sized non-cohesive sediments which have lower maximum permissible velocities than more heterogeneous sediments. For the main channel bed material particle size provides the best representation of permissible velocity (Fishenich, 2001).

Determine bend centerline radius. This is the radius of a circular arc which best approximates the bend's curvature between the upstream and downstream bends or inflection points. The centerline is located using ½ the width of the approach channel (T_w) along the bend axis.

Step 6: Initial calculation of A^* (projected percent of the flow cross sectional area occupied by the vanes

Calculate the projected cross sectional area of the spur initially by assuming a flat bottom at the maximum flow depth and the average bank angle on the outside of the bend. Note that a more refined estimate can be made later based upon the cross section geometry nearest each spur dike or upon a distance-weighted cross section geometry.

From the spur tip to the channel bed the area is determined by the angle of repose of the riprap material. 1V to 1.5H or 1:V to 1.2H is generally used for this calculation. The projected spur length is used.

The percent of the flow cross sectional area occupied by the spur dikes is the projected spur dike cross sectional area divided by the baseline average bend cross sectional area (Step 1), multiplied by 100 for.

The projected area of the dike will likely need to be adjusted in design Step 6 so that it does not cause the flow to exceed the maximum permissible velocity for protection of the bank material.

Step 7: Initial and iterative calculation of Velocity Reduction

The primary design variable from which to estimate the stability of the eroding outside bank of the bend is $MVR_{o.}$, while AVR_o shows the bend average. Secondary design variables are the velocity ratios for the centerline and inner bank. These can be used as indicators of the erodibility of the bed and inner bank. Use Equation 12.9 with the appropriate regression coefficients and input variables to calculate $MVR_{o.}$ The bankline velocity predicted by Equation 12.9 is $V_{bank} = MVR_o \times V_{baseline}$. A non-conservative approach would be to use the AVR_o relationship $V_{bank} = AVR_o \times V_{baseline}$, because this will show less velocity reduction.

If V_{bank} is greater than the permissible velocity for protection of the bank material, then the $L_{W\text{-}PROJ}$ should be incrementally increased and L_{ARC} incrementally decreased until . V_{bank} equals or is less than the permissible velocity. For bends with R_c/T_w less than 3, the value of A^* should not be greater than 19 percent (Scurlock et al. 2012b), unless the value of MVR_o is greater than 0.4 or AVR_o is greater than 0.2.

Step 8: Determine final spur dike length and spacing

Using the existing cross section and spur dike locations throughout the bend, interpolate between cross sections if needed to determine the actual percentage of the projected cross-sectional spur area to average cross-sectional area of the baseline bend. All spur dike areas should be averaged together since Equation 12.9 is based upon bend average values with a coefficient to account for measured data within two standard deviations of the central tendency line predicted by the regression equation. Equation 12.9 represents the maximum bend velocity reduction.

Step 9: Determine the final centerline and inside of bend velocity changes

Compute $MVR/AVR_{c,}$, MVR/AVR_i and compare with the permissible velocities for protection of the bed material and the inner bank material. Centerline and inner bank velocity should increase following construction because some of the cross sectional area will be blocked by the spur dike structures, there will be added roughness, and the outer bankline velocity will be reduced. Centerline and inner bank velocities when compared to permissible velocities will give an indication of how stable the bed and inner bank are.

Step 10: Determine spur dike key length, angle, riprap size, top width, scour, riprap volume

The procedures for the spur dike key length, riprap size, top width, scour, and launch riprap volume are identical to the geometric design procedure for spur dikes.

Step 11: <u>Review constructability</u>

The constructability issues are identical to those of the geometric design procedure.

12.3.3 Discussion and Recommendations

Risk and Failure

The most common causes of spur failure are undermining due to scour at the tips and flanking by the river (bank erosion around and behind the structure). Providing scour protection in the form of self-launching riprap and/or a wider crest width reduces the potential for an undermining failure. Keying the structure into the bank can provide protection from outflanking (McCullah and Gray, 2005).

A secondary cause of spur failure is the entrance angle changing due to upstream channel migration, resulting in a larger flow entrance angle with concurrent increases in velocity and scour (Chester Watson, 2006, personal communication, Professor, Department of Civil and Environmental Engineering, Colorado State University). A countermeasure for this situation can be adding more launchable rock that is needed to protect for spur dike tip scour as a safety measure.

Spur dikes have a greater risk of riprap erosion than other transverse structures due to hydrodynamic forces on spur tips, and scour generated by the spurs. Countermeasures for this risk include placement of launchable riprap and active monitoring and maintenance. There should be some distance between the protected bankline and riverside infrastructure to allow for bank scalloping.

Monitoring and Maintenance

Regular monitoring and maintenance are required for the spur dikes to remain functioning during their service life. Due to the hydrodynamic forces on the dike tips and scour generated by spur dikes, riprap erosion is common. The amount of riprap erosion should be monitored to ensure that it does not exceed the amount accounted for in the design.

12.4 J-Hooks

J-hooks are discontinuous, transverse structures angled into the flow that redirect the flow from eroding banks. Essentially, J-hooks are vanes with a tip placed in a downstream pointing “J” configuration (Figure 12–9). The “J” tip is partially embedded in the riverbed, so it is submerged during low flows. Vanes were discussed previously, so this section just covers the J-hook portion. Some items from the vane section are repeated here, if they relate specifically to the J-hook portion.

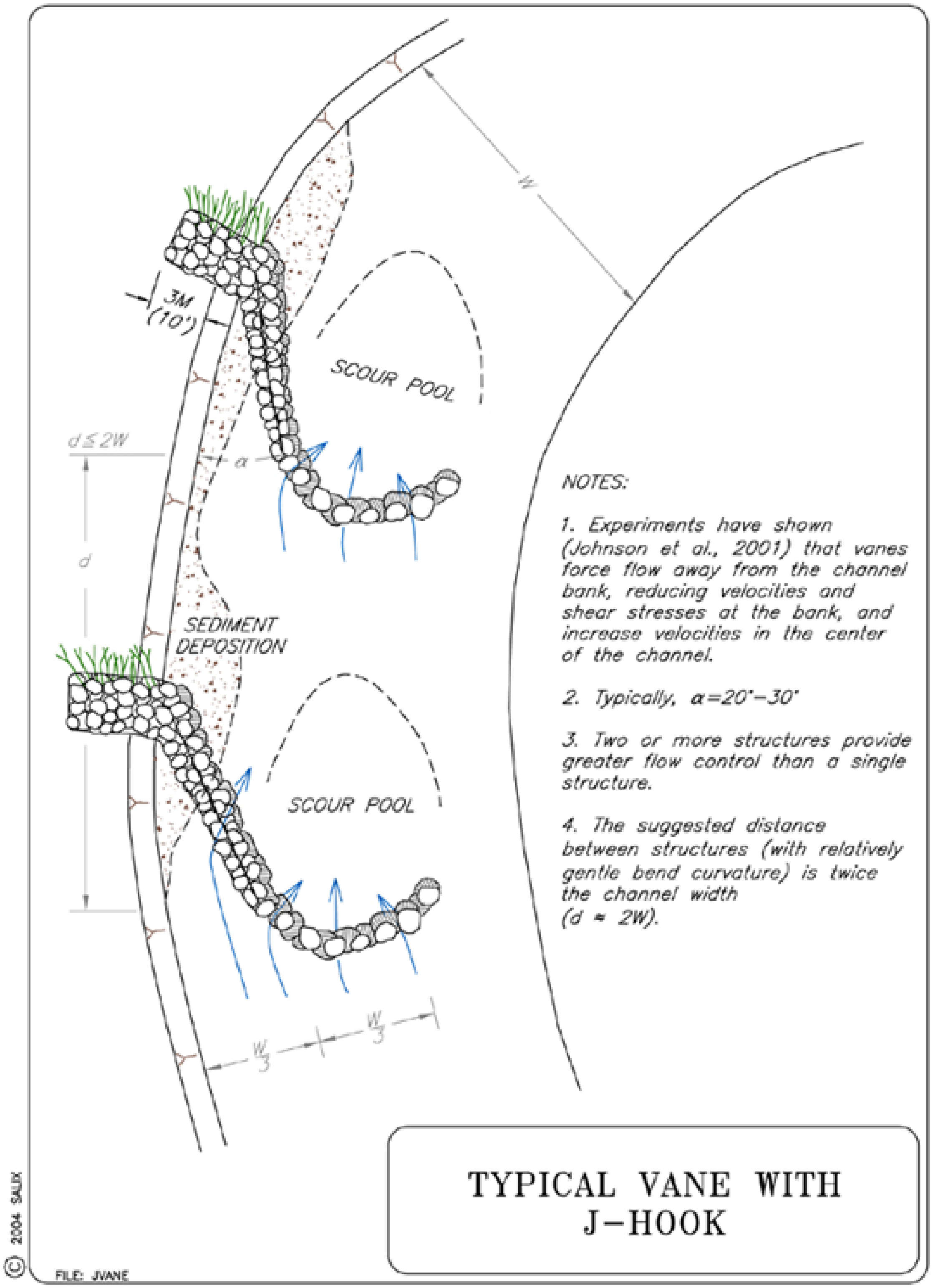


Figure 12–9. Vane with J-hooks typical drawing. Used by permission of the Transportation Research Board (report No. 544) and Salix Applied Earthcare (McCullah and Gray, 2005).

The crest slopes upward from near the riverbed elevation to generally the bank-full stage elevation at the bankline. The “J” tip is intended to produce additional in-stream habitat by creating a scour pool downstream from the “J” tip, especially in gravel to cobble substrates (McCullah and Gray, 2005) and a riffle below the pool. A scour hole is created which can provide holding pools and refuge cover during high and low flows. Interface (shear) zones are created between fast- and slow-moving flow, and spawning habitat forms via gravel sorting in the downstream portion of the pool and created riffle. These effects can improve benthic habitat, create or maintain pool and riffle habitat, and improve fish rearing and adult holding habitat.

The vane portion of the J-hook is angled upstream to redirect overtopping flows away from the protected bank. Some sediment deposition may occur upstream and downstream from the structures resulting from the redirected flows. In situations where sediment deposition occurs between the structures, additional bank protection can form over time. The added feature is that the “J” creates an additional scour hole and can produce a riffle downstream (McCullah and Gray, 2005). The J-hook scour hole is larger, wider, and deeper than one created by vane structures alone.

The application of J-hook vanes is very similar to the application of vanes.

12.4.1 Design Procedures

Design considerations, including spacing, length, height, orientation and vertical angles for geometric and velocity reduction based design for the vane portion of the J-hook are as described in previous section. The following sections only cover the design considerations and procedures for the “J-hook” portion of the vane. Key design variables for J-hooks, are rock size and type, footers, alignment, and installation elevation.

Step 1: J-hook location

Position the J-hook to be in the center third of the channel (Figure 12–9). There is no guidance available on the shape of the J-hook, but a visual location similar to Figure 12–9 is advisable. Conceptually the shape of the J-hook should be a smooth arc that is smaller than a semi-circle. If the shape of the J-hook is close to a semi-circle the scour would be greater than the length of arc shown in Figure 12–9.

Stones forming the J-hook are partially embedded in the streambed so they are submerged even during low flows. Stones are positioned to form a smooth arc in the downstream direction.

Step 2: Rock Sizing and Placement

J-hook stones should be sized using individual stone force balance with stones embedded (Section 10.43).

J-hook footer stones should be long and flat with the longest axis being at least 3.5 times the shortest axis (McCullah and Gray, 2005). Stones forming the J-hook crest should be large enough to reach the desired height when buried in the streambed, and to resist movement during design flow events. If these two criteria indicate stones of different sizes, the larger of the two sizes should be used.

Footer stones should be heavier, longer and flatter than the average vane stones. As a rule of thumb, the heaviest footer stone should be comparable to the D_{100} or larger for the vane (Maryland, 2000). In fine gravel or sandy streams an extra layer of footer stones may be necessary to accommodate the additional scour. Even in small streams it is not uncommon to have 6 ft of scour in J-hooks (McCullah and Gray, 2005).

The excavation for placement of the J-hook should be about twice the footer rock thickness (McCullah and Gray, 2005). Conceptually, no more than about 10-15% of the header rock should be above the existing channel grade. Thus, 85-90% of the vane rock (including the footer stone) would be embedded.

Step 3: Estimate Scour

Scour depth and launch riprap volume for the vane portion of J-hooks will be determined using the procedures for bendway weirs. The “Z” value for vanes is the same as for bendway weirs (0.5). For the J-hook stones the scour depth is usually estimated to be about the same as the thickness of the footer stones for gravel bed channels. Local experience becomes important for determining if one or two footer stones should be used.

Step 4: Review Constructability

J-hook rocks should be placed on top of footer rocks such that each vane rock touches adjacent rocks and rests upon two halves of each footer rock below it. The vane J-hook header rocks should be placed upstream of the footer rock to form a step which is about 1/3rd as wide as the top dimension of the footer rock (Maryland, 2000).

The scour hole may or may not be partially excavated. If immediate fish habitat improvements are desired, then excavation can be completed during construction.

No guidelines are given for the excavation, but some of the footer rock should remain embedded: at least ½ of the footer rock thickness is conceptually recommended.

12.4.2 Discussion and Recommendations

Risk and Failure

The most common causes of J-hook failure is undermining due to scour. Providing scour protection in the form of footer rocks is necessary to prevent the downstream scour hole from undermining the rock forming the "J" tip. Undersized stones can be eroded during high flows and can also contribute to failure. There is a high likelihood of bankline scalloping between structures and structure erosion during extreme flow events. Therefore, vanes with J-hooks should not be used near high value river side infrastructure or where there is a potential for loss of life.

13 Hardened Banks

Banks can be hardened with a variety of materials including concrete walls or slabs, grouted riprap, wire enclosed riprap (wire placed on the bank slope beneath and on top of the rock), soil cement, riprap, concrete armor units, broken concrete, wood cribs, log cribs, tires, car bodies, cabled broken concrete, and gabions (wire baskets filled with rock). The preferred material is riprap due to the articulation of individual particles that allows some shifting and adjustment, and presumably the less negative environmental impact than some of the other options. Riprap is a layer of large rock placed on the banks of a channel that prevents the softer earth beneath from being eroded or undercut. Eroding/scouring high-flow events can occur many times during the life of a structure. Riprap revetments can be used for other applications including road embankments and for ocean wave protection as presented here is specific to riprap applications on river or stream banks.

A full-bank riprap revetment should generally be considered only after all other options are exhausted. A riprap revetment eliminates the highly diverse habitat of riparian vegetation and reduces river function through elimination of natural river processes and loss of access to the floodplain. A reduction in river function reduces environmental benefits, limits sustainability and can require maintenance/renovation whenever a natural storm event exceeds the design event. This occurs on average more than once per design life if the design life was used to determine the hydrology (FHWA, 2012). Using a larger storm event to size riprap for high energy systems can push the design outside the limits of obtainable and cost-effective riprap. Alternative methods described in previous chapters should be reviewed before settling for riprap revetments, a solution that is less than optimal in all three measures of river engineering success: feasibility, sustainability and environmental effectiveness.

Methods presented here include riprap blanket revetments or stone toes, riprap windrow, and trench filled riprap. Riprap revetment includes stones placed from the toe up the bank. Stone can be placed only in the toe, leaving the upper bank undisturbed, or in other cases the bank above the stone toe can be treated with vegetation. In these cases, design of the stone toe is similar to design of the riprap revetment. Riprap windrow consists of stone piled above potential erosion sites in anticipation that the river will complete the work of placement by eroding to the base of the windrow. Erosion at the toe of the stockpile causes the stone riprap to launch down the channel bank, armoring the slope. Trench filled riprap is similar to riprap windrow except that the stones are placed in an excavated trench in the pre-determined location and alignment. Longitudinal peak stone toe is a windrow placed on the stream bed forming a new fixed channel bankline.

Bank protection methods apply to cases where erosion of the toe, mid and upper bank is the primary mechanism for bank failure. This includes small bank slope failures or slump block failures. Bank protection works may be needed to protect against bank slips and to reduce the hydraulic load acting on the soil (Hey, 1994; Brookes, 1988; Escarameia, 1998; McCullah and Gray, 2005). In situations where the bank slope is unstable due to geotechnical processes, the designer should consult other sources for direction on geotechnical designs.

13.1 Upstream and Downstream Limits of the Work

The upstream and downstream limits of bank protection depend on meander migration (in the case of a meandering river), length of observed eroded streambank and/or potential length of streambank which could be subjected to damage or erosion. In the case of a braided river, the length of streambank showing signs of erosion, and potential areas that may be subject to erosion, need to be protected. Other considerations involve keeping riverbank stable in an area where an eroding bank can jeopardize the safety of infrastructure.

Longitudinal bank protection for a meandering river form is usually placed where erosion is occurring along the concave bank. The upstream limit of the work is generally the upstream crossing on point of tangency, or slightly downstream. Model studies (USACE, 1981) indicate erosion protection in a bend should extend downstream a distance of at least 1.5 times the width of the approach channel upstream of the bend (Figure 13–1). The minimum extent of bank protection determined from Figure 13–1 should be adjusted according to field inspections to determine the limits of active scour, channel surveys at low flow, aerial photography, geomorphic analysis, and field investigations at high flow.

Lagasse et al. (2009) states “Investigators of field installations of bank protection have found that protection commonly extends farther upstream than necessary and not far enough downstream. However, such protection may have been necessary at the time of installation. The lack of a sufficient length of protection downstream is generally more serious, and the downstream movement of meander bends should be considered in establishing the downstream extent of protection.”

The stone toe should be placed from the upstream to the downstream crossing if there is a likelihood of upstream bend migration occurring, and/or there is high value infrastructure being protected. However stream process is lost when the bend is locked into position against bend migration so alternative means of bank stabilization should be explored before settling on hardening banks, specifically where there is a long extent of bank coverage required.

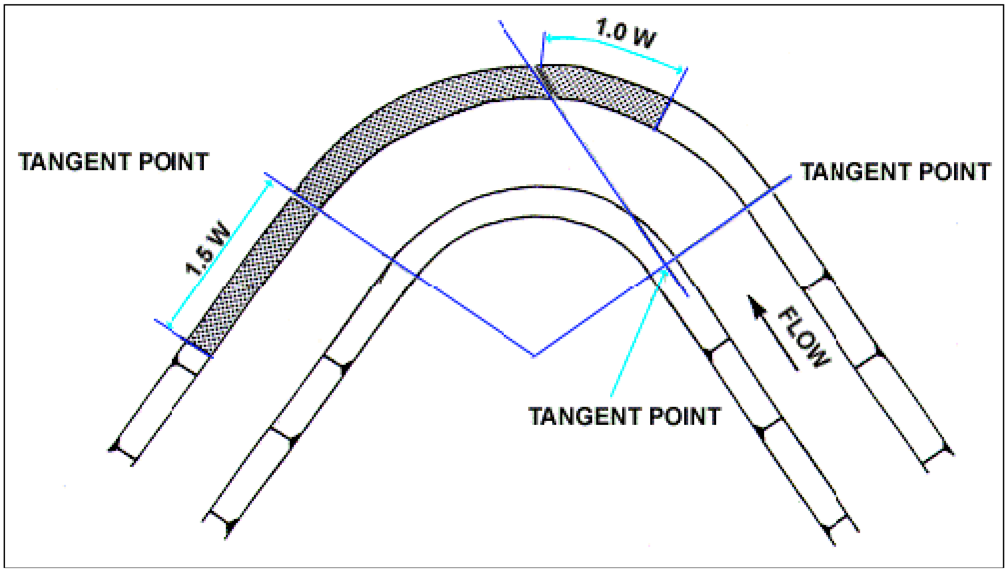


Figure 13–1. Extent of protection required at a channel bend (after Lagasse et al. 1991). The hatched area is the area of hardened bank protection.

13.2 Sizing Riprap

The basis of designing a riprap revetment is sizing the rock. Methods presented here are applicable to all bank hardening methods presented in succeeding chapters. There are many methods available and this presentation is not all-inclusive, however a sensitivity analysis has been provided on the presented methods to aid in selecting an appropriate riprap sizing equation for the site. The recommended approach is to use a minimum of three methods to define the range in values. Selection of the riprap size could be based on an average value from the range, or it may be a high or low value depending on site specific characteristics such as the geomorphic factors. There are spreadsheets and software available for computing riprap size, but the designer should be familiar with the individual riprap sizing methods to ensure they are applied correctly.

Riprap sizing methods evolved from non-scouring, non-silting maximum permissible velocity concepts (permissible velocity is the critical velocity at which channel erosion begins). These concepts were used for early canal designs that were based upon experience and observation of many canals in different bed and bank materials. Isbash (1936) presents an equation for mean velocity against stone, which became popular for some riprap design because of lack of other significant data.

$$V = C[2g(S-1)]^{1/2}(d_{50})^{1/2} \tag{13.1}$$

Where,

V = Mean velocity against stone, ft/s or m/s
C = Isbash constant (0.86 or 1.20 – see discussion)
g = Acceleration of gravity, ft/s^2 or m/s^2
S = Specific gravity of stone ($S = \frac{\gamma_s}{\gamma_w}$)
d_{50} = Median diameter of spherical stone, ft or m

The lower value for the Isbash constant represents the flow velocity at which loose surface stones first begin to roll. The higher value represents the flow velocity at which stones protected by adjacent particles begin to move and roll until they find another “seat”.

In 1949, California Division of Highways appointed a Joint Bank Protection Committee to study “primarily the special treatment of banks of streams, lakes or tidewater and secondarily the treatment of highway embankments to prevent erosion by surface waters….” The outcome of this study was a compilation of data and reports that became the first edition of the California Bank and Shore Protection Manual published in November 1960. An equation and nomograph were developed for slopes no steeper than 1.5H:1V:

$$W = \frac{0.00002V^6 S}{(S-1)^3 sin^3(\rho-\alpha)} \tag{13.2}$$

Where:

W = Minimum weight of outside stone for no damage, lb
V = Stream velocity to which bank is exposed, ft/s
= 4/3 the average stream velocity for impinging velocities (on outside of bends in line with the central thread), ft/s
= 2/3 the average velocity for tangent (parallel) velocity, ft/s
s = Specific gravity of the stones ($S = \frac{\gamma_s}{\gamma_w}$)
ρ = 70° for randomly placed rock
α = Embankment face slope (ft/ft)

Stevens (1968) developed a stability factor approach for riprap at culvert outlets based on shear stress that has been adapted to revetments. He considered the forces acting on a particle in the plan of the side slope. The equations given below are for horizontal or parallel flow on an embankment. The expression for the stability factor, SF, for horizontal flow on a side slope with an angle of θ and using rock with an angle of repose of ϕ is

$$S.F. = 0.5S_m\left\{\sqrt{\zeta^2+4}-\zeta\right\} \tag{13.3}$$

Where

$$\xi = S_m \eta sec\theta \tag{13.4}$$

$$S_m = \frac{tan\phi}{tan\theta} \tag{13.5}$$

Solving for the stability number, η, in terms of the stability factor gives

$$\eta = \frac{S_m^2 - S.F.^2}{S.F.\ S_m^2} cos\theta \tag{13.6}$$

Where

S.F. = Stability factor
Θ = an angle for horizontal flow on a side slope
Φ = angle of repose for rock

If the shear stress on the slope, τ_0, is known, the riprap size, d_m can be obtained from

$$d_m = \frac{21\tau_0}{(Sg-1)\gamma\eta} \tag{13.7}$$

Where, d_m is in ft, τ_0 is in lb/ft^2, and γ is in lb/ft^3

The revised HEC-11 (Brown and Clyde, 1989) revetment riprap equation is derived based on the Shields equation for incipient motion, average shear stress ($\tau_0 = \gamma R S_f$), the Manning equation to compute friction slope, and the Strickler equation to compute Manning n as a function of particle size. Additional factors are included for bank angle, riprap specific gravity, and desired stability factor. The equation in English units is

$$d_{50} = 0.001 C_{sg} C_{sf} \frac{V_a^3}{d_{avg}^{0.5} K_1^{1.5}} \tag{13.8}$$

Where

d_{50} = Median diameter of stone, ft
V_a = Average channel velocity, ft/s
d_{avg} = Average channel depth, ft
C_{sg} = $2.12/(S_g - 1)^{1.5}$
S_g = Riprap specific gravity ($Sg_r = \frac{\gamma_s}{\gamma_w}$)
C_{sf} = (Stability factor/1.2)$^{1.5}$
K_1 = $[1 - sin^2\theta/sin^2\phi]^{0.5}$
θ = Bank angle (degrees)
ϕ = Riprap angle of repose (degrees)

For metric units, the constant in Equation 13.8 needs to be 0.00594 (0.001/ 0.3048$^{1.5}$).

An initial equation (Maynord et at., 1989) based on velocity and using dimensional analysis for finding riprap rock size, d_{30}, instead of the commonly used d_{50} was modified to include coefficients to account for stability, velocity distribution, blanket thickness, and side slope correction. Equation 13.9 is the final equation. Values of coefficients are given graphically in Appendix B of EM 1601 (USACE, 1991):

$$\frac{d_{30}}{y} = S_f C_S C_V C_T \left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w} \right)^{0.5} \frac{V_{ss}}{\sqrt{K_1 g y}} \right]^{2.5} \tag{13.9}$$

Where

d_{30} = Particle size for which 30% is finer by weight, ft or m
y = Depth of flow above particle, ft or m
S_f = Safety factor
γ_w = Specific weight of water, lb/ft^3 or kg/m^3
γ_s = Specific weight of particle, lb/ft^3 or kg/m^3
V_{ss} = Characteristic velocity, depth-averaged velocity at point 20% upslope from toe
$V_{ss} = V_{avg}[1.74 - 0.52 Log(R_C/W)]$ for natural channels (Figure 13-2)
g = Acceleration due to gravity, ft/s^2 or m/s^2
C_S = Stability coefficient
= 0.3 for angular rock; 0.375 for rounded rock

$$\begin{bmatrix} for\ blanket\ thickness = \\ 1\ d_{100}(max)\ or\ 1.57 d_{50}(\text{max}) \\ whichever\ is\ greater, and\ \ d_{85}/d_{15} = 1.7\ to\ 5.2 \end{bmatrix}$$

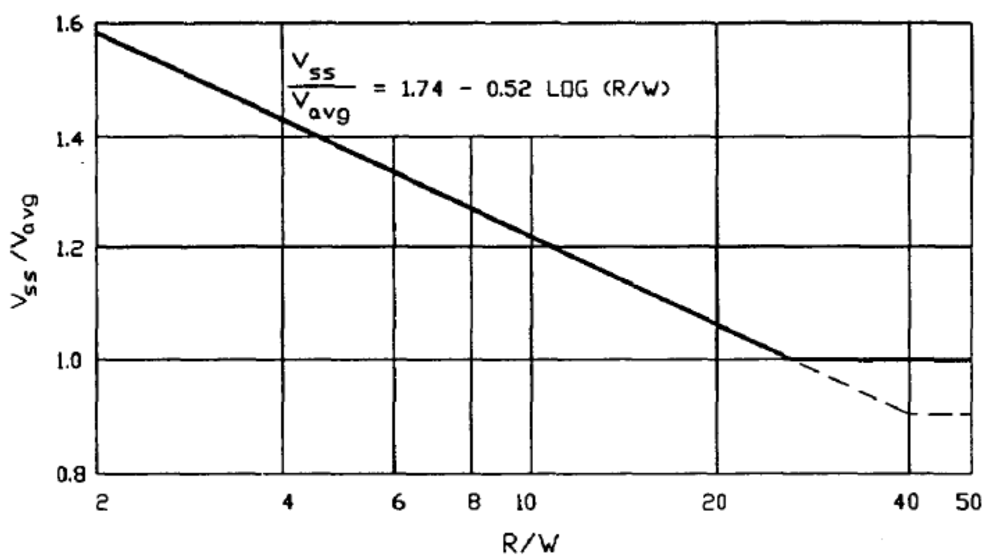


Figure 13–2. Riprap design velocities for a natural channel, from USACE (1991, plate B-33). V_{ss} is depth-averaged velocity at 20 percent of slope length from toe.

C_V = Velocity distribution coefficient (Figure 13–3)
= 1.0 straight channels, inside of bends
= 1.283-0.2 $Log(R_C/W)$ for outside bends (1 for $R_C/W > 26$)
= 1.25 downstream from concrete channels
= 1.25 at end of dikes
R_C = Centerline radius of bend, ft on m
W = Water-surface width at upstream end of bend, ft or m
K_1 = Side slope correction factor (use Figure 13–4)
C_T = Blanket thickness coefficient, function of d_{85}/d_{15} (Figure 13–3)

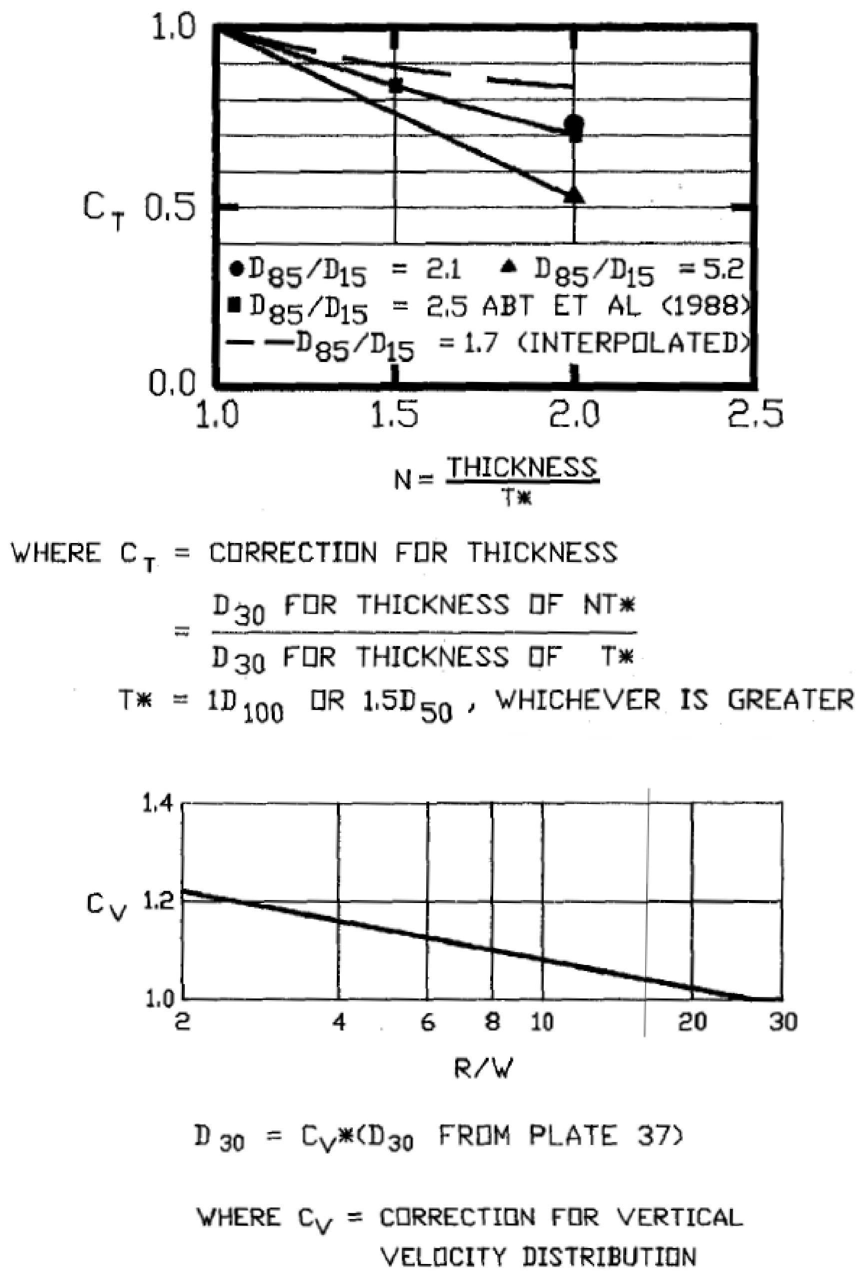


Figure 13–3. Correction for vertical velocity distribution in bend and riprap thickness, from USACE (1991, plate B-40).

An equivalent mathematical expression to Figure 13-4 for K_1 is:

$$K_1 = \mathrm{erf}\left(0.41 {S_s}^{1.443}\right) \qquad (13.9a)$$

Where 'erf' is the standard error function.

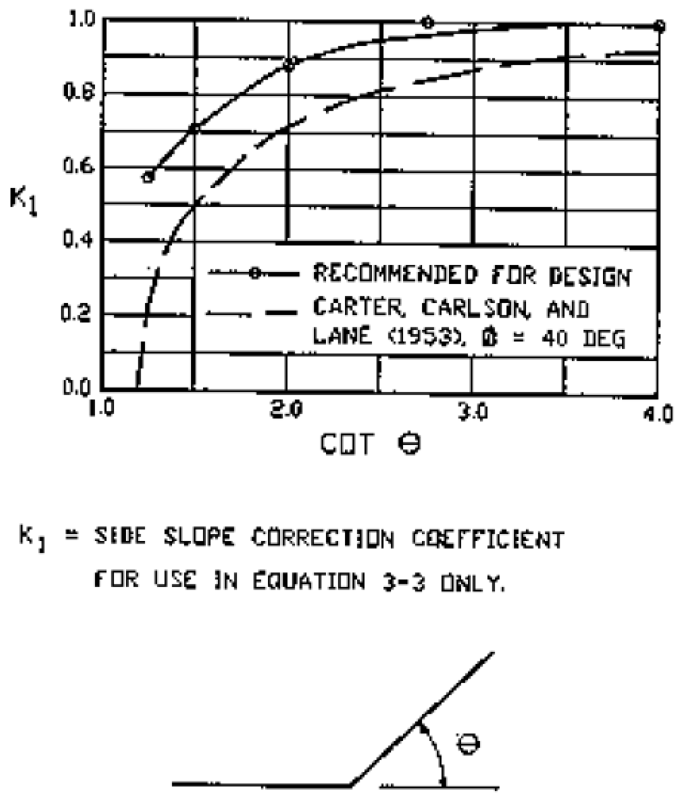


Figure 13–4. Correction for side-slope angle, from USACE (1991, plate B-39).

The Escarameia and May (1992) equation for sizing revetment riprap is

$$d_{n50} = C \frac{U_b^2}{2g(S-1)} \qquad (13.10)$$

Where:

d_{n50} = Characteristic size of stone, size of equivalent cube, ft or m
C = Coefficient that accounts for turbulence intensity, *TI*; for Riprap = 12.3*TI*-0.20
TI = Ratio of root mean square velocity fluctuation over mean velocity measured at a point 10% of flow depth above bed and varies from 0.12 to 0.60 for different structures
U_b = Mean velocity measured at a point 10% of flow depth above bed, ft/s or m/s
g = Acceleration due to gravity, ft/s^2 or m/s^2
S = Specific gravity of stone

In most cases of design, the *TI* is not known, is difficult to obtain, and must be assumed. For *TI* less than 0.5, a relationship between U_b and U_d, depth average velocity, was obtained from field measurements and can be used if values for U_b are not available:

$$U_b = (-1.48TI + 1.04)U_d \tag{13.11}$$

A provisional equation that has not been verified for *TI* greater than 0.50 is

$$U_b = (-1.48TI + 1.36)U_d \tag{13.12}$$

For straight channels, U_d can be substituted for U_b and values of *C* are 1.0 for continuous revetments and 1.25 for edges.

Pilarczyk's (1990) riprap equation is

$$d_{n50} = \frac{\Phi}{\Delta}\frac{0.035}{\Psi_{cr}} K_T K_h K_S^{-1} \frac{U_d^2}{2g} \tag{13.13}$$

Where

d_{n50} = Median diameter of stone, ft or m
Φ = 0.75 for continuous protection, and 1.0-1.5 at edges and transitions, and 3.0 for jet impact or screw race velocity
Δ = S-1
Ψ_{cr} = 0.032 for rock riprap
K_T = 1.0 for normal river turbulence, 1.5 - 2.0 for high turbulence (e.g., downstream of stilling basins, local disturbances, sharp outer bends)
K_h = $(d_{n50}/y)^{0.2}$ where *y* is depth of flow above toe of bank
K_s = Product of a side slop term and a longitudinal slope term
U_d = Depth average velocity, ft/s or m/s
g = Acceleration due to gravity, ft$/s^2$ or m$/s^2$

Sensitivity Analysis for Design Equations

The six equations discussed in this section appear to be more widely used for design than other equations found in the literature. The six equations are from HEC-11 (Brown and Clyde, 1989), Escarameia and May (1992), Pilarczyk (1990), EM 1601 (USACE, 1991) supplemented by Maynord et al. (1989) and Maynord (1990), Isbash (1935, 1936) and CABS (Racin et al., 2000).

Of the six equations considered, four include flow depth as a variable (HEC-11, Pilarczyk and EM 1601 [Maynord]). Although flow depth should be a factor for bank revetment, it should be a relatively small factor. In both the Pliarczyk and EM 1601 (Maynord) equations, riprap size is proportional to flow depth to the -0.25 power. Although not immediately evident in the standard presentation of Pilarczyk's equation, riprap size is proportional to velocity to the 2.5 power (like Maynord's [EM 1601] equation).

Each of the equations show an analysis for bank revetment on a 2H:1V side slope using angular riprap with a specific gravity of 2.65. Selection of stability factors, safety factors, and turbulence intensity was based on the individual equation guidance. The first plot (Figure 13–5) holds depth constant at 10 ft (3 m) and varies average velocity from 5 to 15 ft (1.5 to 4.5 m/s) for a bend R_c/W equal to 10.

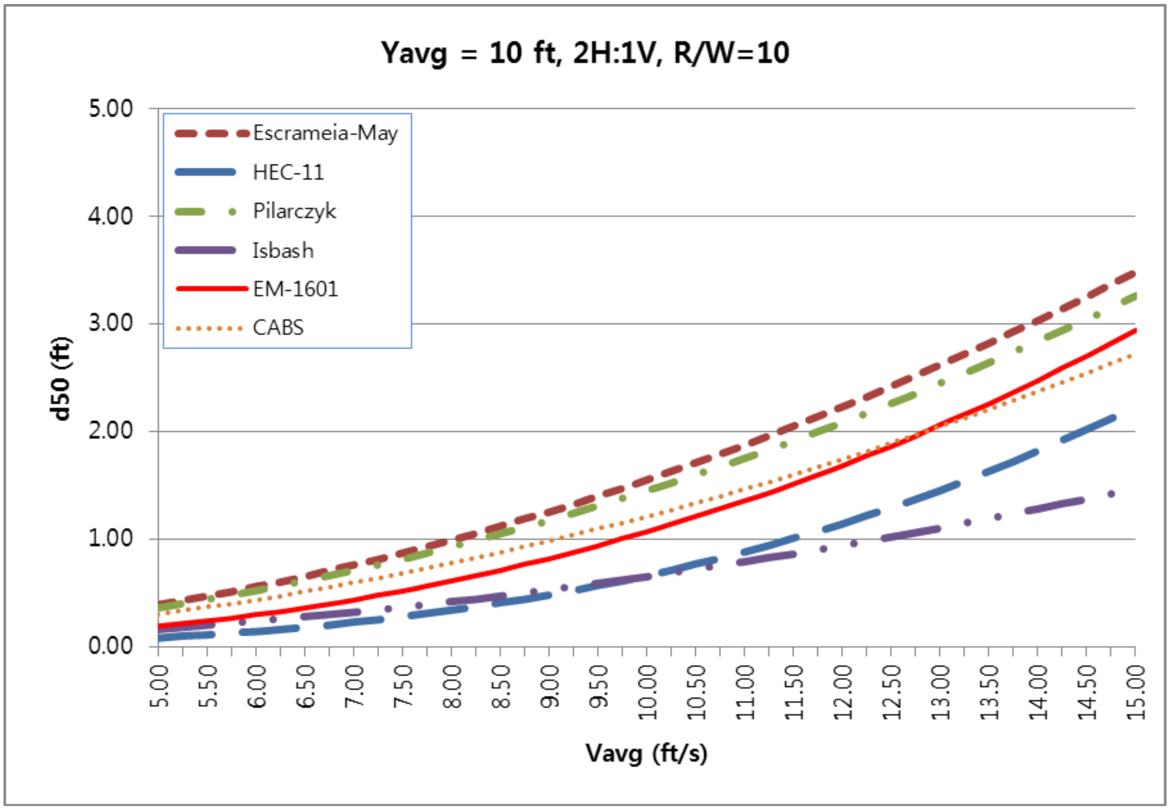


Figure 13–5. Riprap size versus velocity for a bend of R_c/W=10 bend.

The second plot (Figure 13–6) was developed for severe, curvature bend (R_c/W equal to 3). In Figure 13–6 the EM 1601 and Pilarczyk equations produced similar results. However, had R_c/W been different, slightly higher or lower, the EM 1601 equation would have changed while the Pilarczyk equation would not. This difference is because the EM 1601 equation computes a design velocity based on the average velocity and a function of R_c/W, whereas the Pilarczyk equation has a factor (K_T) that takes on a value of 1.5 for sharp outer bends.

13.3 Riprap Revetment

Riprap revetments are the most common form of a designed riprap installation, and provide continuous toe and bank protection against most all-bank erosion mechanisms (Figure 13–7). A revetment has stone protection extending from the toe up the bank. Depending on the design, a revetment can extend partially up the bank or fully to the top of bank. Sizing riprap was treated separately in the previous section. Bank shaping is usually required to provide a relatively uniform slope and alignment for the riprap placement. This calls for removal of any vegetation that was previously contributing to bank stability. Rock riprap is constructed in a layer that is several particles thick, and it adjusts to local displacement of material without complete failure of the installation. This aspect of rock riprap is often referred to as the articulating or “self-healing” characteristic.

13.3.1 Design Procedure

At this point, project design criteria, ecological factors, hydrology, geomorphic factors and general hydraulic and scour factors have been assembled previously to aid in selecting a suitable bank stabilization method. It is assumed that this information is available now as a resource to the design. Important aspects of design criteria, hydrology, and permitting are described in chapter 2. Determine the geomorphic factors including sediment and sediment continuity issues that may impact channel stability as discussed in chapter 3. Define general hydraulics including energy (chapter 4) and potential for scour (chapter 5) that may influence the channel and the extent of the floodplain. Steps of the design will recommend revisiting some of these investigations, to obtain more detailed information for this design.

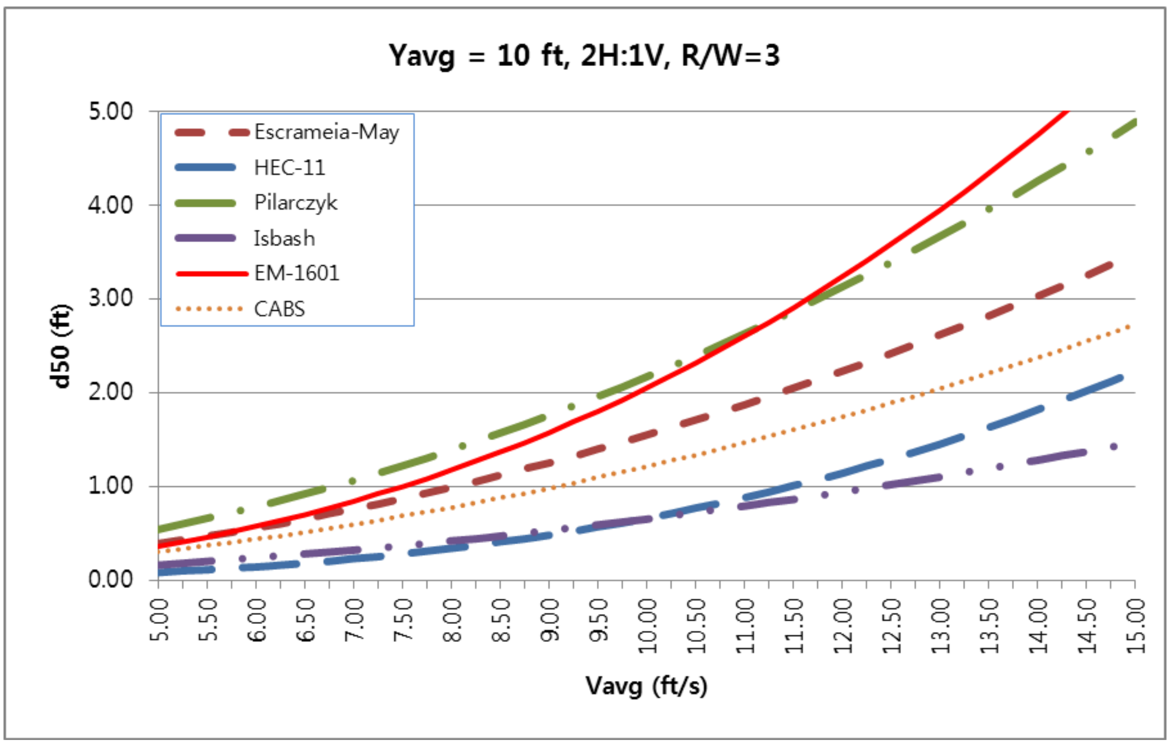


Figure 13–6. Riprap size versus velocity for a bend of R_c/W=3.

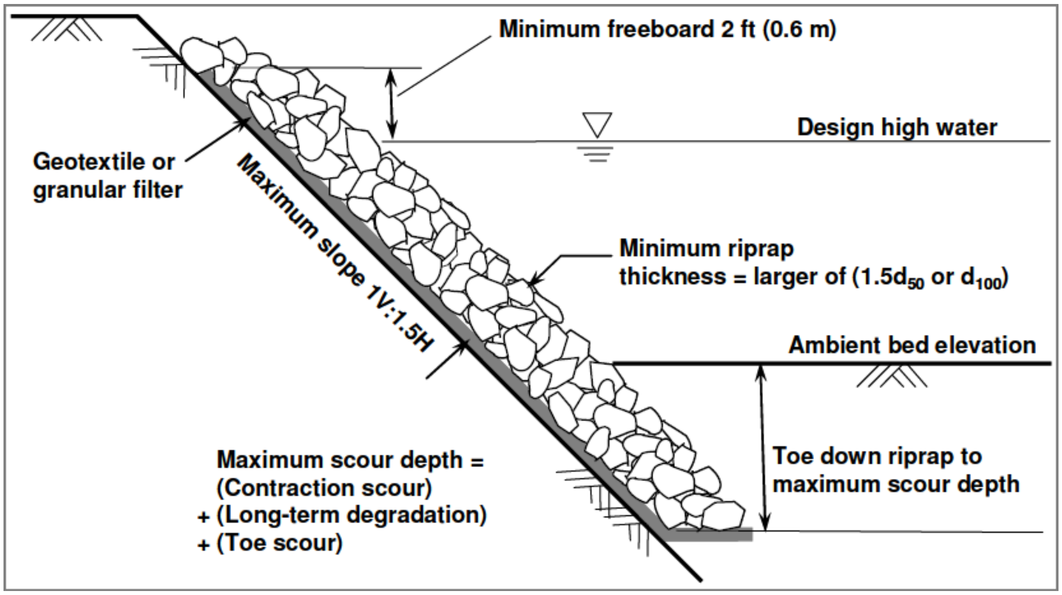


Figure 13–7. Riprap revetment with buried toe (after Lagasse et al. 2009).

Step 1. Determine flow hydraulics. The durability and design life of a riprap revetment depend on the return interval of the design flood selected. The return period flood for design depends upon the value of riverside facilities and infrastructure. When applicable, the return period design peak flow corresponds to the same level of the flood protection scheme itself (Escarameia 1998). Yet as presented in section 2.4, the design flood return interval should be larger than the design life of the project. If these values match, there is a 2/3 chance the design flow event will be exceeded during the life of the project, and presumably, significant maintenance will be required.

Selecting a suitable design flow event for sizing riprap is the conundrum of riprap design. Selection of a design flood often ranges from a 2- to 100-year return period discharge. A 25-year event provides for what is considered an economical design life and a reasonable design flood, but this design event will, on average, be exceeded at least once during the 25-yr interval based on the risk analysis discussed in section 2.4. If the structure has a 50-yr design life, the riprap will require significant maintenance, on average, three times. A larger flow event can be selected to reduce maintenance, but there is a limit on the feasible availability and acquisition of large rock. There is also a limit on how far the rock sizing equations can be accurately extrapolated. It is not uncommon for the rock size design, on high-energy rivers with steep banks, to exceed what can be acquired at a reasonable cost in the project area. Choices include paying more up front for larger rock, paying more for riprap maintenance and possibly flood damages, or investigate alternative methods of laying back banks and re-establishing floodplain.

Step 2. Determine the desired alignment. Revetments and longitudinal stone toe can be constructed along one bank from the upstream crossing to the downstream crossing. Cost savings can be realized when riprap revetments are constructed in the zone of highest shear stress, which is more towards the downstream part of the bend (Brown, 1985; USACE, 1981). These manuals recommend placing the riprap along the downstream portion of the meander bend only, instead of between the upstream and downstream bend inflection points (see section 5.7 for a complete description of the bank protection limits).

Step 3. Determine erosive forces and water surface elevations using hydraulic models. A 1D or 2D flow model (SRH-1D, HEC-RAS, SRH-2D), or other means, can be applied to estimate the velocity or shear force acting on the bank of the channel. Estimate flow depths, elevations, velocities, and secondary flow patterns that will have an erosive impact on the bankline. The river form and the energy level of the river (chapter 4) will also help in this analysis.

The top elevation of the revetment stones is usually the 2-year return period flow peak but can be larger for high banks near high value infrastructure. Two ft of freeboard can be added to the design water surface elevation on revetments protecting infrastructure, or for flood control elements including levees. The top of the stone toe will often be located at the elevation of the green line.

Riprap toes or revetment generally do not cause a rise in water surface elevation beyond the influence of a change in bank resistance, unless the channel cross sectional area changes or as a result of an expansion or contraction. Expansions and contractions of less than 10% generally do not have a significant effect upon the water surface elevation (Fischenich, 2000). This is especially the case when there are active channel degradation or incision processes underway. Flow velocity near riprap is faster than flow moving past vegetated banks due to the lower friction of riprap. Dense plantings or bioengineering have potential to increase water surface elevation due to increased flow resistance.

Step 4. Estimate scour and design the minimum toe elevation. Riprap revetment includes stones placed from the toe to the top of bank or to an elevation of a design flood, such as the 25-year event water surface elevation.

Riprap revetment should be toed down below the toe of the bank slope to a depth at least as great as the depth of anticipated long-term bed degradation plus toe scour (see chapter 5). Installations in the vicinity of bridges must also consider the potential for contraction scour.

If the channel bed is incising, the toe of the riprap must be installed below the maximum incision depth so it will not fail from undermining. It should also be below the deepest scour depth. The main countermeasure for toe scour is to construct a thickened toe section with an adequate volume of riprap to launch into scour hole while preventing the rest of the revetment to dislodge. This countermeasure can be constructed in flowing or ponded water (figure 13–8, figure 13–9). A second countermeasure is to construct the revetment to the elevation of the maximum estimated scour plus a safety factor by excavating the channel bed. This countermeasure must be constructed in a dry or dewatered condition. Without dewatering, unstable soil materials underlying the bed most likely will slough into the excavation trench. Thus, constructing a thickened toe is usually cheaper and easier. A third countermeasure is to add additional riprap over time as stones launch into the scour hole however, this can lead to additional bank erosion if the launching leaves bank areas devoid of riprap. For the launchable toe scour protection design, sufficient volume of riprap should be added to the toe to allow a full thickness section to be launched to the toe of the bottom of the estimate scour hole plus several feet for a safety factor.

When riprap is placed along the toe of the slope additional scour and local channel lowering occurs. Channel lowering can be attributed to the turbulence created by the added roughness from the riprap stones (Jin et al. 1990).

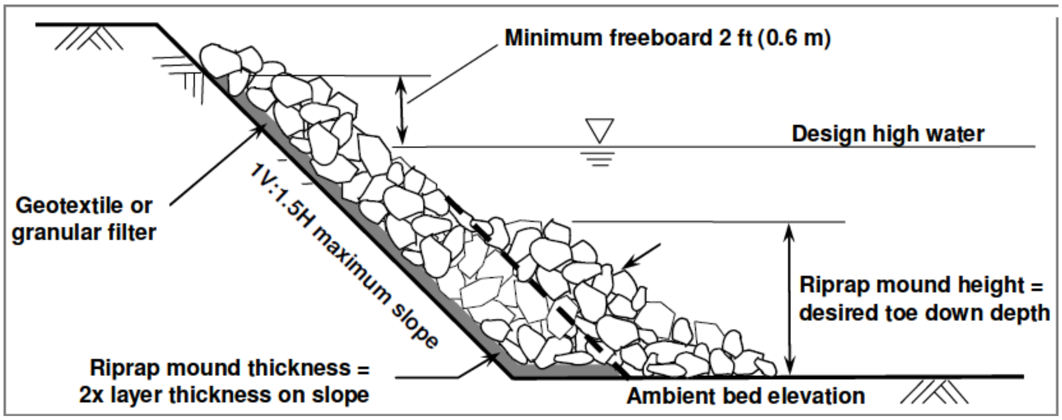


Figure 13–8. Riprap revetment with mounded toe section for launching (after Lagasse et al. 2009).

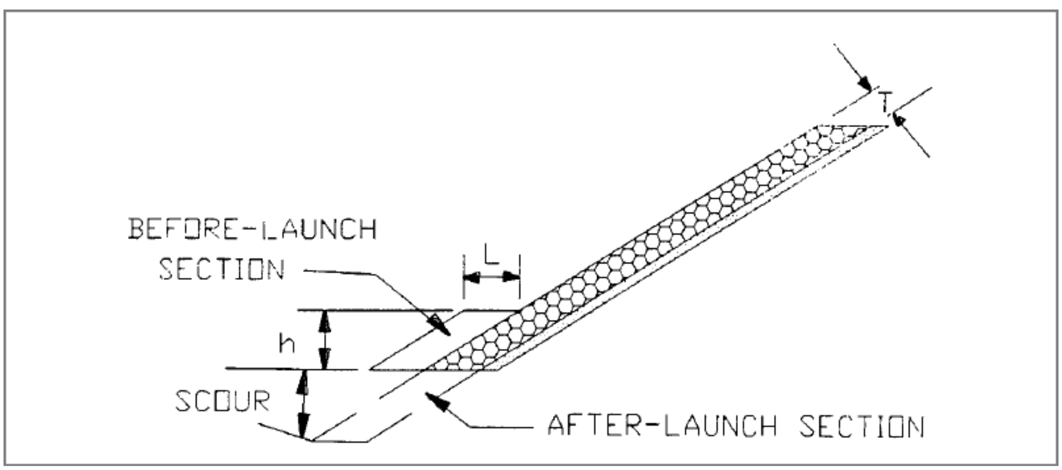


Figure 13–9. Launched stone schematic (after Maynord and White, 1995). As-built section can be placed above, on, or below the streambed. Above bed section can be constructed in a triangular shape.

Scour estimates can be made using the method by Maynard (1995) for the riprap revetment as shown in Appendix A.

Step 5. Determine riprap size, thickness, volume and shape. Design methods and a sensitivity analysis are presented in Section 13.2. Sizes and gradations can also be determined using design methods by Maynard (1995), Brown and Clyde (1989), USACE (1990), and Simons and Senturk (1992).

The following is from Biedenharn et al. (1997) and USACE (1991) on stone sizing:

> ". . . definite stone size results should be used for guidance purposes and revised if appropriate, based upon experience with specific project conditions, . . ."

Stone riprap should be able to withstand toe scour and channel migration (Chapter 5).

The volume of riprap is determined as the product of the length of the bank slope where riprap will be placed, the riprap thickness, and planview length of the project including tie-backs (see step 5 below). The toe scour amount should be added. Toe scour riprap is the volume that would uniformly cover the scour depth at the same bank slope which is above the toe. An additional 20% should be added for situations where a large amount of construction is underwater, and 10% should be added when most of the construction is accomplished outside of flowing river waters.

The volume of riprap material should include the launchable toe rock, tie backs, and loss during underwater placement. The volume should be increased by 20-30% for underwater placement with a launchable toe. If the riprap is placed in a dewatered bankline, and excavation to the anticipated scour depth is possible, the riprap volume should only be increased by about 10%.

The following criteria are recommended in Lagasse et al. (2005) and FHWA (2009) for revetment riprap:

- Layer thickness should not be less than the spherical diameter of the D100 stone nor less than 1.5 times the spherical diameter of the D50 stone, whichever results in the greater thickness.
- Layer thickness should not be less than 1 ft (0.30 m) for practical placement.
- Layer thickness determined either by criterion 1 or 2 should be increased by 50% when the riprap is placed underwater to compensate for uncertainties associated with this placement condition.
- Riprap should be well graded, and angular. Rounded river rock does not have the erosion resistance of angular well graded riprap.

Riprap stones should not be thin and platy, nor should they be long and needle-like. Therefore, specifying a maximum allowable value for the ratio *A/C*, also known as the shape factor, provides a suitable measure of particle shape, since the *B* axis is intermediate between the two extremes of length A and thickness *C*. *A* maximum allowable value of 3.0 is recommended:

$$\frac{A}{C} \leq 3 \tag{13.14}$$

For riprap applications, stones tending toward subangular to angular are preferred, due to the higher degree of interlocking, hence greater stability, compared to rounded particles of the same weight (FHWA, 2009).

Step 6. Design riprap gradation. The gradation design strongly effects the duration or life of the riprap gradation. Stone riprap material should be of sufficient size and gradation to withstand hydraulic forces, provide interlocking support, and prevent loss (erosion) of bank materials through the gaps between larger stones (Biedenharn et al. 1997).

Table 13-1 provides recommended gradations for ten standard classes of riprap based on the median particle diameter d_{50} as determined by the dimension of the intermediate ("*B*") axis. These gradations conform to those recommended in NCHRP Report 568 (Lagasse et al., 2006). The proposed gradation criteria are based on a nominal or "target" d_{50} and a uniformity ratio d_{85}/d_{15} that results in riprap that is well graded. The target uniformity ratio d_{85}/d_{15} is 2.0 and the allowable range is from 1.5 to 2.5 (FHWA, 2009).

Table 13–1. Minimum and Maximum Allowable Particle Size in Inches

Nominal Riprap Class by Median Particle Diameter		d_{15}		d_{50}		d_{85}		d_{100}
Class	***Size***	**Min**	**Max**	**Min**	**Max**	**Min**	**Max**	**Max**
I	6 in	3.7	5.2	5.7	6.9	7.8	9.2	12.0
II	9 in	5.5	7.8	8.5	10.5	11.5	14.0	18.0
III	12 in	7.3	10.5	11.5	14.0	15.5	18.5	24.0
IV	15 in	9.2	13.0	14.5	17.5	19.5	23.0	30.0
V	18 in	11.0	15.5	17.0	20.5	23.5	27.5	36.0
VI	21 in	13.0	18.5	20.0	24.0	27.5	32.5	42.0
VII	24 in	14.5	21.0	23.0	27.5	31.0	37.0	48.0
VIII	30 in	18.5	26.0	28.5	34.5	39.0	46.0	60.0
IX	36 in	22.0	31.5	34.0	41.5	47.0	55.5	72.0
X	42 in	25.5	36.5	40.0	48.5	54.5	64.5	84.0

Note: Particle size corresponds to the intermediate ("B") axis of the particle.

Based on the assumption volume of the stone is 85% of a cube, Table 13–2 provides the equivalent particle weights for the same ten classes, using a specific gravity of 2.65 for the particle density (FHWA, 2009).

Gradations can also be determined using design methods by Maynard (1995), Brown and Clyde (1989), USACE (1990), and Simons and Senturk (1992).

Table 13–2. Minimum and Maximum Allowable Particle Weight in Pounds

Nominal Riprap Class by Median Particle Weight		W_{15}		W_{50}		W_{85}		W_{100}
Class	**Weight**	**Min**	**Max**	**Min**	**Max**	**Min**	**Max**	**Max**
I	20 lb	4	12	15	27	39	64	140
II	60 lb	13	39	51	90	130	220	470
III	150 lb	32	93	120	210	310	510	1100
IV	300 lb	62	180	240	420	600	1,000	2,200
V	1/4 ton	110	310	410	720	1,050	1,750	3,800
VI	3/8 ton	170	500	650	1,150	1,650	2,800	6,000
VII	1/2 ton	260	740	950	1,700	2,500	4,100	9,000
VIII	1 ton	500	1,450	1,900	3,300	4,800	8,000	1,7600
IX	2 ton	860	2,500	3,300	5,800	8,300	13,900	30,400
X	3 ton	1,350	4,000	5,200	9,200	13,200	22,000	48,200

Note: Weight limits for each class are estimated from particle size by: $W = 0.85(\gamma_s d^3)$ where d corresponds to the intermediate ("B") axis of the particle, and particle specific gravity is taken as 2.65.

Step 7. Design edge treatments/transitions for toe and bank. Edge treatments should transition riprap revetments and stone toes into the bank to protect structure from upstream lateral movement within the range of expected meander migration, and protect the downstream bank from exit velocities and secondary flow turbulence. These are often needed for stone treatments that increase flow velocities along the toe. Scalloped erosion can form where a stone surface meets the downstream vegetated bank, or where the vegetation meets the downstream riprap revetment.

Upstream and downstream terminations should utilize a key trench that is dimensioned in relation to the d_{50} size of the riprap (FHWA, 2009). The key trench can be an increase in the riprap layer thickness or an alternate wing of concrete. The increase in volume of riprap is to provide articulating material if the edge of the bank protection begins to unravel. Consult the HEC23 manual (FHWA, 2009) for more guidance on the design of edge treatments.

Step 8. Riprap Acquisition. From FHWA (2009):

The designer should begin thinking about possible sources of rock for riprap. Sometimes it can be found on site, or can be produced at a nearby quarry. Good quality large rock can be surprisingly difficult to find. Trucking riprap any distance can be costly.

In addition to size, the rock has to withstand being submerged and also abraded by sediment in the flow. Not all rock from a quarry is durable. Standard test methods relating to material type, characteristics, and testing of rock and aggregates typically associated with riprap installations (e.g., filter stone and bedding layers) are provided in this section and are recommended for specifying the quality of the riprap stone. In general, the test methods recommended in this section are intended to ensure that the stone is dense and durable, and will not degrade significantly over time.

Rocks used for riprap should only break with difficulty, have no earthy odor, no closely spaced discontinuities (joints or bedding planes), and should not absorb water easily. Rocks comprised of appreciable amounts of clay, such as shales, mudstones, and claystones, are never acceptable for use as fill for gabion mattresses. Table 13-3 summarizes the recommended tests and allowable values for rock and aggregate.

Step 9. Design the granular filter. A geotextile filter should not be used in a river bank if there is any potential for the bank to erode or shift. Loose edges of geotextile fabric are easily worked loose during high flows, subsequently blowing out banks instead of protecting banks. Riprap is best placed on a smooth slope and alignment, covered with a granular filter to help prevent piping failures. Filters should be used to improve the life of riprap installations, even when the riprap is well graded.

A Colorado State University spreadsheet automates the design of a granular filter outlined in River Mechanics by Julien (2002) on pages 247-248. Knowing the gradation of the riprap and the gradation of the soil, determine if a granular filter blanket is required, and if so, find an appropriate gradation.

The relationships below can be used to determine if a filter is needed and to select a gradation for the filter material (USACE, 1980). The subscripts "upper" and "lower" refer to the riprap and soil, respectively, when evaluating filter need; the subscripts represent the riprap/filter and filter/soil comparisons when selecting a filter blanket gradation.

$$\frac{D_{15\,upper}}{D_{85\,lower}} < 5 \tag{13.15}$$

$$5 < \frac{D_{15\,upper}}{D_{15\,lower}} < 40 \tag{13.16}$$

$$\frac{D_{50\,upper}}{D_{50\,lower}} < 40 \tag{13.17}$$

In the above relationships, "upper" refers to the overlying material and "lower" refers to the underlying material. The relationships must hold between the filter blanket and base material and between the riprap and filter blanket.

Table 13–3. Recommended Tests for Riprap Quality

Test Designation	Property	Allowable value	Frequency[1]	Comments
AASHTO TP 61	Percentage of fracture	<5%	1 per 20,000 tons	Percentage of pieces that have fewer than 50% fractured surfaces
AASHTO T 85	Specific gravity and water absorption	Average of 10 pieces: S_g >2.5 Absorption <1.0%	1 per year	If any individual piece exhibits an S_g less than 2.3 or water absorption greater than 3.0%, an additional 10 pieces shall be tested. If the second series of tests also exhibits pieces that do not pass, the riprap shall be rejected.
AASHTO T 103	Soundness by freezing and thawing	Maximum of 10 pieces after 25 cycles: <0.5%	1 per 2 years	Recommended only if water absorption is greater than 0.5% and the freeze-thaw severity index is greater than 15 per ASTM D 5312.
AASHTO T 104	Soundness by use of sodium sulfate or magnesium sulfate	Average of 10 pieces <17.5%	1 per year	If any individual piece exhibits a value greater than 25%, an additional 10 pieces shall be tested. If the second series of tests also exhibits pieces that do not pass, the riprap shall be rejected.
AASHTO TP 58	Durability index using the micro-deval apparatus	>90 – Severe >80 – Moderate >70 – Mild	1 per year	Severity of application per Section 5.4, CEN (2002). Most riverine applications are considered mild or moderate.
ASTM 3967	Splitting tensile strength of intact rock core specimens	Average of 10 pieces: >6 MPa	1 per year	If any individual piece exhibits a value less than 4MPa, an additional 10 pieces shall be tested. If the second series of tests also exhibits pieces that do not pass, the riprap shall be rejected.
ASTM D 5873	Rock hardness by rebound hammer	See Note (2)	1 per 20,000 tons	See Note (2)
Shape	Length to thickness ratio a/c	<10%, d_{50} < 24 in. <5%, d_{50} > 24 in.	1 per 20,000 tons	Percentage of pieces that exhibit A/C ratio greater than 3.0 using the Wolman Count method (Lagasse et al. 2006)
ASTM D 5519	Particle size analysis of natural and manmade riprap materials		1 per year	See Note (3)
Gradation	Particle size distribution curve		1 per 20,000 tons	Determined by the Wolman Count method (Lagasse et al. 2006), where particle size "d" is based on the intermediate ("B") axis

(1) Testing frequency for acceptance of riprap from certified quarries, unless otherwise noted. Project-specific tests exceeding quarry certification requirements, either in performance value or frequency of testing, must be specified by the Engineer.

(2) Test results from D 5873 should be calibrated to D 3967 results before specifying quarry-specific minimum allowable values.

(3) Test results from D 5519 should be calibrated to Wolman Count (Lagasse et al. 2006) results before developing quarry-specific relationships between size and weight; otherwise, assume W = 85% that of a cube of dimension "d" having a specific gravity of Sg

The layer thickness of stone filters should be a minimum of 4 times the d_{50} of the filter stone, or 6 inches, whichever is greater. The thickness of granular filters should be increased by 50% when placing a granular filter under water (FHWA, 2009).

> The filter must retain the coarser particles of the subgrade while remaining permeable enough to allow infiltration and exfiltration to occur freely. It is not necessary to retain all the particle sizes in the subgrade; in fact, it is beneficial to allow the smaller particles to pass through the filter, leaving a coarser substrate behind (FHWA, 2009).

Step 10. Evaluate Constructability. Coffered areas are difficult to dewater, especially if attempting to excavate the full depth of the toe. Some permitting will allow construction in standing water with the coffered area stilling the water to prevent large sediment plumes downstream. For stream work, assume most construction will be in-the-wet.

The contractor should evaluate the risk of high flows during the construction window and incorporate these assessments into construction planning including coffering operations. The designer can help to keep construction costs down by considering both construction windows and flow regimes in the project scheduling.

Bankline alignments should be graded to form a smooth arc prior to riprap placement. This provides for flow lines which are parallel to the bankline and a smooth transition with the upstream and downstream bank alignment. The bank slope should also be graded prior to riprap placement.

When riprap is dumped or pushed off the bank top for placement on a slope, there is sorting with the large sized material resting near the bottom of the bank toe. Riprap functions best when placed without material sorting. This can best be accomplished by moving stones in place with a hydraulic excavator after dumping on the bank slope. A hydraulic excavator with sufficient reach should be used to bring the large material up the slope, shape the launchable toe section, and ensure that the revetment is uniform thickness, and uniform size distribution.

Front-end loaders work well for spreading granular filters on slopes flatter than approximately 1V:4H. A typical minimum thickness for granular filters is 0.5 to 1.0 ft depending on the size of the overlying riprap and whether a layer of bedding stone is placed between the filter and the riprap. Filter material can be placed under water using a large diameter tremie pipe to control the placement location and thickness. Placement in this way also reduces the potential for rock in the filter to segregate.

Construction issues include access, bank clearing and shaping, having large enough equipment for large stone placement, and turbidity due to bank shaping and stone placement. Adequate scour protection and flanking protection are needed for the method to be reliable. Stone may be more costly than other materials, especially depending on local availability. Heavy equipment is required on large projects for efficient placement (Watson et al., 2005). Riprap is considered unaesthetic for most locations and does not compare favorably with other types of bank stabilization or channel maintenance methods for environmental effectiveness.

13.3.2 Discussion and Recommendations

Riprap revetments, when properly designed and constructed, provide reliable erosion protection for high value infrastructure by hardening banks.

Risk and Failure. Level of reliability and durability is high unless there are channel instabilities such as continuing incision and channel migration processes in the river. However this method eliminates all riparian vegetation and associated habitat from the bank. Riprap is relatively sterile in comparison.

The most common cause of riprap failure is excessive scour, upstream channel migration and inadequate tie-backs, or insufficient rock sizes and gradation. There is no guarantee that a riprap revetment is fail proof. The risk is due to the variability of hydro-dynamic lift and drag forces acting on the riprap particles, and channel dynamics.

Monitoring and Maintenance. Monitoring and maintenance of longitudinal or direct bank stabilization methods helps ensure successful performance over the lifespan of the protection. Even properly designed protection requires some maintenance eventually. Because of the dynamic nature of river channels, a lack of maintenance often results in major failures. Therefore, monitoring of the bank protection in a dynamic environment is more important than monitoring structures in a static environment. A critical component of the longitudinal riprap protection work (the toe) is underwater and not visible for observation. Evidence of excessive toe scour can be dislodged riprap along the bank. Monitoring cross sectional and toe scour changes often involves significant but cost-effective effort and expense. Foresight is essential, because it is too late to begin an effective monitoring program once unforeseen damage requires major repair.

There should be formal requirements or guidance for monitoring and maintenance of the protection works. If there is no existing requirements or guidance available, the project manager should develop one such document.

Maintenance requirements include replacing riprap that has dislodged and stabilizing the upstream bend. Inspection following peak flows will help ensure continued success of the riprap by identifying potential weak points where riprap has eroded or been undermined. Inspection and maintenance will ensure continued stability of riprap. Riprap on steep 1V to 1.5H banks, in high energy streams can take on the appearance of a bulging over-steepened base as smaller particles are washed out of the mix. The revetment is more durable with a 1V to 2H slope.

Maintenance also may include replanting where vegetation die off on banks is common, and replacing biodegradable erosion control fabrics where needed to insure vegetation re-growth. This is especially true in arid climate zones with fluctuating ground water tables. Inspection should focus on potential weak points such as the upstream and downstream transition between undisturbed and treated banks (WDFW, 2003).

13.4 Riprap Windrow and Trench Filled Riprap

Illustrations of a riprap windrow and trench filled riprap are presented in Figure 13–10. These methods are useful for actively eroding sites when the stream bed cannot be accessed for construction, when permitting based on sediment concerns does not allow for construction or when a site may require erosion protection at some future and unspecified time (i.e. spatial certainty but temporal uncertainty). These installations can also cost less than riprap revetment construction since all work can be performed on dry banks without dewatering requirements. Balanced against these advantages is the disadvantage of not being able to uniformly apply riprap coverage. Some maintenance is required to address locations where riprap did not distribute uniformly over the slope.

Model investigations of windrow revetments and rock fill trenches were used to develop some of the design guidance presented here (USACE, 1981; Lagasse et al., 2009). The degree of riprap erosion protection required is a function of the channel depth, bank height, material size and estimated scour. “Bank height does not significantly affect the final revetment; however, high banks tend to produce a non-uniform revetment alignment. Large segments of bank tend to break loose and rotate slightly on high banks, whereas low banks simply "melt" or slough into the stream.” Windrow revetments and rock fill trenches have high durability and project life as long as supplemental riprap is added after launching.

13.4.1 Design Procedure

Refer to the steps in the Riprap Revetment Design Procedures for guidance on most aspects of the Riprap Windrow and Trench Filled Riprap design. Listed below is guidance specific to these methods.

Determine riprap size and gradation. Trench filled revetments and windrows should be constructed of well-graded, self-launching stone (USACE 1981) that is of adequate size. Size and gradation can be determined by referring to section 13.2 and 13.3 Riprap Revetments.

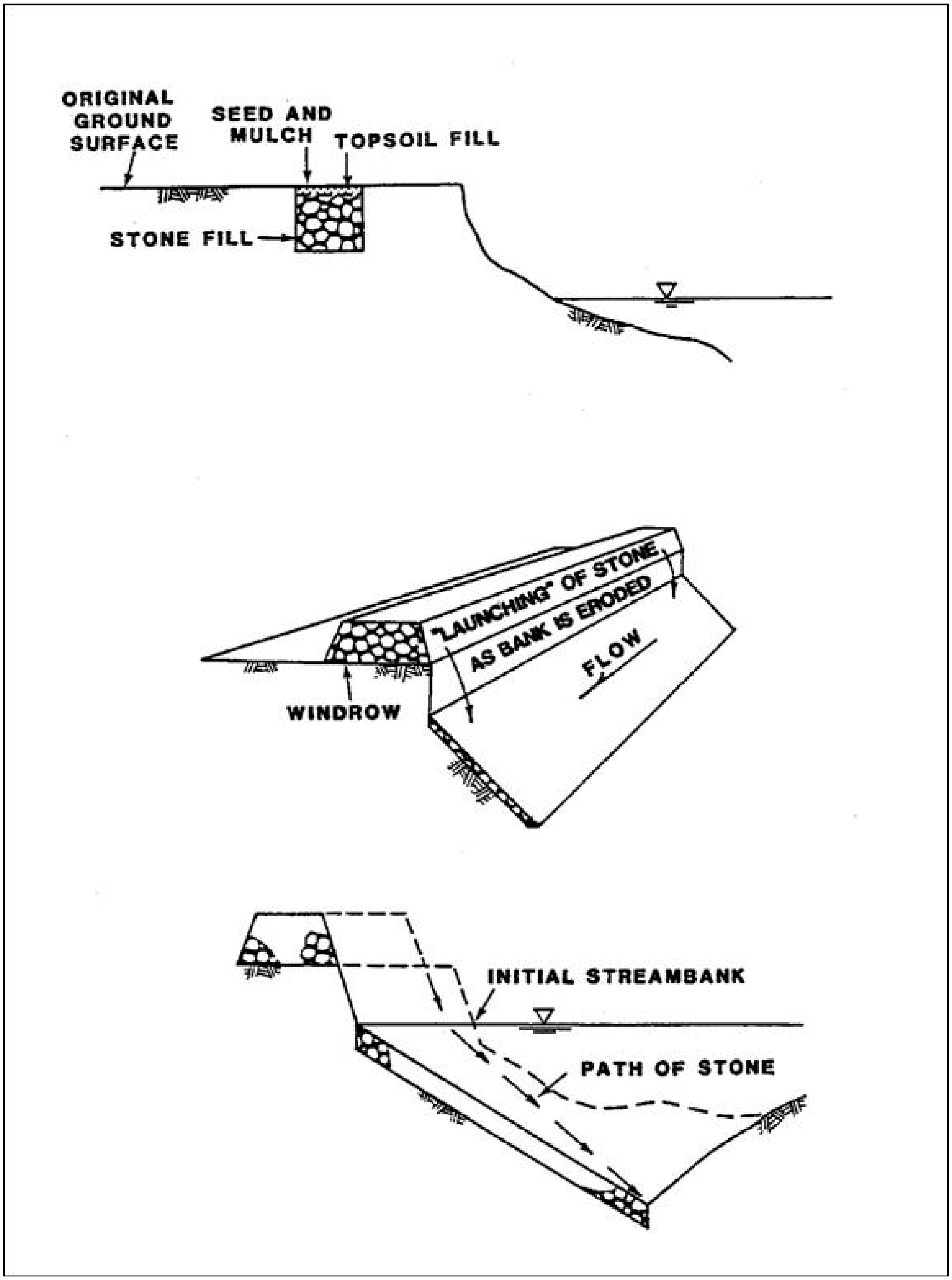


Figure 13–10. Riprap windrow and trench filled riprap (after USACE, 1981)

"Stone size influences the thickness of the final revetment, and a smaller gradation of stone forms a more dense, closely chinked protective layer. Stones must be large enough to resist being transported by the stream, and a well-graded stone should be used to ensure that the revetment does not fail from leaching of the underlying bank material. Large stone sizes require more material than smaller stone sizes to produce the same relative thickness of revetment. In general, the greater the stream velocity, the steeper the side slope of the final revetment. The final revetment slope will be about 15% flatter than the initial bank slope." (USACE, 1981; Lagasse et al., 2009)

Granular filters help improve the durability of a riprap revetment, but a granular filter cannot be placed with a trench filled revetment or a windrow. Without a granular filter, adequate riprap gradation becomes even more essential.

Determine windrow or trench end treatments or tie-backs. "A windrow segment should be extended landward from the upstream end to reduce the possibility of outflanking of the windrow."

Supplemental riprap should be placed at the bottom of the trench. If the vertical launch distance is less than 15 ft and it includes dry placement, the volume should be increased by 25 percent. If the launch distance is less than 15 ft with underwater placement, then the volume of riprap in the launching area should be increased by 50 percent. With dry placement a greater than 15 ft launch distance, the volume in the bottom of the launch area should be increased by 50 percent and with wet placement, 75 percent (Biedenharn et al., 1997).

For trench filled revetments, the height of the stone section is generally one-half to one times the width (Biedenharn et al., 1997).

Trench filled riprap can be constructed using a rectangular trench or a trench for a trapezoidal buried section. Trench filled riprap is best constructed on a slope of about 1:1 with a thickened toe section containing the launchable rock.

Placing the stone at the lowest practical elevation constructed during low flows can often place the toe of the trench below the high flow water surface elevation. This is the most advantageous placement because the launch distance is the shortest. A greater volume of stone is required for trench filled riprap because of non-uniform launching. A method to determine this volume is contained in Biedenharn et al (1997) and USACE (1991). The thickness of the riprap should equal one times the spherical diameter of the upper limit of the W_{100} stone or 1.5 times the spherical diameter of the upper limit of the W_{50} stone, whichever is greater Biedenharn et al (1997).

Material from the trench excavation can be used to raise the local height of the eroding bank to prevent non-uniform overtopping. This aids in more uniform launch rates.

Windrows placed in a trapezoidal shape are best because this cross section supplies a steady supply of stones. A triangular shape is not desirable because the quantity of stone diminishes as the windrow is undercut (Biedenharn et al., 1997).

The upstream and downstream ends of riprap windrows or trench filled riprap should be protected against erosion by placing tiebacks at the ends of longitudinal stone toe.

Upstream and downstream tiebacks should be designed based on local experience and geomorphic analysis. These locations can be the zones of slackwater upstream of and downstream from the project site (NRCS, 1996). Length of tiebacks is based upon expected channel migration during launching flow events. Tiebacks should be angled about 30 degrees from the primary flow direction. Tiebacks with an angle of 90 degrees have resulted in failures at the downstream end of the structure due to flow expansion (McCullah and Gray, 2005).

Trench Size, Riprap Volume and Windrow size. A triangular shaped windrow is the least desirable, a trapezoidal shape provides a more uniform blanket of riprap on the eroding bank, and a rectangular provides the best coverage. A rectangular shape is most easily placed in an excavated trench.

The trench should be excavated to the lowest practical level during low flows. The trench will most often be trapezoidal with 1:1 side slopes. The height of the stone should be ½ to 1 times the width for launching.

The volume of riprap is determined as the product of the length of the bank slope where riprap will launch plus a triangular section which will remain in the trench after launching (or for the windrow it is the bank height, and slope plus scour), the riprap thickness, and planview length of the project including tie-backs (see step 5 below). The toe scour amount is estimated based on maximum scour depth. Toe scour riprap is the volume that would uniformly cover the scour depth at the existing bank slope. A minimum of 25% should be added to the riprap volume.

Windrows should be trapezoidal shaped to provide launching that is as uniform as possible and supplies a steady supply of stones. Volume per linear foot along the axis of the windrow would be the volume per linear foot determined in the paragraph above.

Geotechnical Analysis. A geotechnical analysis is recommended to determine bank stability with the addition of the weight of a riprap trench or windrow (Biedenharn et al., 1997).

Constructability Assessment. Trench or windrow should be in a smooth alignment and the bank slope graded prior to riprap placement.

When riprap is dumped or pushed off the top of bank for placement on a slope, sorting occurs with the large sized material resting near the bottom of the bank toe. A hydraulic excavator with sufficient reach should be used to bring the large material up the slope, shape the launchable toe section, and ensure that the revetment is uniform thickness, and uniform size distribution.

Constructability issues should be less than riprap revetment construction due to the opportunity to excavate under mostly dry conditions. Other issues include access and ensuring bank stability during construction with heavy equipment. Stone should be added after the windrow launches, on an “as-needed” basis until bank stabilization is complete. Site-specific conditions will determine how much additional stone is needed. Additional stone is needed because of non-uniformity of bank erosion and launch rates. The bank may need some vegetation clearing for large equipment to construct the windrow or trench.

Active Maintenance. Additional riprap in selected reaches may be necessary after launching has occurred to compensate for inefficient launching or where there is inefficient launching with slab failures or small rotational slips (Biedenharn et al., 1997).

The efficiency of launching is higher for trenched riprap than for riprap windrows because riprap launches a longer distance for windrows and experiences a less uniform launch rate and greater size sorting. During augmentation riprap should be placed with hydraulic excavators or other equipment to insure uniform thickness and uniform size gradation.

Augmentation of end treatments or tie-backs with additional riprap may be necessary after the windrow or trench filled riprap has launched. In the case of launched riprap windrows, tiebacks may need to excavated into the bank after launching and riprap augmentation

13.4.2 Discussion and Recommendations

Riprap Windrows and Trench filled riprap are useful when the river is eroding in a predictable pattern and can prevent the river from eroding beyond the desired location and alignment. The efficiency of launching is higher for trenched riprap than for riprap windrows because riprap launches a longer distance for windrows and experiences a less uniform launch rate and greater size sorting.

Risk and Failure. The level of reliability is high, provided that the riprap augmentation takes place post launching. Common failure modes include:

- Installation on streambanks composed of cohesive soils
- Trench excavation causing bank instability as a result of vegetation disturbance
- Inadequate size and quantity of rock so it does not fully launch
- Inadequate coverage after launching and lack of filter decreases the life of the installation.

Counter measures include construction on the bank with non-cohesive soil material and ensuring there is adequate size and quantity of riprap to overcome any effects of loss of bank strength due to vegetation disturbance and the launching process. The river reach should be stable for this method to be effective.

Additional riprap in selected reaches may be necessary after launching has occurred to compensate for inefficient launching or where there is inefficient launching with slab failures or small rotational slips (Biedenharn et al., 1997).

Monitoring and Maintenance. Monitoring and maintenance of longitudinal or direct bank stabilization methods is essential to ensure successful performance over the lifespan of the protection. Even properly designed protection requires some maintenance eventually. Because of the dynamic nature of river channels, a lack of maintenance often results in major failures. Therefore, monitoring of the bank protection in a dynamic environment is more important than for any structures in a static environment. Critical component of the longitudinal riprap protection work (the toe) is underwater, thus not visible to simple observation. Evidence of excessive toe scour can be dislodged riprap along the bank.

Monitoring cross sectional and toe scour changes often involves significant but cost-effective effort and expense. Foresight is essential, because it is too late to begin an effective monitoring program once unforeseen damage requires major repair.

There should be formal requirements or guidance for monitoring and maintenance of the protection works. If there is no existing requirements or guidance available, the project manager should develop one such document.

Maintenance requirements include replacing riprap that has dislodged, adding additional tieback length, and stabilizing the upstream bend. Inspection following peak flows will help ensure continued success of the riprap by identifying potential weak points where riprap has eroded or been undermined. Inspection and maintenance will ensure continued stability of riprap.

Maintenance also may include replanting where die off is common, and replacing biodegradable erosion control fabrics where needed to insure vegetation re-growth. This is especially true in arid climate zones with fluctuating ground water tables. Inspection should focus on potential weak points such as the upstream and downstream transition between undisturbed and treated banks (WDFW, 2003).

13.5 Longitudinal Peak Stone Toe

Longitudinal Peak Stone Toe (LPST) can be used to re-align a channel or constrict an overly wide channel. The top elevation of a stone fill toe is usually well below the top bank elevation (figure 13–11). A top elevation and crown width for the stone are specified along with bank grading and/or filling to provide for a consistent cross-section of stone, to provide both protection and a launchable toe. These structures are not suitable for reaches where rapid bed degradation is likely, or where scour depths along the toe will be greater than the height of the LPST, unless the toe foundation can be constructed below the depth of degradation or scour. A stone toe is usually quantified as volume per linear foot.

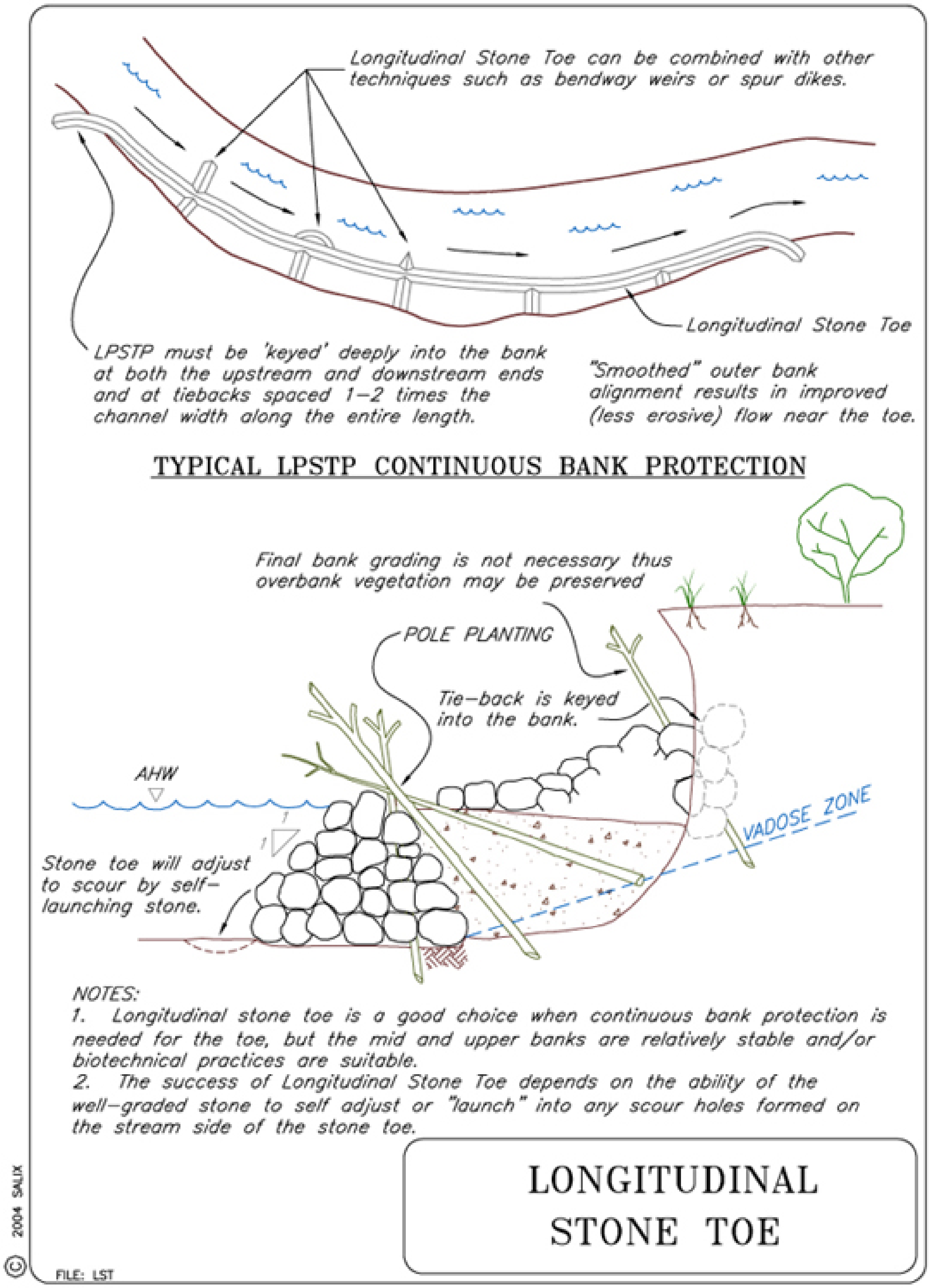


Figure 13–11. Longitudinal peak stone toe. Used by permission of the Transportation Research Board (Report No. 544) and Salix Applied Earthcare (NCHRP 2005).

A longitudinal stone toe can be constructed with or without bioengineering. If the design is integrated with vegetation, stone riprap is placed from the toe of the slope up to an elevation where riparian vegetation normally grows. This can be determined by either the green line elevation or the channel forming discharge.

Vegetation is used to protect the remainder of the slope up to the top of the bank or a peak flow design discharge such as the 25-year event water surface elevation. For more information on stone toes and vegetated banks refer to chapter 9 on vegetation methods.

Design of a longitudinal stone toe is similar to the design of riprap revetment presented in section 13.3 so most steps in this section can be followed. Listed below are items where the design varies from steps in the riprap revetment section.

13.5.1 Design Procedure

Only design steps for the main design variables are presented here. Refer to riprap revetments for more guidance on stone sizing, gradation and other factors.

Design procedures were developed from experience, including field observations and data, with some information from Lagasse et al. (2009), McCullah and Gray (2005), and NRCS (2005, 2007).

Determine height of peaked stone toe. The elevation of the longitudinal peak stone toe should be the water surface elevation corresponding to the discharge below which vegetation does not grow or the toe zone of the bank slope. Section 4.4 contains details on estimating this discharge.

Determine the number and location of tie-backs. Tie backs are typically constructed 1-2 channel widths apart. If the stream has suspended sediment which deposits in the channel and forms bars 2 times the channel width would be sufficient.

For streams without much suspended sediment the tie backs should be more closely spaced (1/2 to 1 times the channel width) to prevent flow acceleration between tie backs in the absence of sediment deposition.

As part of the project geomorphic analysis the expected channel migration and maintenance capability. If no maintenance is available upstream tie-back length should be sufficient to protect structure from upstream lateral movement within the range of expected meander migration. Length should be increased near high value riverside protected infrastructure.

Downstream tie back could be constructed a sufficient length to protect against erosion due to high exit velocities and turbulence. Usually 30-50 ft is sufficient.

The larger distance would be best used when there is high value infrastructure being protected.

There can be more need for tiebacks instead of end treatments with the longitudinal stone toe if the structure is set in the flow with no upper banks. LPST structures must be keyed into the bank at both the upstream and downstream ends and at regular intervals along the entire length. Typically, the keys are spaced at 50 to 100 ft intervals up to 1 to 2 channel widths on larger waterways. Keys at the upstream and the downstream ends of the LPST should go far enough into the river bank so river migration will not flank the LPST. In some cases this may be the distance of anticipated upstream channel migration. The key trenches at the upstream and downstream ends should be excavated into the bank at an angle of approximately 30°, with the primary flow direction and of sufficient length that flows will not be able to get around them during the design storm. A gentle angle is important for the end keyways, often referred to as "refusals", because it allows for smooth flow transitions coming into and flowing out of the treated reach. Tiebacks or "refusals" oriented at 90° to the bank have resulted in many failures at the downstream end of the structure, due to flow expansion at that point (D. Derrick, personal communication, 2000).

Where the bank materials are highly erodible, and the adequacy of an unsupported stone placed along the toe of the bank may be marginal, stone dikes can be placed at intervals as tiebacks to prevent erosion from forming behind the structure. A spacing of one to two multiples of channel width can be used between tiebacks. At the very least a tieback at the downstream limit of the structure is recommended.

For a longitudinal peak stone toe (LPST), often used in a high energy, sand bed rivers, tie-backs may also be used in place of edge treatments. Tie-backs extend the revetment to areas of non-eroding velocities and relatively stable banks and can be constructed at the ends of longitudinal stone toe. Geomorphic conclusions on the channel stability and meander rates should be consulted when designing tie-backs. Length should be increased if there is high uncertainty about the river alignment stability or if the revetment is protecting high-value infrastructure.

Upstream and downstream tiebacks should be designed based on local experience and geomorphic analysis. These locations can be the zones of slackwater upstream of and downstream from the project site (NRCS 1996). Length of tiebacks is based upon expected channel migration, and maintenance capability. If no maintenance, then length should be sufficient enough to protect structure from upstream lateral movement within the range of expected meander migration.

Length should be increased near high value riverside protected infrastructure.
In some cases the length of tiebacks should be the historical width of the meander belt width.

Tiebacks should be angled about 30 degrees from the primary flow direction.

Tiebacks with an angle of 90 degrees have resulted in failures at the downstream end of the structure due to flow expansion (McCullah and Gray 2005).

Determine riprap volume. The amount of stone used is based on tons or volume per linear foot. In determining the tonnage you first must calculate the depth of scour resulting in the stone placement. Two tons/linear ft are the most common tonnage, resulting in approximately 5 ft of toe protection. The volume of riprap is determined as the product of the plan view length of the LPST multiplied by the LPST cross sectional area. The cross sectional area is the height based upon the elevation below which vegetation does not grow with 1V:1.5H side slopes. An additional volume is added for scour and is the volume that would uniformly cover the scour depth at the same bank slope near the LPST or 1V:1.5H. At least 20% should be added to the volume to account for riprap loss during launching peak flow events. Tieback riprap volume should be the same as the LPST volume per linear foot or increased by 20% due to increased riprap erosion potential from turbulence.

Construction Considerations. When riprap is placed in the longitudinal peak stone toe there will be sorting with the large sized material resting near the bottom of the peaked section. Equipment should be used to make the peak stone toe and tieback should be used to bring the large material up the slope, shape the launchable toe section, and ensure that the revetment is uniform thickness, and uniform size distribution.

13.5.2 Discussion and Recommendations

Some launching may occur and this needs to be accounted for in the longitudinal volume of the stone toe. Flanking protection is critical especially in incised channels with unstable banks.

Risk and Failure. The success of the method depends on the launching of material into the scour hole. The weight of stone (loading of toe) might prevent some shallow geotechnical bank failures. The LPST captures alluvium and upslope failed material on bank side of structure. It functions well where outer bank alignment makes abrupt changes, where the bank must be built back into the stream (realignment of channel, or construction of a backfilled vegetative bench or terrace for habitat improvement and/or velocity attenuation), where a minimal continuous bank protection is needed, or where a “false bankline” is needed. It can also be used in combination with other methods (bendway weirs, spur dikes, bioengineering, joint planting, live siltation, and live staking).

Longitudinal stone toe may be flanked during extremely high flows if the key trenches are incorrectly built or if the tiebacks are spaced too widely or are constructed with inadequate amounts of stone. Terminal keyways or "refusals" oriented at 90° to the bank have resulted in many failures at the downstream end of the structure, due to flow expansion at that point (McCullah and Gray, 2005) and should be angled no more than 30°.

Monitoring and Maintenance. Monitoring and maintenance of longitudinal or direct bank stabilization methods is essential to ensure successful performance over the lifespan of the protection. Even properly designed protection requires some maintenance eventually. Because of the dynamic nature of river channels, a lack of maintenance often results in major failures. Therefore, monitoring of the bank protection in a dynamic environment is more important than for any structures in a static environment. Critical component of the longitudinal riprap protection work (the toe) is underwater, thus not visible to simple observation. Evidence of excessive toe scour can be dislodged riprap along the bank. Monitoring cross sectional and toe scour changes often involves significant but cost-effective effort and expense. Foresight is essential, because it is too late to begin an effective monitoring program once unforeseen damage requires major repair.

There should be formal requirements or guidance for monitoring and maintenance of the protection works. If there is no existing requirements or guidance available, the project manager should develop one such document.

Maintenance requirements include replacing riprap that has dislodged, adding additional tieback length, and stabilizing the upstream bend. Inspection following peak flows will help ensure continued success of the riprap by identifying potential weak points where riprap has eroded or been undermined. Inspection and maintenance will ensure continued stability of riprap.

Maintenance also may include replanting where die off is common, and replacing biodegradable erosion control fabrics where needed to insure vegetation re-growth. This is especially true in arid climate zones with fluctuating ground water tables. Inspection should focus on potential weak points such as the upstream and downstream transition between undisturbed and treated banks (WDFW, 2003).

14 Future Directions

Methods that provide the most opportunity for geomorphic processes to continue generally have the greatest potential for long term sustainability and include methods in the categories of preserving the floodplain and reestablishing floodplain. There will always be locations where short-term costs will appear to drive the design and traditional bank hardening methods are selected, but experience has shown that long-term costs should be realistically assessed before settling for the less than optimal outcomes. Steepened banks, concrete and riprap were the methods of choice in the previous century, but we are now living with the fallout from those readily-applied methods. A single application of steepening banks and locking a bend to an immovable point appears innocuous, but the collective impact of repeatedly applying this method to a river system, especially in combination with flow reduction, can commit Reclamation to decades of costly corrections. The goals of bank stabilization are feasibility, sustainability and environmental effectiveness and omitting one of these elements degrades the success of the design. The intent of these guidelines is to aid the designer in the transition from traditional bank hardening methods to more geomorphic-based, and subsequently, more cost-effective bank stabilization solutions.

Beginning in the late 1800’s and early 1900’s, the general approach in river engineering was to control and develop the resource for economic benefit. Beginning with the 1980’s and continuing today, the use of native materials and geomorphic principals in river engineering is increasingly emphasized for environmental benefits, sustainability, and longer term economics. The publication of project applications and reported successes and failures on different stabilization methods are more readily available. In these guidelines, authors attempt to both consolidate design instructions, and confirm and winnow guidance for only the more successful methods. The process of developing six chapters of non-traditional methods is still in its infancy. Some figures can be improved based on Reclamation experiences and the inclusion of more figures would aid in presenting design guidance. It is the hope of the authors that this first version will be revised on many occasions. Specific areas recommended for improvements in future revisions are described below.

Chapters 7 and 8. Protecting floodplains and expanding floodplains both contain introductions to basic concepts and provide design guidance. Later versions could be updated with more site specific examples of project successes and failures, productive teaming, and creative solutions to working in developed/urban areas.

Chapter 9. The vegetation chapter in this first version presents concepts of vegetation design and some basic methods of installing/establishing vegetation. There are many topics that can be expanded in this chapter. Additional bank resistance values for different types of vegetation or vegetation communities could possibly be acquired through a more extensive literature search, and these values directly support an engineered approach to plant design. Background and descriptive information on individual species would aid planting designs. A description of planting design approaches was also considered but had to be set aside for a later version. Planning, design and handling actions during project construction can have a significant impact on the success of plant techniques, and should be included. And there are multiple other bioengineering methods with confirmed success using both live and harvested plant materials. In this first version of the Guidelines, the focus is on vegetation planting. Additional bioengineering techniques may be added in future versions of these Guidelines. Other bioengineering methods include brush layers, brush mattresses, brush or tree revetments, brush trenches, vertical bundles, and willow wattles that are all constructed from riparian willow shrubs. Fiberschine, erosion control fabric and hay bales can also be utilized to stabilize an eroding site. One source on these techniques is The Practical Streambank Bioengineering Guide by USDA, Natural Resources Conservation Service (Bentrup and Hoag 1998).

Chapter 10. A Large Wood National Manual (LWNM) is currently under development by US Bureau of Reclamation and US Army Corp of Engineers on design of woody debris. When this manual is available, the chapter on woody debris and rock boulders in these guidelines should be updated to reflect the methods presented in the LWNM manual.

Chapter 11. Channel design, similar to floodplain protection and floodplain expansion chapters, should be expanded with lessons learned from constructed sites that have been in operation through, ideally, a full range of high flow events. There are a wide range of papers available on newly constructed sites but more can be learned from both the failures and success of projects that have been in operation for a decade or more. Also additional reference materials on selecting stable channel form and geometry can be included, and more instruction on design of elements including channel lining.

Chapter 12. Very limited design guidance has been available for transverse structures. A large laboratory flume and numerical modeling study was undertaken by Reclamation and Colorado State University specifically to develop better guidelines for transverse structures as part of the writing of this manual. Adjustments to these methods should be reported in each successive version as the number of applications of transverse structures designed with this method increases.

Chapter 13. Bank hardening methods are focused on riprap. The first version of this chapter contains a description of design methods in Chapter 13, but several methods are reference and not described. Methods presented in Chapter 13 are not expected to change significantly in upcoming years but descriptions of more design methods may be included in later revisions.

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15 APPENDIX A – Scour Computation Methods

Presented here are equations for estimating degradation and computing bend scour; confluence scour, pier scour, and abutment scour.

A.1 Long Term Aggradation or Degradation

Approaches to estimating degradation depth include:

- 1D or 2D flow and sediment modeling;
- A stable slope analysis or armoring analysis
- a stable slope estimate knowing a downstream control
- a geomorphic assessment; and/or
- an assignment of depth based on a field review of other sites in the area

A.1.1 Modeling

Simulations using 1D or 2D flow and sediment models can be used to compute the degradation depth along a reach of river including the project site. The models should simulate a long-term flow series. Suggestions from ASCE (2004) on the flow series to use include:

- Actual historical flow series
- Continuous application of the channel forming discharge (bankfull, often 1.5-yr, or 1-yr to 5-yr for perennial streams; and 5-yr to 10-yr for ephemeral streams with bankfull discharge as the upper limit, Simons, Li & Associates, 1985)
- Continuous application of the "average annual event" integrated in terms of runoff volume and sediment yield

Changes in bed elevation after the simulated flow regime provides a quantitative value of aggradation or degradation.

A.1.2 Stable Slope and Armoring Methods

Schoklitsch Method

The Schoklitsch Method to calculating the stable slope, as presented in Pemberton and Lara (1984), is:

$$S_L = K\left(\frac{DB}{Q}\right)^{\frac{3}{4}} \tag{15.1}$$

Where:

S_L = Stable slope (ft/ft)
K = 0.00174 inch-pound units
B = Bankfull width (ft)
D = Mean bed particle size (mm)
Q = Dominant discharge (ft^3/s)

Note: Assumes zero or negligible sediment transport

Meyer-Peter, Muller Method

The Meyer-Peter, Muller Method for calculating the stable slope, as presented by Pemberton and Lara (1984), is:

$$S_L = K \left(\frac{Q}{Q_B}\right) \left(\frac{n_s}{{D_{90}}^{1/6}}\right)^{3/2} \frac{D}{d} \tag{15.2}$$

Where:

S_L = Stable slope (ft/ft)
K = 0.19
$\frac{Q}{Q_B}$ = Ratio of total flow to flow over the bed of the channel (ft^3/s)
Q = Dominant discharge (ft^3/s)
n_s = Manning's n for the streambed
D_{90} = Bed sediment diameter for 90% finer (mm)
D = Mean bed sediment diameter (mm)
d = Mean depth (ft)

Note: Assumes zero or negligible sediment transport

Shields Diagram Method

The use of Shields diagram (Pemberton and Lara, 1984) for computing a stable slope involves the relationship of the boundary Reynolds number varying with the dimensionless shear stress shown on figure A-1 (Simons, Li & Associates, 1985; figure 5.12).

$$R_* = \frac{U_* D}{\upsilon} \tag{15.3}$$

Where:

R_* = Boundary Reynolds number
U_* = Shear velocity = $\sqrt{S_L R g}$
S_L = Slope (ft/ft)
R = Hydraulic radius or mean depth for wide channels (ft)
g = Acceleration of gravity (32.2 ft/s^2)

D = Mean particle diameter (ft)
υ = Kinematic viscosity of water varying with temperature (ft^2/s)

and:

$$T_* = \frac{T_c}{(\gamma_s - \gamma_w)\,D} \tag{15.4}$$

Where:

T_* = Dimensionless shear stress
T_c = Critical shear stress
γ_s = Specific weight of particles (165.4 lb/ft^2)
γ_w = Specific weight of water (62.4 lb/ft^3)
D = Mean particle size (ft)

Note: Assumes zero or negligible sediment transport

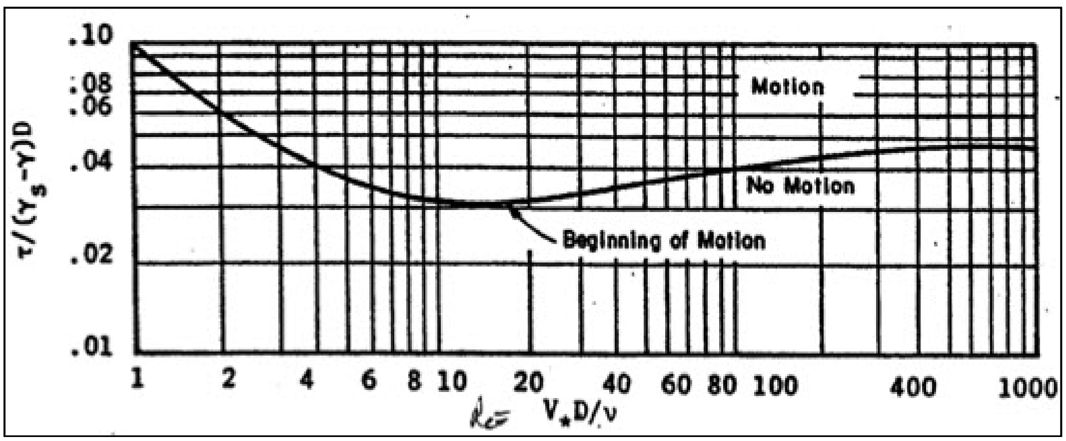

Figure A–1. Shields' relation from beginning of motion (adapted from Gessler 1971).

Lane's Tractive Force Method

The Lane's Tractive Force Method presented in Pemberton and Lara (1984) is:

$$S_L = \frac{T_c}{(\gamma_w d)} \tag{15.5}$$

Where:

S_L = Stable slope (ft/ft)
d = Mean flow depth (ft)
T_c = Critical tractive force (lb/ft^2) 〔may be read from figure A-2, use value from the curves for canals with clear water〕
γ_w = Specific weight of water (lb/ ft^3)

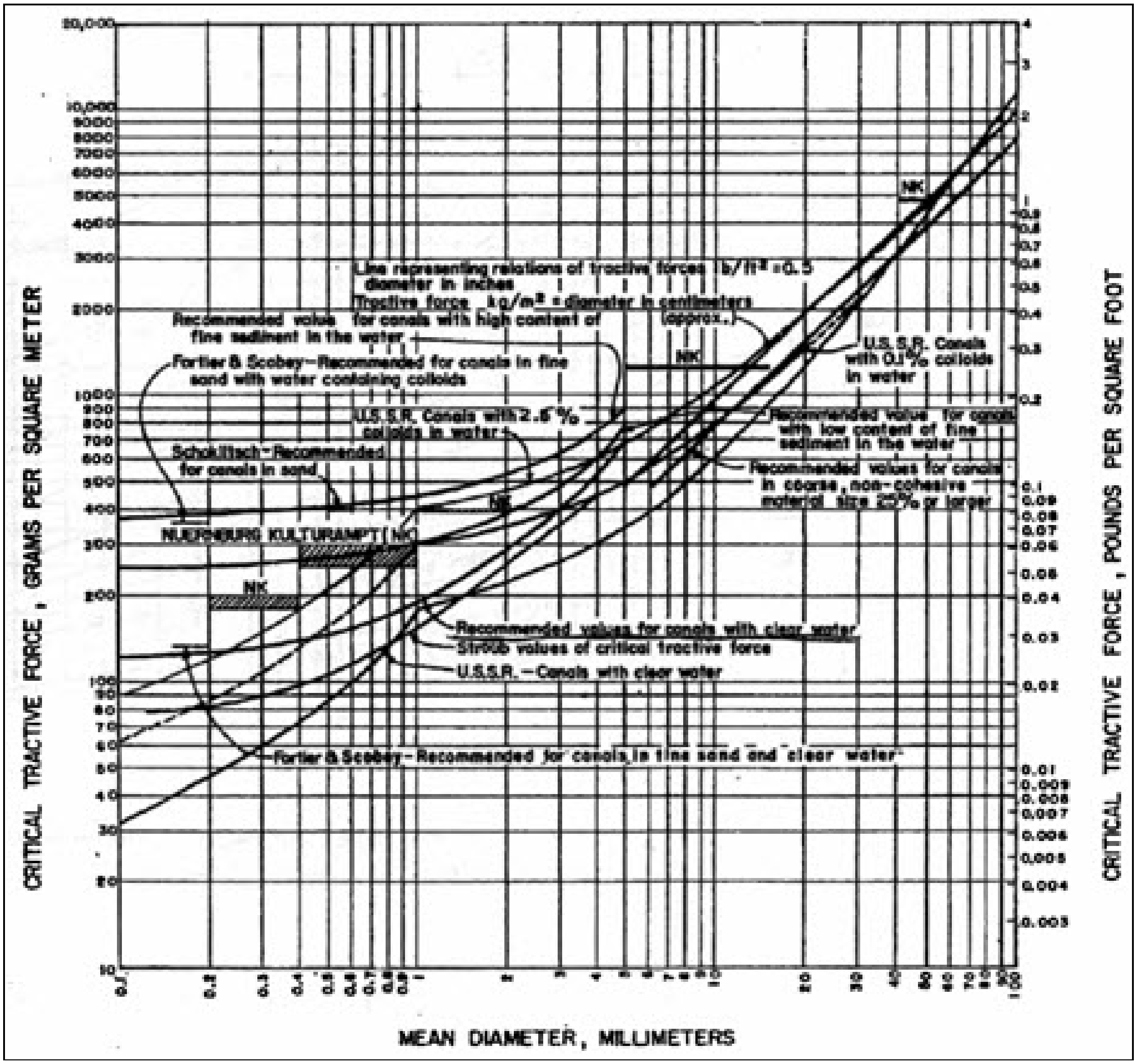

Figure A–2. Tractive force versus transportable sediment size, after Lane (Pemberton and Lara, 1984; figure 4).

Notes:

Method is based on results of the study by Lane (Pemberton and Lara, 1984), where he summarized the results of many studies in the relationship of critical tractive force versus mean particle size. This method also assumes zero or negligible sediment transport.

A.2 Bend Scour Equations

A bend in a channel will induce transverse or “secondary” currents that scour sediment from the outside of a bend and cause it to be deposited along the inside of the bend. The following four methods to predict the depth of scour in a river bend were developed after methods presented in Neil (1973) and Pemberton and Lara (1984).

A.2.1 Zeller Bend Scour Equation

The Zeller equation for estimating bend scour is presented in Simons, Li & Associates (1985), as:

$$y_{bs} = 0.0685 y_{max} V^{0.8} / (y_h^{0.4} S_e^{0.3}) * [(2.1(sin^2(\alpha/2)/cos\alpha)^{0.2} - 1)] \quad (15.6)$$

Where:

y_{bs}	=	Zeller depth of bend scour (ft^2),[2] measured below minimum channel elevation
y_{max}	=	Maximum depth of upstream flow (ft)
y_h	=	Hydraulic depth of upstream flow (ft)
V	=	Mean velocity of upstream flow (ft/s)
S_e	=	Upstream energy slope (ft/ft)
	=	Angle formed by projection of channel centerline from point of curvature (P.C.) to a point which meets a line tangent to the outer bank of channel (degrees)
α	=	Angle formed by projection of channel centerline from P.C. to a point which meets a line tangent to the outer bank of channel (degrees)

Note the term with the α coefficients. Rather than determine the angle α, the entire $\sin^2(\alpha/2)/\cos\alpha$ term can be determined using the following formula:

$$sin^2(\alpha/2)/cos\alpha = W/(4r_c) \quad (15.7)$$

Where:

r_c	=	Radius of curvature to centerline of channel (ft)
W	=	Channel top width of upstream flow (ft)

The longitudinal extent of the bend scour component is difficult to quantify. The Rozovskii (1961) equation can be used for predicting the distance from the end of a bend to where the secondary currents will be a minimum.

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[2] Measurements used in this appendix are as follows: ft = feet, ft/s = feet per second, ft/ft = feet per foot, ft^2/s = square feet per second, ft^3/s = cubic feet per second, ft^3/s/ft = cubic feet per second per foot, lb/ft^2 = pounds per square foot, lb/ft^3 = pounds per cubic foot

As a conservative estimate of the longitudinal extent of bend scour, both upstream and downstream of the curve, assume it extends a distance X upstream of the P.C. and a minimum of X downstream of the P.T. The location of the point P.C. and P.T. are shown in Figure A-3. The following is a modified version of the Rozovskii (1961) equation to estimate distance X:

$$X = 2.3\left(C/g^{1/2}\right)y = \left(0.6y^{1.7}\right)/n \qquad (15.8)$$

Where:

- X = Distance from the end of channel curvature (point of tangency [P.T.]) to the downstream point at which secondary currents have dissipated (ft)
- C = Chezy coefficient = (1.486/n) $R^{1/6}$
- g = Gravitational acceleration (32.17 ft^2/s)
- y = Depth of flow: use the maximum flow depth, exclusive of scour within the bend (ft)
- n = Manning's roughness coefficient

Zeller's equation estimates the maximum scour in sand bed channels.
The equation is based on the assumption of constant stream power through the channel bend.

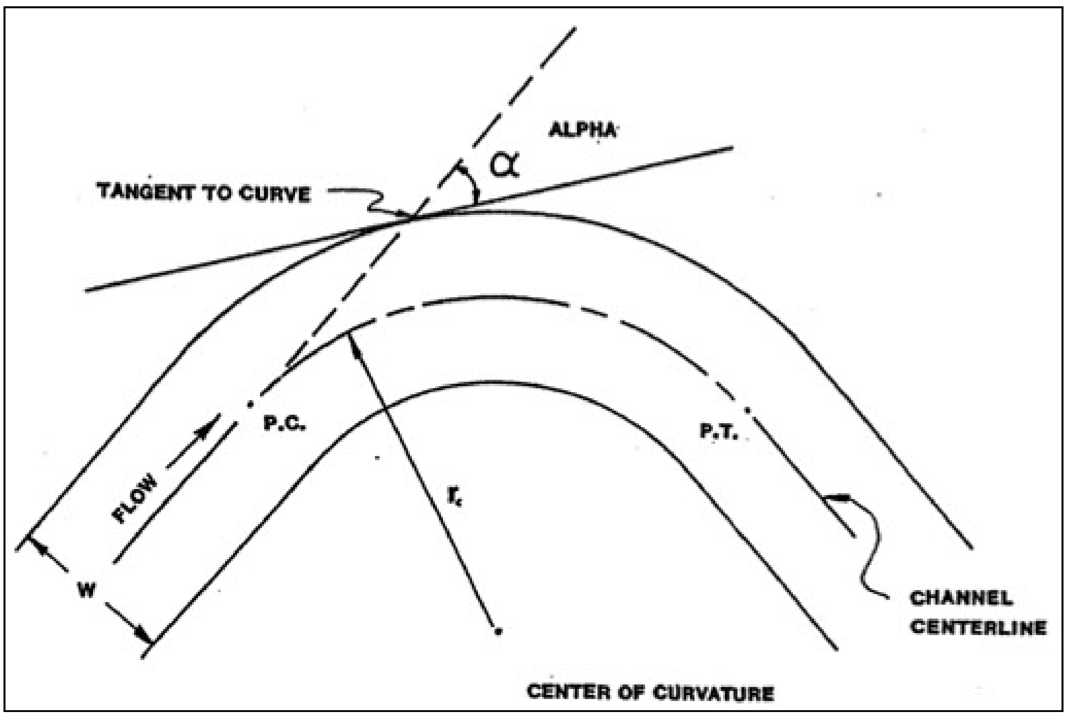

Figure A–3. Illustration of terminology for bend scour (Simons, Li & Associates, 1985, figure 5.25).

A.2.2 Maynord Bend Scour Equation

Bend scour calcuated using the Maynord (1996) methods is:

$$y_{mxb} = y_u * \left(1.8 - 0.051\left(\frac{r_c}{W_u}\right) + 0.0084(W_u/y_u)\right) \quad (15.9)$$

$$y_{bx} = y_{mxb} - y_u \quad (15.10)$$

Where:

y_{mxb}	=	Maximum water depth in the bend (ft)
y_u	=	Average water depth in the crossing upstream of bend (ft)
y_{bs}	=	Depth of bend scour below thalweg (ft)
r_c	=	Centerline radius of bend (ft)
W_u	=	Water surface width at upstream end of bend (ft)

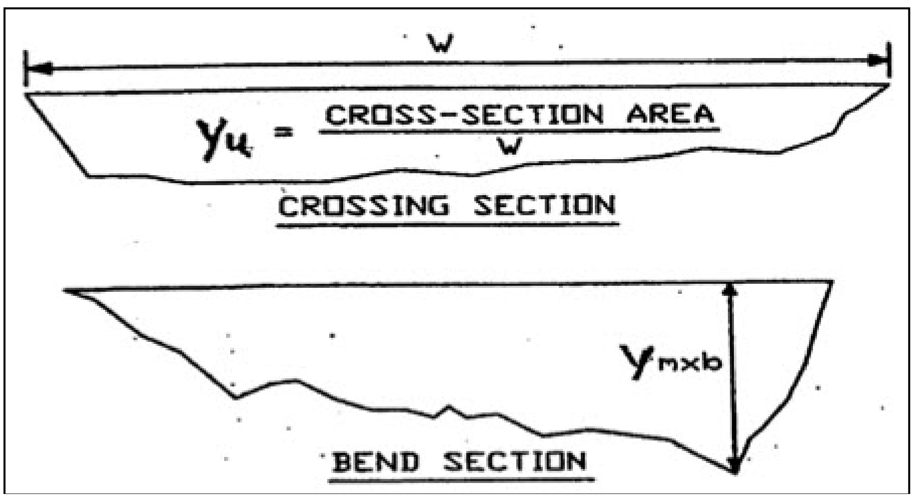

Figure A–4. Definition sketch for bend scour (Maynord, 1996).

Notes:

No safety factor incorporated into this equation. A safety factor of 1.08 is recommended.

The equation is limited to: $r_c / W_u < 10$ and $W_u / y_u < 125$
If $r_c / W_u < 10$, then $r_c / W_u = 1.5$ recommended
If $W_u / y_u < 125$, then $r_c / W_u = 20$ recommended

Not recommended where there is significant overbank flow. This method is limited to overbank depths of less than 20 percent of main channel depth.

A.2.3 Thorne Bend Scour Equation

Bend scour calculated using the Thorne, et al. (1985) method is:

$$y_{max} / y_u = 2.07 - 0.19 * \log_{10}(r_c / W_u - 2) \tag{15.11}$$

$$y_{bs} = y_{max} - y_u \tag{15.12}$$

Where:

y_{max}	=	Maximum water depth in the bend (ft)
y_u	=	Average depth in the crossing upstream of bend (ft)
y_{bs}	=	Depth of bend scour below thalweg (ft)
r_c	=	Centerline radius of bend (ft)
W_u	=	Water surface width at upstream end of bend (ft)

This equation is limited to $r_c / W_u > 2$.

A.2.4 Corps of Engineers Bend Scour

Scour in bends can be determined from design curves found in Plate B-42 of the USACE manual (USACE, 1994b). These are designated as safe design curves because they fall on the conservative side of the data. They are based on the ratio of maximum water depth in the bend to the mean water depth in the approach channel. Note that the maximum depth in the bend ranges from about 1.5 to 3.5 times the mean depth in the approach channel. Figure A–5 represents the upper limit for channels with irregular alignment – use 10-percent reduction from the bend scour design curve for relatively smooth alignment.

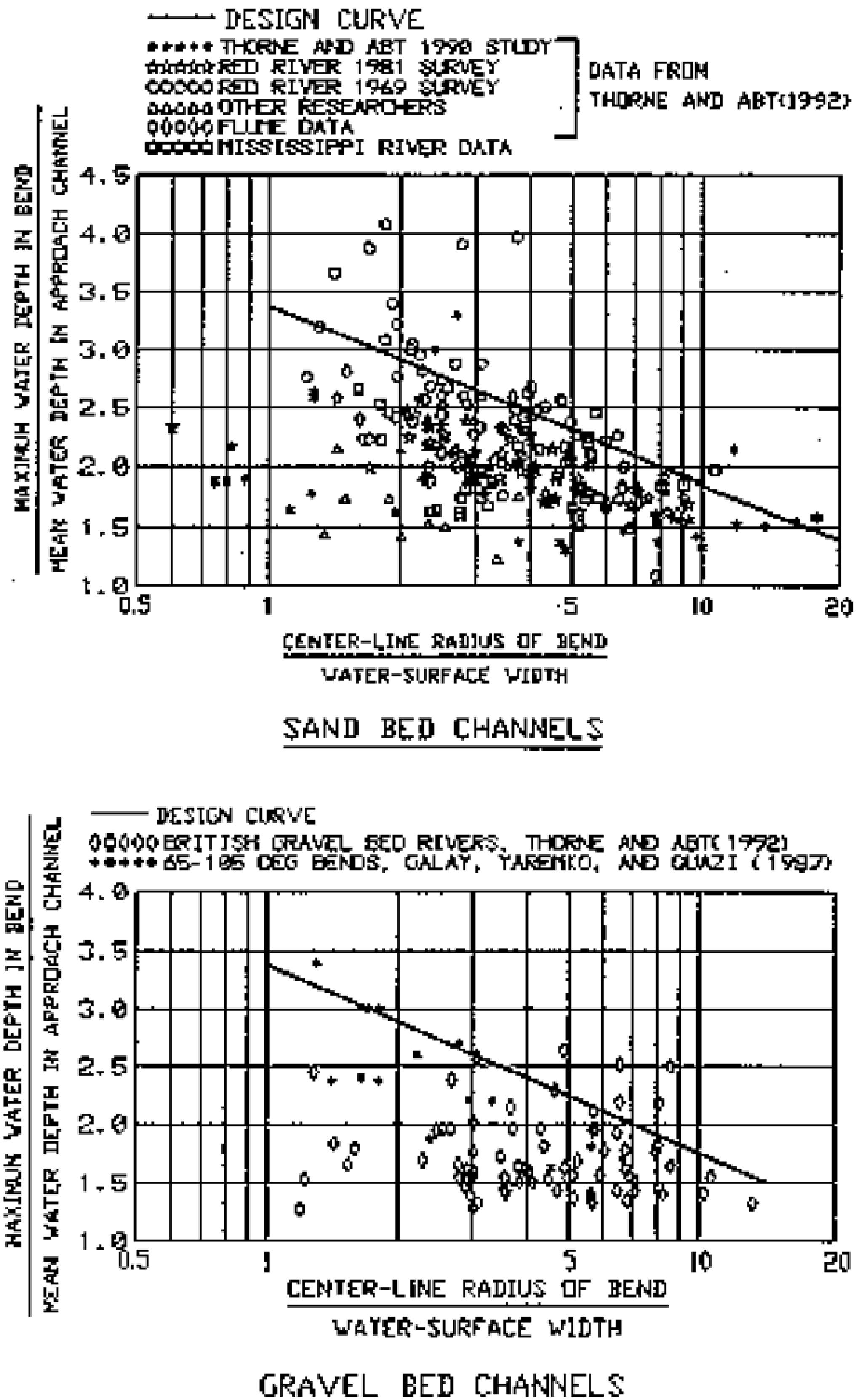

Figure A–5. Scour depth in bends (USACE, 1994; plate B-41).

A.3 Confluence Scour Equation

At the confluence of a tributary channel, the Ashmore and Parker (1983) method can be used for both sand and gravel sizes:

$$y_{mcf} / y_u = 2.24 + 0.031\,\theta \qquad (15.13)$$

$$y_{cfs} = y_{mcf} - y_u \qquad (15.14)$$

Where:

y_{mcf}	=	Maximum water depth at the confluence (ft)
y_u	=	Mean anabranch flow depth for converging channels (ft)
y_{cfs}	=	Depth of confluence scour below thalweg (ft)
θ	=	Angle formed by two converging anabranches (degrees)

Notes: Function is specified for θ of 30 to 90 degrees, and sand and gravel beds. Less scour results in fine sand or cohesive bed material.

A.4 Scour Equations for Near-Structure Locations

A.4.1 Contraction Scour – Modified Laursen's Live-Bed Equation

Shown below is the modified version of Laursen's live-bed equation (Laursen,1960), as presented in HEC-18 (FHWA, 2012):

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_1} \qquad (15.15)$$

Where:

y_s	=	$y_2 - y_0$
y_1	=	Average depth in the upstream main channel (ft)
y_2	=	Average depth in the contracted section (ft)
y_0	=	Existing depth in the contracted section before scour (ft) (see note 7 on the following page)
Q_1	=	Flow in the upstream channel transporting sediment (ft^3/s)
Q_2	=	Flow in the contracted channel (ft^3/s)
W_1	=	Bottom width of the upstream main channel that is transporting bed material (ft)
W_2	=	Bottom width of main channel in contracted section less pier width(s) (ft)
k_1	=	Correction factor for the mode of bed material transport from table A-1.

Table A–1. Correction Factor, k_1, for Mode of Bed Material Transport

U^*/ω	k_1	Mode of bed material transport
< 0.50	0.59	Mostly contact bed material discharge
0.50 to 2.0	0.64	Some suspended bed material discharge
>2.0	0.69	Mostly suspended bed material discharge

Where:

U^* = $\boldsymbol{\tau_0/\rho} = \sqrt{g y_1 S_1}^{\,2}$, shear velocity in the upstream section (ft/s)

ω = Fall velocity of bed material based on the D_{50} (figure A-6)

g = Acceleration of gravity (32.2 ft^2/s)

S_1 = Slope of energy grade line of main channel (ft/ft)

τ_o = Shear stress on the bed (lb./ft^2)

ρ = Density of water (1.94 slugs/ft^3)

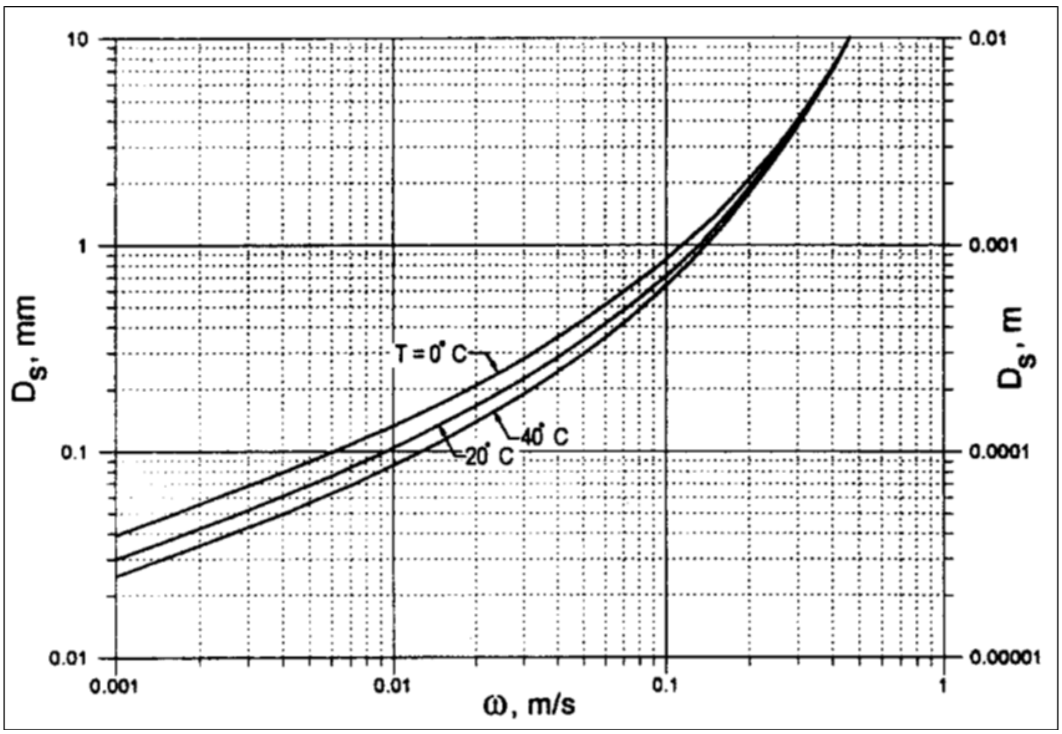

Figure A–6. Fall velocity of sand-sized particles with specific gravity of 2.65 in metric units.

Notes:

1. When there is overbank flow on a flood plain being forced back to the main channel by the approaches to the bridge. There are three potential cases:
   a. The river channel width becomes narrower, either due to the bridge abutments projecting into the channel or the bridge being located at a narrowing reach of the river;
   b. No contraction of the main channel, but the overbank flow area is completely obstructed by an embankment; or
   c. Abutments are set back from the stream channel. See HEC-18 (FHWA, 2012) for more information on Case 1 and Cases 2-4.
2. Q_2 may be the total flow going through the bridge opening, as in case a and b. It is not the total flow for case c. For case 1c, contraction scour must be computed separately for the main channel and the left and/or right overbank areas.
3. Q_1 is the flow in the main channel upstream of the bridge, not including overbank flows.
4. The Manning n ratio can be eliminated in Laursen live-bed equation as explained here. The ratio can be significant for a condition of dune bed in the upstream channel and a corresponding plane bed, washed out dunes, or antidunes in the contracted channel. However, Laursen's equation does not correctly account for the increase in transport that will occur as the result of the bed planning out (which decreases resistance to flow, increases the velocity, and increases the transport of bed material at the bridge). That is, Laursen's equation indicates a decrease in scour for this case; whereas in reality, there would be an increase in scour depth. In addition, at floodflows, a plane bedform will usually exist upstream and through the bridge waterway, and the values of Manning n will be equal. Consequently, the n value ratio is not recommended or presented in the equation.
5. W_1 and W_2 are not always easily defined. In some cases, it is acceptable to use the top width of the main channel to define these widths. Whether top width or bottom width is used, it is important to be consistent, so that W_1 and W_2 refer to either bottom widths or top widths.
6. The average width of the bridge opening (W_2) is normally taken as the bottom width, with the width of the piers subtracted.
7. Laursen's equation will overestimate the depth of scour at the bridge if the bridge is located at the upstream end of a natural contraction, or if the contraction is the result of the bridge abutments and piers. At this time, however, it is the best equation available.
8. In sand channel streams where the contraction scour hole is filled in on the falling stage, the y_0 depth may be approximated by y_1. Sketches or surveys through the bridge can help in determining the existing bed elevation.

9. Scour depths with live-bed contraction scour may be limited by coarse sediments in the bed material armoring the bed. Where coarse sediments are present, it is recommended that scour depths be calculated for live-bed scour conditions using the clear-water scour equation (given in HEC-18 [FHWA, 2012]), in addition to the live-bed equation, and that the smaller calculated scour depth be used.
10. See FHWA, 2012 for example problems and for information on adjusted approaches for cohesive soils, erodible rock, open bottom culverts, and pressure flow at bridges.

A.4.2 Local Scour at a Pier – CSU Equation

The CSU local pier scour equation developed by Richardson et al., (1990), as reported by FHWA, (2012) is:

The HEC-18 approach, based on the Colorado State University (CSU) equation, predicts a maximum scour depth for alluvial sand bed streams and is used for both live-bed and clear water conditions.
The HEC-18 equation is:

$$\frac{y_s}{y_1} = 2\,K_1\,K_2\,K_3\,\left(\frac{a}{y_1}\right)^{0.65} Fr_1^{\,0.43} \qquad (15.16)$$

As a rule of thumb, the maximum scour depth for round nose piers aligned with the flow is:

$y_s \leq 2.4$ *times the pier width (a)* for $Fr_1 \leq 0.8$
$y_s \leq 3.0$ *times the pier width (a)* for $Fr_1 > 0.8$ (15.17)

In terms of y_s/a, Equation 15-16 is:

$$\frac{y_s}{a} = 2\,K_1\,K_2\,K_3\,\left(\frac{y_1}{a}\right)^{0.35} Fr_1^{\,0.43} \qquad (15.18)$$

Where:

y_s = Scour depth (ft)
y_1 = flow depth directly upstream of the pier (approach flow depth) (ft)
K_1 = Correction factor for pier nose shape from table A-2.
K_2 = Correction factor for angle of attack of flow from table A-3 or equation 15-19.
K_3 = Correction factor for bed condition from table A-4.
A = Pier width (ft)
L = Length of pier (ft)
Fr_1 = Froude number directly upstream of the pier = $V_1/(gy)^{1/2}$
V_1 = Mean velocity of flow directly upstream of the pier (ft/s)
g = Acceleration of gravity (32.2 ft/s^2)

Table A–2. Correction Factor, K_1, for Pier Nose Shape

Shape of pier nose	K_1
Square nose	1.1
Round nose	1.0
Circular cylinder	1.0
Group of cylinders	1.0
Sharp nose	0.9

The correction factor, K_2, for angle of attack of the flow, a, is calculated using the following equation:

$$K_2 = (Cos\ \alpha + \frac{L}{a}\ Sin\ \alpha)^{0.65} \tag{15.19}$$

Where:

a = Skew angle of flow with respect to the pier

If *L/a* is larger than 12, use *L/a* = 12 as a maximum in equation 15-19 and table A–3. Table A–3 illustrates the magnitude of the effect of the angle of attack, a, on local pier scour.

Table A–3. Correction Factor, K_2, for Angle of Attack, *a*, of the Flow

Angle, α	*L/a* = 4	*L/a* =8	*L/a* = 12
0	1	1	1
15	1.5	2	2.5
30	2	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5

The correction factor, K_3, accounts for the effects of bedforms and bedform troughs where *H* is defined as the bedform height.

Table A–4. Correction Factor, K_3, for Bed Condition

Bed condition (bedforms)	Dune height, *H*, ft	K_3
Clear water scour	NA	1.1
Plane bed and anti-dune flow	NA	1.1
Small dunes	10 > H > 2	1.1
Medium dunes	30 > H >or= 10	1.2 to 1.1
Large dunes	H > or = 30	1.3

Notes from HEC-18 (FHWA, 2012):

1. The correction factor K_1 for pier nose shape should be determined using table A–2 for angles of attack up to 5 degrees. For greater angles, K_2 dominates and K_1 should be considered as 1.0. If L/a is larger than 12, use the values for L/a = 12 as a maximum in table A–3 and Equation 15-18.

2. The values of the correction factor K_2 should be applied only when the field conditions are such that the entire length of the pier is subjected to the angle of attack of the flow. Using this factor will significantly over predict scour if: (1) a portion of the pier is shielded from the direct impingement of the flow by an abutment or another pier; or (2) an abutment or another pier redirects the flow in a direction parallel to the pier. For such cases, judgment must be exercised to reduce the value of the K_2 factor by selecting the effective length of the pier actually subjected to the angle of attack of the flow. Equation 15-18 should be used for evaluation and design. Table A–3 is intended to illustrate the importance of angle of attack in pier scour computations and to establish a cutoff point for K_2 (i.e., a maximum value of 5.0).

3. The correction factor K_3 results from the fact that for plane-bed conditions, which are typical of most bridge sites for the flood frequencies employed in scour design, the maximum scour may be 10 percent greater than computed with equation 15-17. In the unusual situation where a dune bed configuration with large dunes exists at a site during flood flow, the maximum pier scour may be 30 percent greater than the predicted equation value. This may occur on very large rivers, such as the Mississippi. For smaller streams that have a dune bed configuration at flood flow, the dunes will be smaller, and the maximum scour may be only 10 to 20 percent larger than equilibrium scour. For anti-dune bed configuration, the maximum scour depth may be 10 percent greater than the computed equilibrium pier scour depth.

4. Piers set close to abutments (for example, at the toe of a spill through abutment) must be carefully evaluated for the angle of attack and velocity of the flow coming around the abutment.

5. See HEC-18 (FHWA, 2012) for information on the treatment of pier groups, wide piers, complex pier foundations, multiple skewed columns, scour debris, hole top widths, coarse bed materials, cohesive bed materials, and erodible rock.

A.4.3 Abutment Scour

Froehlich Equation

Froehlich (Transportation Research Board, 1989; FHWA, 2012) analyzed 170 live-bed scour measurements in laboratory flumes by regression analysis to obtain the following equation:

$$\frac{y_s}{y_a} = 2.27\, K_{a1}\, K_{a2}\, \left(\frac{L'}{y_a}\right)^{0.43} Fr_1^{\;0.61} + 1 \tag{15.20}$$

Where:

y_s = Scour depth (ft)
y_a = Average depth of flow on the flood plain (A_e/L) (ft)
A_e = Flow area of the approach cross section obstructed by the embankment (ft^2)
L_a = Length of embankment projected normal to the flow (ft)
K_{a1} = Coefficient for abutment shape (table A–5 and figure A–7)
K_{a2} = Coefficient for angle of embankment to flow, θ
L' = Length of active flow obstructed by the embankment (ft). Length of blockage of ineffective flow is subtracted from total length of embankment. If the flow in a significant portion of the cross section has low velocity and/or is shallow, then the length of embankment blocking this flow should not be used. One-dimensional flow models including SRH-1D (Huang, J., and B. Greimann, 2013) and HEC-RAS (USACE, 2010b) can easily compute conveyance versus distance across a cross section. See HEC-18 (FHWA, 2012) for additional guidance on estimating L′.
Fr_1 = Froude number of approach flow upstream of the abutment = $V_e/(gy_a)^{1/2}$
V_e = Q_e/A_e (ft/s)
Q_e = Flow obstructed by the abutment and approach embankment (ft^3/s)
g = Acceleration of gravity (32.2 ft/s^2)

Table A–5. Abutment Correction Factor K_1 for Shape of Opening

Description	K_{a1}
Vertical wall abutment	1.00
Vertical wall with wing walls	0.82
Spill-through abutment	0.55

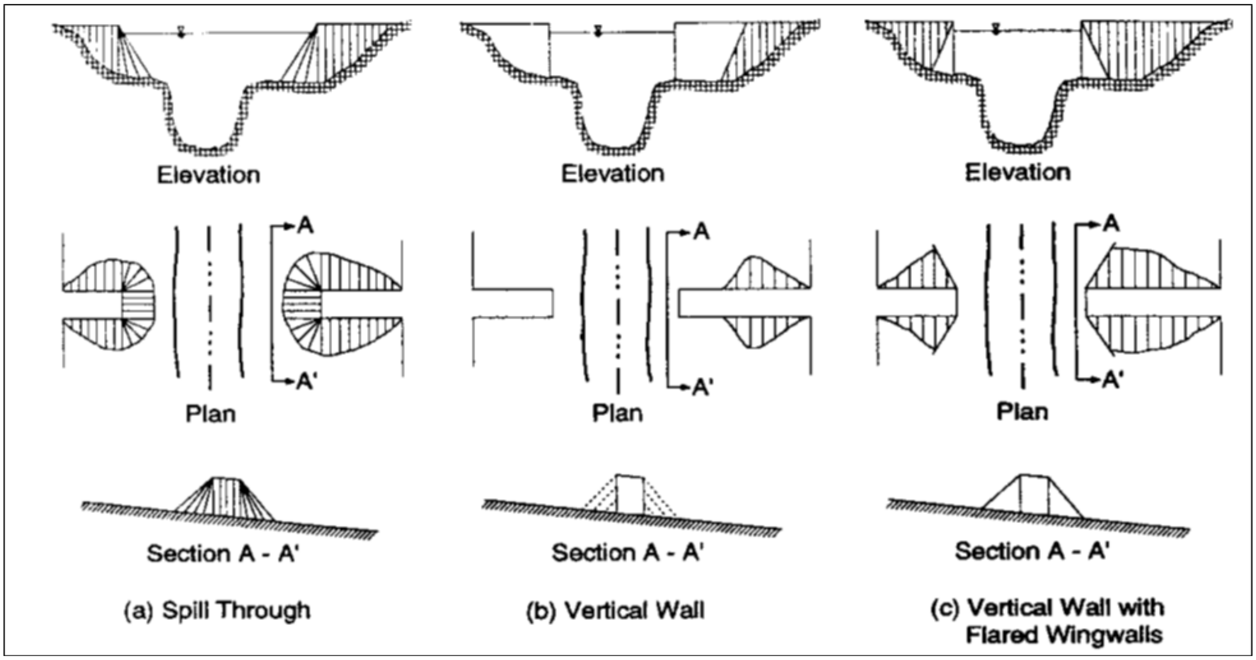

Figure A–7. Categories of abutment shape (FHWA, 2012).

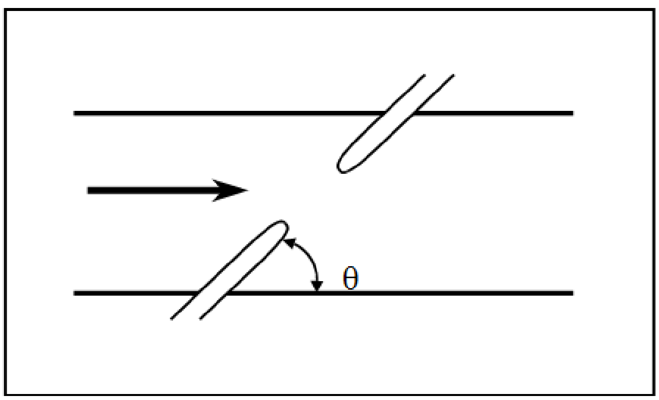

Figure A–8. Orientation of abutment embankment angle to the flow (FHWA, 2012).

$$K_{a2} = \left(\frac{\theta}{90}\right)^{0.13} \tag{15.21}$$

Where:

θ < 90 degrees if the embankment points downstream (figure A-8),
θ > 90 degrees if the embankment points upstream

It should be noted that equation 15.20 is not consistent with the fact that as L' tends to 0, y_S also tends to 0. The 1 was added to the equation to envelope 98 percent of the data.

HIRE Abutment Scour Equation

As presented in FHWA (2012) and FHWA (2001) the HIRE equation for abutment scour is:

$$\frac{y_s}{y_1} = 4\, Fr_1^{\,0.33}\ \ \frac{K_{a1}\, K_{a2}}{55} \tag{15.22}$$

Where:

y_s = Scour depth (ft)
y_1 = Flow depth directly upstream of the abutment (approach flow depth) on the overbank or in the main channel (ft)
K_{a1} = Correction factor for abutment shape from table A–5
K_{a2} = Correction factor for skew angle of abutment to flow calculated as shown in figure A-7.
Fr_1 = Froude number directly upstream of the abutment = $V_1/(gy)^{1/2}$
V_1 = Mean velocity of flow directly upstream of the abutment (ft/s)
g = Acceleration of gravity (32.2 ft^2/s)

Based on USACE field data from spurs in the Mississippi River, this equation is applicable when:

$$L/y_1 > 25 \tag{15.23}$$

Where:

L_a = Abutment length (ft)
y_1 = Flow depth upstream of the abutment (ft)

NCHRP 24-20 Abutment Scour Approach

The NCHRP 24-20 equation (FHWA, 2012; and NCHRP, 2010b) is based on a contraction scour estimate. Contraction scour is multiplied by a factor to account for large-scale turbulence adjacent to the abutment. Flow is more concentrated in the vicinity of the abutment, and the contraction scour component is larger than average conditions in the constricted opening (FHWA, 2012). The three scour conditions (figure A-9) are:

1. Scour occurring when the abutment is in (or close to) the main channel
2. Scour occurring when the abutment is set back from the main channel
3. Scour occurring when the embankment breaches and the abutment foundation acts as a pier.

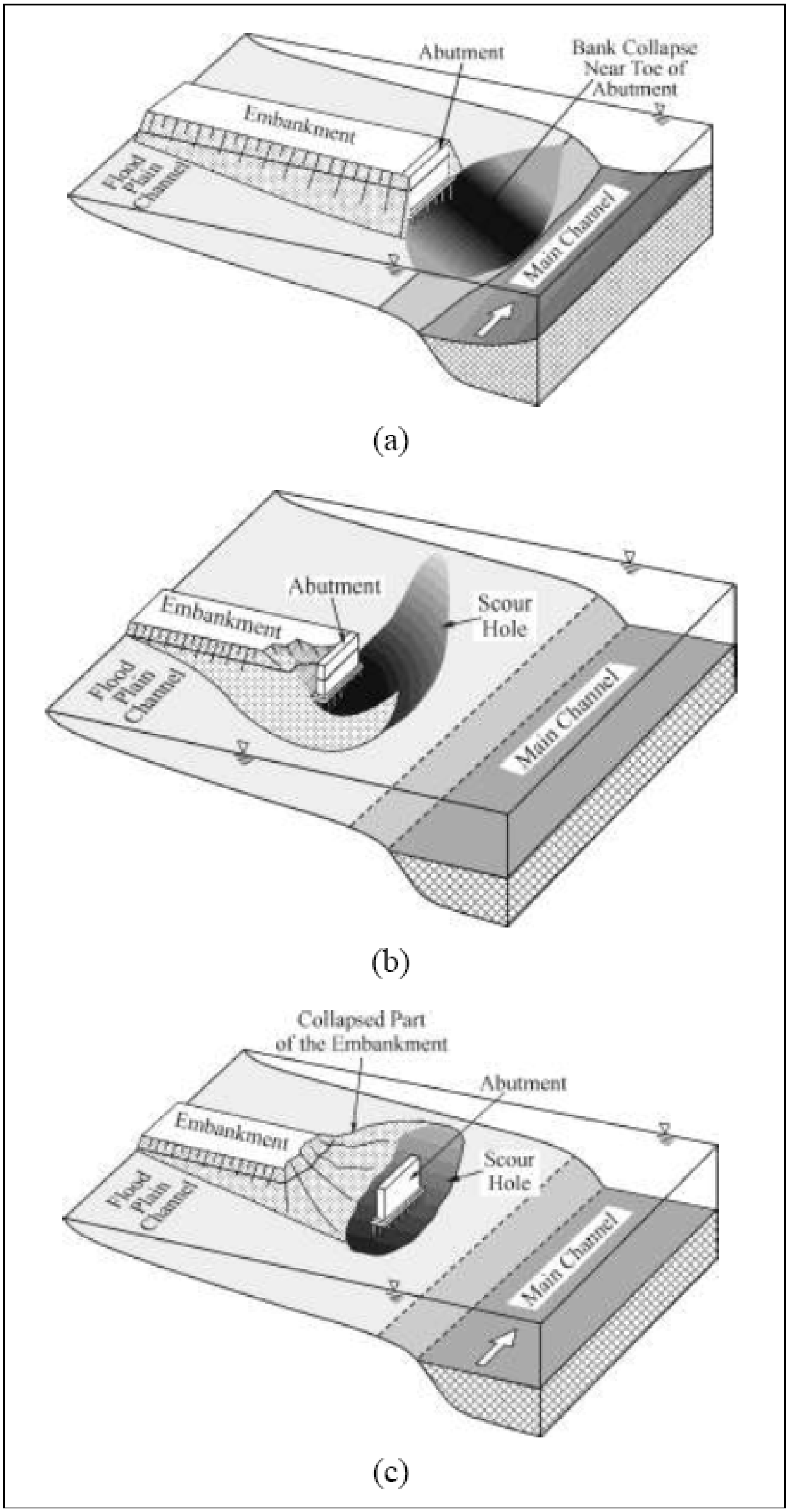

Figure A–9. Abutment scour conditions (NCHRP 2010).

The NCHRP 20-24 approach assumes that there is a limiting depth of abutment scour when the geotechnical stability of the embankment or channel bank is reached. The equation gives a total scour depth that includes contraction scour effects. Contraction scour should not be added separately when using this equation. Three advantages to using this equation are noted in HEC-18:

1. Effective embankment length, *L'*, which can be difficult to determine, is not used in these computations.
2. Equations are more physically representative of the abutment scour process.
3. Contraction scour is included and does not need to be computed separately.

Scour equations for conditions a and b are:

$$y_{max} = \alpha_a y_c$$
or (15.24)
$$y_{max} = \alpha_b y_c$$

$$y_s = y_{max} - y_o \qquad (15.25)$$

Where:

y_{max}	=	Maximum flow depth resulting from abutment scour (ft)
y_c	=	Flow depth including live-bed or clear-water contraction scour (ft)
α_a	=	Amplification factor for live-bed conditions
α_b	=	Amplification factor for clear water conditions
y_s	=	Abutment scour depth (ft)
y_o	=	Flow depth prior to scour (ft)

Condition A

If $L_a \geq 0.75B_1$, then Condition A and the contraction scour calculation are performed using a live-bed scour calculation.

Where:

L_a = Abutment length (ft)
B_1 = Width of the flood plain (ft)

The contraction scour equation is:

$$y_c = y_1 \left(\frac{q_{2c}}{q_1}\right)^{6/7} \qquad (15.26)$$

Where:

y_c = Flow depth including live-bed contraction scour (ft)
y_1 = Upstream flow depth (ft)

q_1 = Upstream unit discharge (ft^2/s)
q_{2c} = Unit discharge in the constricted opening accounting for non-uniform flow distribution (ft^2/s)

Unit discharge can be estimated either by discharge, Q, divided by width, w, or by the product of velocity and depth, $v \times y$.

Condition B

If $L_a < 0.75B_1$, then Condition B and the contraction scour calculation are performed using a clear water scour calculation. Two clear water contraction scour equations can be used. The first equation is the standard equation based on grain size:

$$y_c = \left(\frac{q_{2f}}{K_u {D_{50}}^{1/3}}\right)^{6/7} \quad (15.27)$$

Where:
y_c = Flow depth including clear water contraction scour (ft)
q_{2f} = Unit discharge in the constricted opening accounting for non-uniform flow distribution (ft^2/s)
K_u = 11.17, English units
K_u = 6.19, International System of Units (SI units)
D_{50} = Particle size with 50 percent finer (ft)

A lower limit of particle size of 0.2 mm is reasonable because cohesive properties limit the critical velocity and shear stress for cohesive soils. If the critical shear stress is known for a flood plain soil, then an alternative clear water scour equation can be used:

$$y_c = \left(\frac{\gamma}{\tau_c}\right)^{3/7} \left(\frac{n q_{2f}}{K_u}\right)^{6/7} \quad (15.28)$$

Where:
n = Manning n of the flood plain material under the bridge (ft)
τ_c = Critical shear stress for the flood plain material (lb/ft^2)
γ = Unit weight of water (lb/ft^3)
K_u = 1.486, English units
K_u = 1.0, SI units

Notes:

The recommended procedure for selecting the velocity and unit discharge for abutment scour calculation is to use two-dimensional modeling. If one-dimensional modeling is used, velocity and unit discharge are estimated as presented in FHWA (2012).

The value of αa is selected from figure A-10 for spill through abutments and αb figure A-11 for wing-wall abutments. The solid curves should be used for design. The dashed curves represent theoretical conditions that have yet to be proven experimentally.

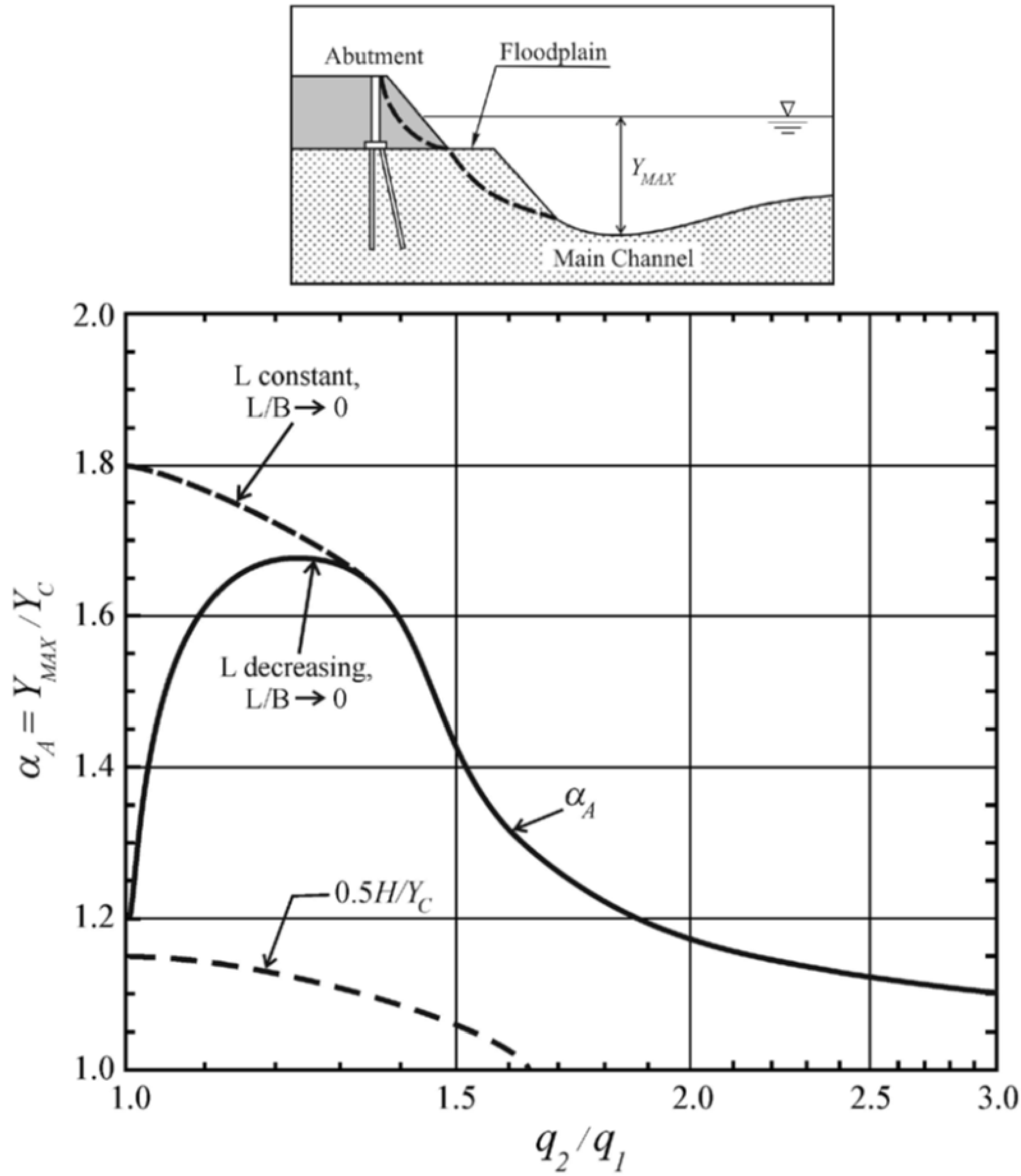

Figure A–10. Scour amplification factor for spill-through abutments and live-bed conditions (NCHRP 2010).

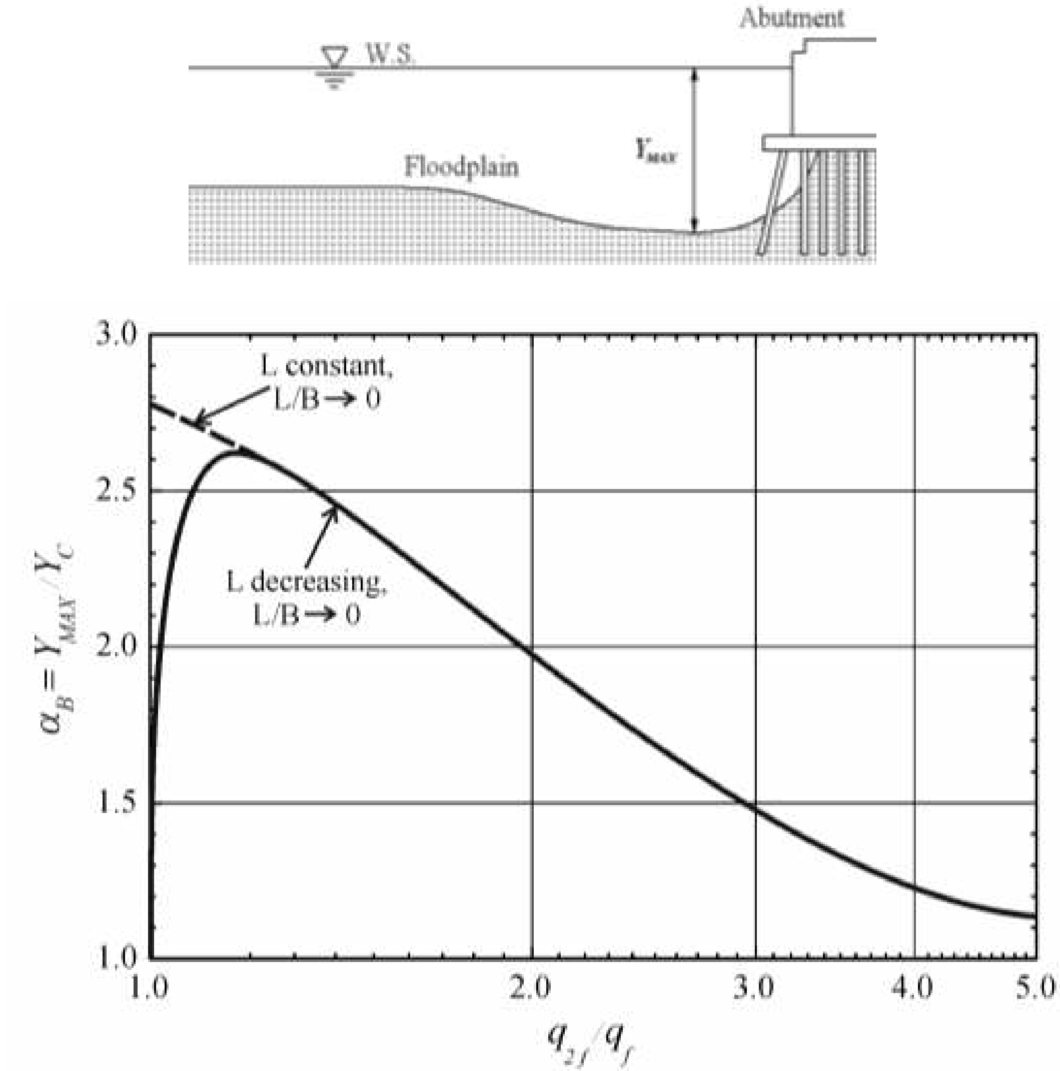

Figure A–11. Scour amplification factor for wing-wall abutments and live-bed conditions (NCHRP 2010).

Design curve for short-contraction, scour-amplification factor, α_b, for wing-wall abutments subject to Scour Condition B (abutment set back on a wide floodplain)

For scour estimates determined for either condition (a) or (b), the geotechnical stability of the channel bank or embankment should be considered. If the channel bank or embankment is likely to fail, then the limiting scour depth is the geotechnically stable depth, and erosion will progress laterally. This may cause the embankment to breach, and another scour estimate can be performed treating the abutment foundation as pier.