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Coastal and Hydraulics Laboratory

US Army Corps
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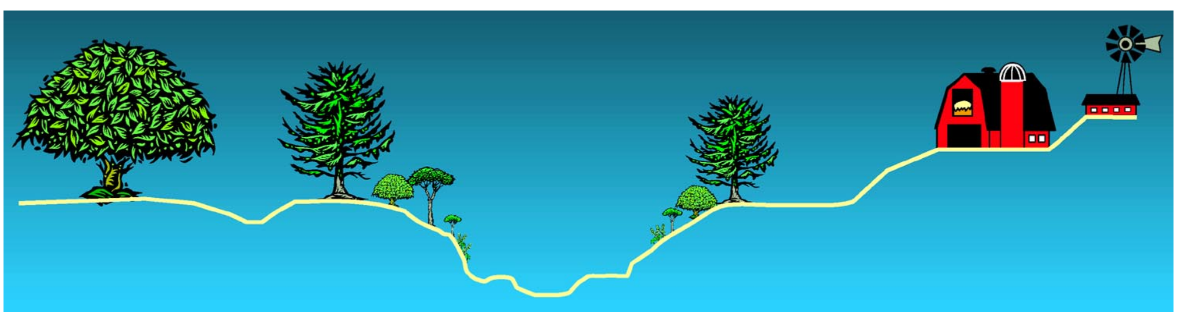
Development Center

Hydraulic Design of Stream Restoration Projects

Ronald R. Copeland, Dinah N. McComas, Colin R. Thorne,
Philip J. Soar, Meg M. Jonas, and Jon B. Fripp

September 2001



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Preface

The work described in this report was authorized by Headquarters, U.S. Army Corps of Engineers (HQUSACE) as part of the Flood Damage Reduction Research Program. The guidelines developed herein are products of Work Unit 32878 "Channel Restoration Design." The Program Monitor was Mr. Richard J. DiBuono, HQUSACE. The Program Manager was Ms. Carolyn Holmes, Coastal and Hydraulics Laboratory (CHL), U.S. Army Engineer Research and Development Center (ERDC). The principal investigator for the work unit was Dr. Ronald R. Copeland, CHL.

The report was prepared by Dr. Ronald R. Copeland, CHL, Ms. Dinah N. McComas, CHL, Drs. Colin R. Thorne and Philip Soar, University of Nottingham, and Ms. Meg Jonas and Mr. Jon Fripp, U.S. Army Engineer District, Baltimore. The U.S. Army Corps of Engineer Committee on Channel Stabilization and Dr. David Biedenharn, CHL, made valuable contributions and provided review and comment. Committee on Channel Stabilization members include: Dr. Ronald R. Copeland, CHL, Chairman; Ms. Dinah N. McComas, CHL, Secretary; Mr. Larry E. Banks, U.S. Army Engineer District, Vicksburg; Mr. William E. Branch, U.S. Army Engineer Division, Northwest; Dr. Craig J. Fischenich, Environmental Laboratory, ERDC; Ms. Meg M. Jonas, U.S. Army Engineer District, Baltimore; Mr. Thomas J. Pokrefke, CHL; Mr. John I. Remus, U.S. Army Engineer District, Omaha; Mr. Edward F. Sing, U.S. Army Engineer Division, South Pacific; Mr. Michael F. Spoor, U.S. Army Engineer District, Huntington; Mr. Scott E. Stonestreet, U.S. Army Engineer District, Sacramento; and Mr. Howard M. Whittington, U.S. Army Engineer District, Mobile. Technical Review was also provided by Mr. Doug Otto, U.S. Army Engineer District, Mobile; Mr. Pat Foley, U.S. Army Engineer District, St. Paul; and Mr. Rene Vermeeren, Mr. Dave Cozakos and Mr. Nick Adelmeyer, U.S. Army Engineer District, Los Angeles.

The study was performed under the supervision of Dr. Yen-Hsi Chu, Chief of the River Sedimentation Branch, CHL, Mr. Thomas J. Pokrefke, Acting Deputy Director of CHL, and Mr. Thomas W. Richardson, Acting Director of CHL.

At the time of publication of this report, Dr. James R. Houston was Director of ERDC, and COL John W. Morris III, EN, was Commander and Executive Director.

1 Introduction

Purpose

The purpose of this document is to provide a systematic hydraulic design methodology to hydraulic engineers involved in stream restoration projects. The objective of the methodology is to fit the stream restoration project into the natural system within the physical constraints imposed by other project objectives and constraints.

The hydraulic design of a stream restoration project should provide for a channel that is in dynamic equilibrium with its sediment load. A stable channel is defined as a channel where the planform, cross section, and longitudinal profile are sustainable over time. The channel may migrate and still be stable, although channel migration may not always be acceptable due to project constraints. The magnitude of long-term aggradation and/or degradation in a stable channel should be small enough to allow for economical channel maintenance. The design methodology presented herein is systematic, i.e., when used by different engineers, with the same project objectives, design results should be similar. The methodology is based on sound physical principles and is applicable to both fixed and mobile boundary streams.

The systematic approach to hydraulic design of stream restoration projects presented herein has several components. Chapter 2 outlines defining project objectives and constraints. Chapter 3 provides an overview of how to determine hydrologic data that may be of importance in the hydraulic design process. Chapter 4 outlines the stability assessment methodologies that are important to establish baseline geomorphological conditions and to evaluate the effectiveness and geomorphological impacts of project alternatives. Chapter 5 presents the methodology for hydraulic design of project features and for assessing hydraulic and sediment transport impacts of alternatives. This systematic approach is iterative, as shown in Figure 1.

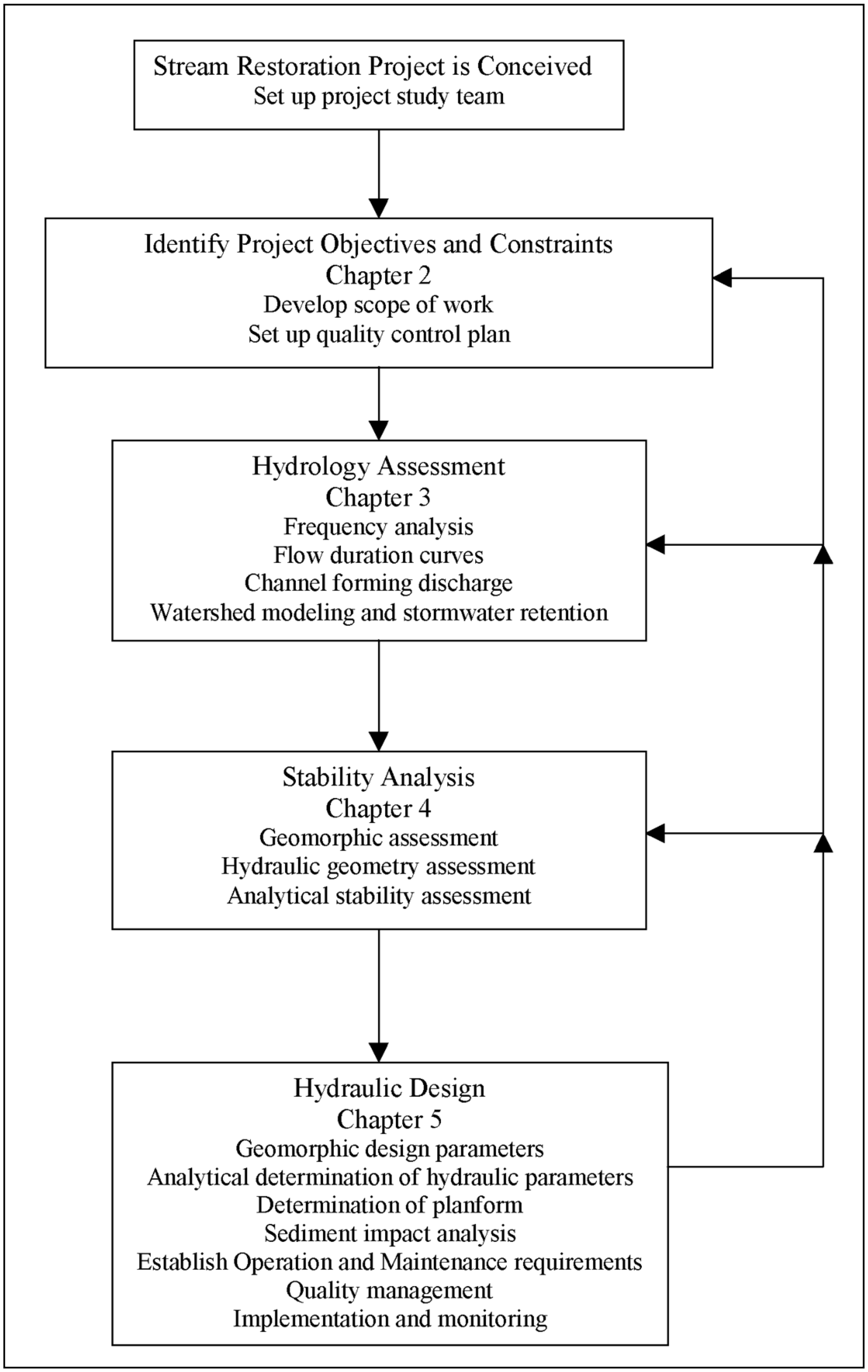

Figure 1. Flow chart for systematic approach to hydraulic design of stream restoration projects

Definition of a Stream Restoration Project

Stream restoration projects typically are intended to improve or restore environmental conditions in the stream and the adjacent stream corridor. Channel stability is often essential to the maintenance of favorable environmental conditions. Restoration projects do not necessarily require returning a system to some predisturbance condition, as this is seldom feasible. The objective of a restoration project is then a partial recovery of the natural geomorphic, hydraulic, and ecological functions of the stream. It follows that the project design team requires expertise in the fields of geomorphology, hydraulics, and ecology.

U.S. Army Corps of Engineers stream restoration projects are frequently associated with or are part of a flood-control project. Thus, projects have more than one objective and compromises may be required to meet essential portions of each objective. For example, it may not be possible to construct a channel that both carries the design flood flow and provides optimum transport of the upstream sediment load.

The scales and purposes of stream restoration projects vary significantly depending on project objectives. In general, stream restoration projects can have one or more of the following three general goals:

a. Enhance channel stability and thus reduce channel maintenance.

b. Improve the environment.

c. Provide aesthetic or recreation benefits.

Channel stability is usually addressed by providing bank protection and/or grade control. Typical examples that primarily address aspects of channel stability are provided in Figures 2 through 7. The projects in these figures combine traditional engineering protection methods with vegetation to provide for a stable stream. Other projects have been designed primarily to provide habitat enhancement. Figures 8 through 10 show features that were added to existing projects to provide better aquatic habitat. Figures 11 and 12 show a river restoration project in San Antonio, Texas where the primary objectives were aesthetics and recreation. Many stream restoration projects are multiobjective. These projects can be technically challenging but have widespread support since they address many concerns. Several examples of multiobjective projects are provided in Figures 13 through 15.

Figure 2. Bank erosion of vegetated bank, Johnson Creek, MS

Figure 3. Bank stabilization with stone toe protection, Johnson Creek, MS

Figure 4. Bank stabilized and revegetated bank, Johnson Creek, MS

Figure 5. Channel bed and bank erosion due to degradation, Hotophia Creek, MS

Figure 6. Channel stabilization with grade control, Hotophia Creek, MS

Figure 7. Stable channel upstream from grade control, Hotophia Creek, MS

Figure 8. Habitat enhancement with the use of low-flow stone weirs, Paint Branch, MD

Figure 9. Bank protection and habitat enhancement using stone lunkers, Rapid Creek, Rapid City, SD

Figure 10. Habitat enhancement features (root wads) added to flood-control project, Sammamish River, WA

Figure 11. Incised urban stream upstream from restoration reach, San Antonio River, San Antonio, TX

Figure 12. River restoration to provide aesthetics and recreation, San Antonio River, San Antonio, TX

Figure 13. Project designed for flood protection with habitat enhancement features boulder clusters and low-flow deflectors, Zumbro River, Rochester, MN

Figure 14. Project designed to provide flood protection, aesthetics, and habitat enhancement, South Platte River, Denver, CO

Figure 15. Project designed to provide flood protection, aesthetics, and recreation, Cherry Creek, Denver, CO

Design Philosophy

A wide variety of analysis techniques can be applied to channel design and stream restoration. A sound stream restoration design incorporates techniques from fluvial geomorphology, engineering and stream ecology. The study area to which these techniques are applied must extend beyond the limits of the project site to the extent that both the project's effect on the stream system and the stream system's effect on the project reach can be determined. This may require analysis of the entire watershed. Many of the principles and techniques found in EM 1110-2-1418, "Channel Stability Assessment for Flood Control Projects," are equally applicable to hydraulic design of stream restoration projects.

Fluvial geomorphology techniques provide insight relative to general responses of a river system to a variety of imposed changes. These techniques are important in analyzing the stability of the existing stream system and in identifying the source of instabilities. Fluvial geomorphology techniques also provide generalized guidance related to appropriate cross-section geometry and channel planform. It is important to recognize that the science of fluvial geomorphology utilizes both qualitative and quantitative analyses of observed features and channel forms. As a result, trends and changes in the fluvial system may be inferred from the history of channel evolution and its past response to human interventions. However, quantitative assessment and design, for a specific project area, requires use of physically-based calculations. These calculations are based on principles and techniques in hydrology, hydraulics, and sediment transport. These considerations demonstrate that the contributions of geomorphology, hydrology, hydraulics, and sediment transport are complementary, not alternatives.

Stream restoration design should consider a variety of flow conditions. Rarely does the behavior of a channel under a single discharge adequately reflect the range of design conditions required of a stream restoration project. So, during the design process, a range of discharges must be considered even though initial stream restoration design focuses on a single design discharge. Figure 16 illustrates a sketch of an idealized channel cross section and the various flow conditions that should be examined. In Figure 16, Q_{lfc} indicates the low-flow condition. The cross-sectional area at this discharge often defines the limiting biologic condition for aquatic organisms. Minimum depths are often specified as a design goal at this discharge. In gravel bed streams, it is often desirable to design the channel section at this flow level so that fine sediment does not deposit at this discharge, as this risks smothering the coarse bed and benthic organisms in a layer of sand and silt. Analysis and design for this area may involve incipient motion and threshold techniques. Q_{olw} indicates the ordinary low water or base flow condition. In many situations, low flow and base flow can be assumed to be the same. The cross-sectional perimeter at a flow of Q_{olw} may be further defined into a benthic zone (where species are attached to or buried in the substrate) and an aquatic zone (which includes organisms in the water). Habitat features are often designed for the cross-sectional area below Q_{olw}. While this area can be very active in a natural stream, it is often desirable to provide some stabilization if a large investment in habitat structures is to be made. Although the geometry of these habitat structures may be designed for stream conditions at a discharge of Q_{olw}, hydraulic parameters at a higher discharge should be used to determine the stability of the structure and/or requirement for bank stabilization. It is important to note that watershed land use changes, such as urbanization, can have a strong effect on the flows at this level.

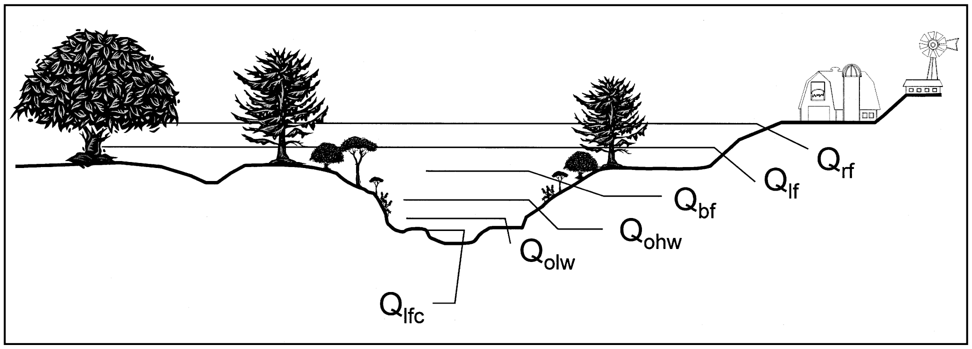

Figure 16. Idealized stream cross section

Q_{ohw} represents ordinary high water in Figure 16. Riparian vegetation typically begins between the ordinary low water and the ordinary high water. If needed, and if design conditions are met, the channel banks above the ordinary low-water elevation may be suitable for vegetative bank protection. Sediment transport typically becomes an issue at this flow, especially in alluvial channels.

Bankfull stage is typically defined at a point where the width to depth ratio is a minimum. In Figure 16, this is shown at Q_{bf}. In many cases, ordinary high water and bankfull stage are synonymous.

There are many geomorphic regime relationships that provide relationships between drainage area, discharge, and the cross-section geometry at bankfull stage. In many situations, the channel velocity begins to asymptotically approach a maximum at this stage. It has also been observed that, in some cases, lateral momentum losses can result in a decrease in channel velocity during rising stages as flow spills onto the floodplain. However, when the floodplain is narrow or heavily obstructed, channel velocities may continue to increase with rising stage. This phenomenon is illustrated in the three idealized cross sections in Figure 17. In Figure 17a, there is a large increase in available conveyance once the flows exceed the top of bank elevation. As a result, a large increase in flow produces only little or no increase in channel velocity. In this situation it may be appropriate to use the bankfull hydraulic conditions to design bank protection or to assess the stability of in-stream habitat structures. However, Figures 17b and 17c illustrate conditions where the overbank conveyance is limited by topography or vegetation. In these situations, the channel velocity can continue to increase with increased flow. As a result, it may be appropriate to use a significantly larger discharge to design channel features such as bank protection and habitat structures. While the difference between the different cross sections presented in Figure 17 is very obvious in the idealized sketches, it is often difficult to assess the hydraulic behavior of overbank flows in the field. Therefore, it is necessary that hydraulic and sediment transport calculations be used to assess channel and overbank conditions under a wide range of inbank and overbank flows.

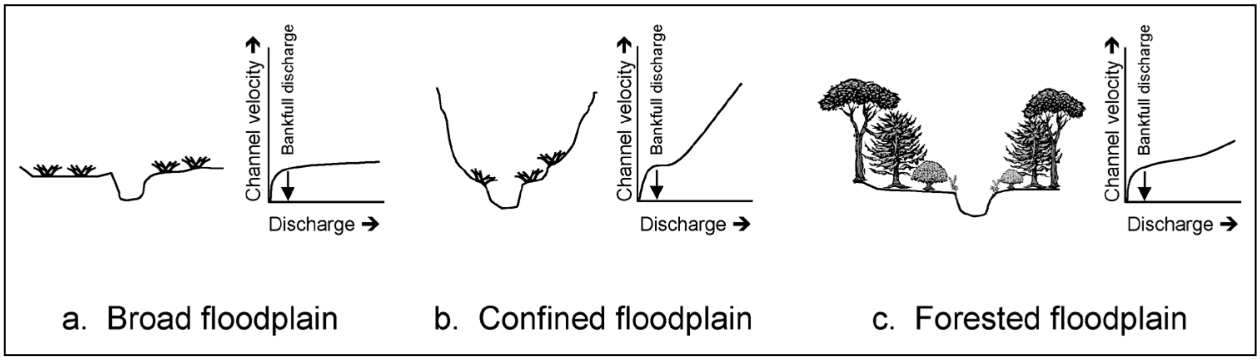

Figure 17. Channel velocity trends

The riparian zone is typically found above the bankfull stage. This area is favorable to plants and animals that inhabit land that is rarely submerged. Aquatic wetlands can also occur in this area. In Figure 16, Q_{lf} represents a large flow that floods the riparian zone. Flows at this stage and below can be adjusted through the use of stormwater management techniques. Sediment transport and continuity analysis should address these flows as well as lower discharges. If channel avulsions are a concern in the study reach, incipient motion analysis techniques and threshold methods may also be required for flows in this zone. If a project involves riparian plantings, it is advisable to assess the expected depths and velocities in the terrestrial zone with reference to the tolerances of the proposed species.

Some consideration should be given to extreme flood flows for stream restoration projects. In Figure 16, Q_{rf} represents a regulatory floodplain such as the 100-year flood. Some analysis should be done to assess the impact of the stream restoration project on flood elevations in the floodplain. Conventional stormwater management techniques are typically insufficient to affect flows at this level. The stability requirements for the features used in the lower parts of the channel are typically defined somewhere between Q_{lf} and Q_{rf}. In addition, sediment continuity continues to be an issue at these flows.

Project Study Teams

Since few people possess all skills necessary to conduct a successful stream restoration study and design, an interdisciplinary approach is required. This is especially the case for the establishment of objectives. While the exact makeup of the team can vary, it should include engineering, geomorphological, and ecological expertise.

Stream restoration projects differ from traditional hydraulic engineering projects in several ways. First, there is more potential variety in the goals. Second, there is less consensus on how the study should be conducted. Questions such as - what surveys and data collection are necessary, what analyses should be performed, what design techniques should be used, and what measures should be implemented, are frequently debated. Third, the success of the project may be judged as much by appearance as by function. And fourth, an interdisciplinary approach using geomorphologists, engineers, and biologists is essential to the success of the project. Since the study team will likely include geomorphologists, biologists, ecologists, landscape architects, and representatives of resource agencies as well as hydraulic engineers, it is important to define areas of responsibility clearly so that the team can function effectively. The hydraulic engineer performs all analyses and stable channel design calculations related to hydrology, hydraulics, water quality, and sedimentation. This may include geomorphic assessment, stream classification, assessment of watershed and channel stability, when the engineer has been trained in the relevant techniques in geomorphology.

This report provides guidelines for determining the types of hydraulic analyses that should be employed for a variety of stream restoration projects. Information is provided that describes setting objectives, data requirements, analytical techniques, limitations, and interpretations of possible results. The coverage provided is sufficient to the task of designing a restoration project with a stable channel capable of conveying the water and sediment supplied from upstream. However, no attempt has been made to include or evaluate all the approaches to channel analyses and design that are currently in use.

2 Project Objectives and Constraints

Project Objectives

As a result of increased public concern for the environment, many Federal, state, and local governments along with grassroots organizations are actively engaged in stream restoration. Engineers are being asked for analyses and designs that focus on restoring, establishing, or maintaining natural stream environments. The perceived success or failure of many stream restoration projects can be as much a function of the success criteria selected at the outset as of the design. Therefore, the importance of establishing achievable study objectives is critical. Once established, these objectives will define the type and amount of data collection, methodologies for assessments and designs, and condition of the design itself. Project objectives should be clearly stated in the scope of work or project management plan.

The first step in a stream restoration project, as with any engineering project, is to clearly define project objectives in cooperation with stakeholders. Is the objective to create an aesthetic setting, a natural setting, to enhance fish or wildlife habitat, to prevent bank erosion or channel degradation, or something else? All these objectives could conceivably be considered legitimate goals of a stream restoration project, though in practice it may be necessary to compromise on one objective in order to meet others.

In establishing objectives for a stream restoration project, it is advisable to assess at least the following six issues:

a. The existing condition of the stream and watershed.

b. The scale and severity of the resource loss or degradation due to stream instability.

c. Causal factors and controls that have resulted in the current stream condition. In this context it is useful to establish whether current instability in the channel is being driven by the current flow regime or is a product of past conditions.

d. The condition into which the channel is likely to evolve without a project. This often involves a strong reliance on geomorphic prediction coupled with engineering judgment. This question incorporates the issue of what keeps the stream reach from restoring itself.

e. The physical constraints on possible restoration measures such as water quality, available rights-of-way or construction area, as well as budget constraints.

f. The range of alternative solutions that are both feasible and acceptable to the stakeholders.

These issues are best addressed through a preliminary watershed reconnaissance, which is discussed in more detail in a later section.

Many studies initiate with the objective defined as simply "fixing" the stream. The generality of such an objective can lead to problems. Clearly defining the objectives offers a clear approach as well as reduces ambiguity for the study team members. Objectives should not only be specific but also be realistic and achievable.

Restoring a stream to a "natural" condition is also often used as an objective in many studies. There can be an attraction to defining this natural condition by aesthetic guidelines, but care must be taken to assure that it is appropriate for both the watershed type as well as the study constraints. It is also important to realize that a stream that is behaving "naturally" within the context of watershed conditions may still be detrimental to riparian land use and may not possess a desirable habitat. For example, a stream reach just above the confluence with a major river could be braided, aggrading, lacking pools, and exhibiting frequent out-of-bank flooding yet be behaving naturally. This reach could be altered to reduce flooding to force the stream into a single thread to enhance in-stream fish habitat for a particular fish type. However, since these changes would be altering the natural condition, it is important to recognize that considerably more engineering effort would be required over what would be required if the target conditions were more in keeping with the existing morphology of the stream.

Restoring streams to some historical condition or to a reference condition may be stated as an objective. If this is the approach, care must be taken to assure that physical or biological changes or differences in the watershed or watersheds do not prohibit a return to a historic or reference condition. The risk is that the historical channel morphology would be inappropriate to the modern watershed context, and thus would require heavy maintenance to sustain it. For example, the objective for an incised and widening stream in an urban watershed could be to restore it to provide habitat for a sensitive fish species that was present before development. Changes in runoff patterns, sediment load and water quality could make this objective impossible to achieve if the project focus is only on the study reach. Stormwater management, sediment trapping, and water quality enhancement may be required to restore the water quality and sediment regimes to predevelopment conditions. The use of historic geometric conditions as an objective can also be problematic. Using the same hypothetical urban stream as an example, the watershed conditions that caused the current stream condition may cause the channel to alter again unless additional stabilization measures are included in the restoration design.

Some stream restoration studies have objectives defined in terms of improving the habitat for a particular species. This can provide general design objectives such as target depths and velocities. One problem with this approach is that the needs of the target species may not be consistent with a channel morphology appropriate to supporting a wide biodiversity in a varied ecosystem. A better approach may be to make biodiversity optimization an objective if it is achievable for the project area.

When a stream restoration project is part of a local flood damage reduction project, design objectives are stated in ER 1110-2-1405. The hydraulic design of a local flood protection project must result in a safe, efficient, reliable, and cost-effective project with appropriate consideration of environmental and social aspects. Issues related to safety include potential hazards to humans and property, creation of a false sense of security, and consequences of flows exceeding the design channel capacity. Issues related to efficiency include both hydraulic conveyance and operation and maintenance. Issues related to reliability include achieving project purposes throughout the project economic life, and the proper functioning of appurtenances such as gates, weirs, deflectors, and bank stabilization. Cost-effectiveness includes both the initial project costs and operation, maintenance, and replacement costs. Channel restoration projects are often perceived to have significantly higher maintenance costs than traditional single objective flood-control projects. This is generally attributed to costs related to maintaining appropriate vegetation density on the overbanks. This cost is offset, however, in a properly designed restored channel that is self-sustaining in terms of sediment transport. Simple flood control channels that have widths and depths that are much larger than the stable regime channel dimensions suffer chronic sedimentation problems and tend to lose conveyance capacity due to invasive vegetation. Both problems require heavy maintenance costs. Environmental and social aspects include fish and wildlife habitats, aesthetics, recreational opportunities, handicap access, and mitigation of adverse impacts. ER 1110-2-1405 lays out the project hydraulic design process and the format for hydraulic design reporting.

In general, the engineering means to achieve the objectives of stream restoration can be divided into three general categories based on the focus of the proposed solution: (a) hydrologic work, (b) habitat work, and (c) hydraulic work. Hydrologic work can be accomplished through the use of stormwater ponds or through the modification of reservoir release schedules to modify the runoff regime as necessary to meet project objectives. Habitat work includes the construction of structures or features on the bed, bank, and/or riparian area to modify the biologic function of the stream. This can include measures that provide in-stream cover, low-flow channels, scour holes, riparian plantings, and substrate modification. Hydraulic work includes a variety of techniques that center on measures that affect the geomorphic characteristics of the channel. They can include measures to provide the channel dimensions and geometries required to produce a stable or regime condition, local works essential to supply the morphological diversity necessary to support a wide range of habitats, and the structures needed to hold the channel in its new alignment by preventing bank erosion.

Flood Damage Reduction Techniques

There is growing public interest in modifying existing flood damage reduction projects to restore natural environmental functions while still providing flood protection benefits. Careful planning, analysis, and design are required for the successful implementation of these changes. Some of the most common modifications are listed here. They are listed in decreasing order of preference from the standpoint of achieving a naturally behaving system.

a. Flood setbacks. This technique involves removing structures from the floodplain and restoring the channel to its historic configuration. The channel dimensions are designed so that the inflowing sediment load can be transported through the project reach without significant aggradation or degradation. Original planform can be restored. The stream is left to freely meander and flood its overbanks. Overbank flooding should occur every one or two years on the average. In urban areas, this option is typically infeasible due to real estate costs.

b. Levee setbacks. This technique is essentially the same as flood setbacks, except that the overbank floodplain is limited by levees. The levees should not encroach upon the meander belt so that the channel may still migrate within this morphologically active corridor.

c. Two stage channels. This type of project involves an upper channel section to provide flood conveyance, with a natural low-flow channel within it to provide habitat enhancement and improved sediment transport capacity. The upper area can be designed to provide public recreation or wetland habitat. The low-flow channel can be designed in a meandering pattern within the upper channel. However, both channels are essentially held in a static but geometrically stable condition. Natural stability in terms of sediment transport may not be maintainable because the top bank elevation of the low-flow channel will be lower than the natural bankfull elevation. Aggradation in the upper channel should be expected if the stream is an alluvial stream, but not as much as would occur in a large single-stage trapezoidal channel. Careful assessment of sediment transport capacity is needed to design the low-flow channel. Modification of an existing flood-control channel to this type can involve alteration to bridge piers and abutments, alteration to utilities and real estate acquisition since the construction of the upper channel will typically involve an expanded area in order to maintain flood conveyance.

d. Relief channels. This technique typically involves restoring the channel to its original configuration and constructing a high-flow channel or relief culvert to provide flood conveyance. The restored channel provides habitat benefits while the high-flow channel can be designed to provide wetland or lowland habitat or for recreational benefits. The high-flow channel is functioning as a detached floodplain. Real estate costs for the high-flow channel can be an issue. Careful consideration of the sediment transport conditions of the stream is required. In streams with high bedload, the loss of transport capacity at the entrance to the bypass channel can result in sediment deposition in the restored channel. Where the relief channel reenters the restored channel, the increase in sediment transport capacity can result in bed degradation.

e. Addition of in-stream habitat features. This can include the addition of boulders, wing deflectors, stone weirs, and lunker-type habitats within the existing flood damage prevention project. A low-flow meandering channel may be established within the flood channel. This low-flow channel is not the same as a natural regime channel and maintenance after flood events may be required. These features should be designed to withstand the forces of the flood flows. The effects of adding in-stream habitat features on channel conveyance and sediment transport must be considered. The reliability of the flood control project should not be compromised.

f. Addition of bank vegetation. Trees and shrubs can provide lowland habitat, channel shading, and aesthetic benefits. This type of project is often the easiest to implement since it involves minimal modifications to the existing project. However, vegetation may increase the channel roughness and a careful hydraulic analysis is required to assess such impacts. A hydraulic analysis can be used to aid in the selection of plant species. In addition, the possible impacts of debris clogging on infrastructure and channel stability should be assessed.

Additional information on ranking flood damage reduction alternatives is available in U.S. Army Corps of Engineers' Engineering Manuals. EM 1110-2-1418 provides a ranking based on channel stability and EM 1110-2-4000 provides a ranking based on sedimentation issues.

Project Constraints

The process of determining constraints is just as important as establishing objectives. An ideal stream restoration design might include a natural channel free to migrate laterally and longitudinally down the valley, connected hydraulically to its floodplain and with natural vegetation along the banks. Such a design would preclude most types of development in the floodplain. This may not be feasible, so a less than ideal solution may be required. Constraints are particularly common in urban floodplains and include rights-of-way, highways and bridges, utility crossings, buildings, archeological and historical sites, and cemeteries. Another common concern is the erosion of polluted sediment in the streambed or in the banks. To maintain water quality standards it may be beneficial to make sure these polluted sediments stay in place. These constraints may make it necessary to stabilize the banks, preventing the natural channel migration process. Another constraint common to Corps projects is flood damage reduction. Overbank flows, which may be beneficial for habitat development, are often not acceptable for economic, political, and/or social reasons.

3 Hydrology

General

Hydrologic computations are an integral part of stream restoration projects. A wide variety of techniques are available to the designer. The level of accuracy required for a specific hydrologic analysis generally depends on the specific characteristics of each individual project. The selection of the appropriate methodology should be done with a firm understanding of the assumptions, accuracy, data requirements, and limitations of the approach chosen. This chapter outlines some of the most common techniques and offers general guidelines regarding selection criteria. For more complete information on the details regarding the assumptions and limitations of specific models, the original documentation associated with each of the models should be reviewed. Final decisions regarding the suitability of a particular model for a particular project must be determined using engineering judgment on a case-by-case basis.

In order to design a stream restoration project with long-term stability, it is necessary to evaluate the full range of flows that will affect the channel. This requires the development of a historical hydrograph and/or a flow-duration curve. Estimates and calculations of existing base flows, channel-forming flows, and flood flows are often required. Estimates of future flow conditions are often required to properly assess future project performance. Base flows often define critical habitat conditions. Estimates of channel-forming discharges are used to determine channel dimensions. Flood flow estimates are used to determine stability of structures and natural channel features, as well as for scour depth prediction. The choice of a maximum design flow for stability analysis should be based on project objectives and consequences of failure. For example, the 100-year discharge might be used to design bank protection in a densely populated area while a 10-year discharge might be appropriate in a rural stream.

Frequency Analysis

Peak discharge analysis

The objective of hydrologic peak frequency analysis is to relate the magnitude of a given flow event with the frequency of that event's occurrence. This is accomplished using stream flow gage records. Data can be stratified by seasons depending on study goals. Gage records should contain at least 10 years of consecutive peak flow data, and they should span at least one wet year and one dry year. The frequency analysis requires that the flow data consist of independent events. A variety of hydrologic techniques are available for the prediction of the frequency of flow events. In general, the hydrologic analysis for the gage should follow the recommendations of Guidelines for Determining Flood Flow Frequency, Bulletin 17B (USIACWD 1982). Typically, three different asymptotic forms of extreme value distributions (Types 1, 2, and 3) are used in the frequency analysis.

Most estimates of infrequent flow events are made with the annual duration series where the number of values in the data subset is equal to the number of years of record. An annual maximum (or minimum) duration series is composed of the largest (or smallest) value in each year. Since these events occur on an annual basis, it is usually safe to assume that each observation is independent.

When the desired event has a frequency of occurrence of less than 2 to 5 years, a partial duration series is recommended. This is a subset of the complete record where the values are above a preselected base value. The base value is typically chosen so that there are no more than three events in a given year. In this manner, the magnitude of events that are equaled or exceeded three times a year can be estimated. Care must be taken to assure that small peaks associated with large events are not included in the analysis to ensure that independence is preserved. The return period for events estimated with the use of a partial duration series is typically 0.5 years less than what is estimated by an annual series (Linsley, Kohler, and Paulhus 1975). While this difference is fairly small at large events (100 years for a partial vs. 100.5 years for an annual series), it can be significant at more frequent events (1 year for a partial vs. 1.5 year for an annual series). It should also be noted that there is more subjectivity at the ends of both the annual and partial duration series.

Gage records provide an actual representation of the hydrologic behavior of a watershed. However, when a gage record is of short duration, or of poor quality, or the results are judged to be inconsistent with field observations or sound engineering judgment, then the analysis of the gage record should be supplemented with other methods. It is important to assess the applicability of the historic gage data to current conditions. For example, rapid increases of imperviousness in an urban watershed may have increased peak flows, rendering historic gage data obsolete. Correction of gage data is possible but can be problematic. If an invalid portion of a record is used, the results will be biased. It is also important to note that many stream restoration sites do not have an appropriate gage in the area.

Regional regression

Federal, state and local agencies have developed regional regression relations to estimate peak discharges at ungaged sites. Regional regression equations are easy-to-use and provide relatively reliable and consistent findings when applied by hydraulic engineers. They are typically statistical models that quantify general regional relationships between flow of a specific recurrence interval and a watershed's physiographic, hydrologic, and meteorological characteristics. The most simple regression relation is of the form:

$$Q_i = a + b\ A_i^c \tag{1}$$

where: Q_i is the dependent variable, such as flow; A_i is an independent variable, such as drainage area; and a, b, and c are regression coefficients derived from the database. The U.S. Geological Survey (USGS) has developed regional regression relations for many states (USGS 1993). Multiple regression analysis contains more than one independent variable and can thus account for more variations between watersheds.

Errors in regression analysis

Regional regression relations should include relevant parameters to account for unique characteristics of the study watershed. It is also important to consider the confidence limits of the regression relationships and how they relate to predicted discharges for the study reach. One of the most commonly used measures of goodness of fit is the correlation coefficient, expressed typically as r^2. This is a measure that describes how well a regression equation explains the relationship between the variables. It is the dimensionless ratio of the explained variation in the dependent variable over the total variation in the dependent variable. A correlation coefficient of 1.00 indicates that the values of the dependent variable can be calculated exactly using the independent variable in the given data set. Since this value is dimensionless, it can be used to compare goodness of fit of different regression relations. It does not provide a quantified expected variation. If the correlation is linear, it does not matter which variable is considered independent. However, if the relationship is nonlinear, the regression coefficients will be dependent on the choice of independent variable and curve fit. It should also be noted that a high degree of correlation (r^2 close to 1) does not necessarily imply causation or direct dependence between the variables. In all cases, the reasonableness of the causation between the independent and dependent variables should be examined. Additionally, variation is expected in natural systems. Data collection techniques should be examined if the calculated r^2 implies near perfect correlation (very close to 1) of field data.

Another measure of the robustness of a regression relation is the standard error of estimate, expressed typically as $S_{Y,X}$. This is the root mean square of the estimates. It is a measure of the scatter about the regression line of the independent variable. In general, the standard error of estimate is not reflexive, i.e., $S_{Y,X}$ is not a measure of how well the independent variable correlates to the dependent variable. The standard of estimate has similar properties to the standard deviation. Since the standard error of estimate has dimensions, it provides a measure of possible variation.

Flow-Duration Curves

Determination of the quantity of sediment transport is dependent on flow duration estimates. Flow-duration curves are typically calculated using the entire period of record. Procedures for this are described in "Hydrologic Frequency Analysis," EM 1110-2-1415. Data are typically sorted by magnitude, and the percent of the time that each value is exceeded is calculated. Data can be stratified by seasons depending on study goals. Regional flow-duration curves can be developed using drainage area. Drainage area can be used to transfer flow-duration information from gaged sites to ungaged areas; however, the ratio of gaged to ungaged drainage area should be between 0.5 and 2 for reliable results. Typically, there is more error in transferring or estimating the ends of a flow-duration curve. Flow duration is dependent on watershed conditions. If regional flow duration relations are to be developed, it is recommended that a measure of watershed development be included as an independent variable.

Two methods for estimating a flow-duration curve for ungaged sites are described in Appendix A. They are the drainage area – flow-duration curve method and the regionalized duration curve method. With the drainage area – flow-duration curve method graphs of a specified recurrence interval discharge versus drainage area are developed for a number of sites on the same stream or within hydrologically similar portions of the same drainage basin. If data are reasonably homogenous, power functions may be fit using regression and used to generate a flow-duration curve for the ungaged location. With the regionalized duration curve method a nondimensional flow-duration curve is developed for a hydrologically similar gaged site by dividing discharge by bankfull discharge or a specified recurrence interval discharge. Then a specified recurrence interval discharge is computed for the ungaged site using the aforementioned USGS regression equations. Finally, the flow-duration curve for the ungaged site is derived by multiplying the dimensionless flows (Q/Q_2) from the nondimensional curve by the site Q_2. It should be noted that both methods simply provide an approximation to the true flow-duration curve for the site because perfect hydrologic similarity never occurs.

It is often important to determine how the proposed restoration project will perform with low or normal flows. In addition, seasonal flow variations can have critical habitat importance. For example, a project goal may include a minimum flow depth during a critical spawning period for salmonoid species and a lower minimum depth for resident fish species. The same techniques used to develop flow-duration curves for sediment analysis can also be used to assess and design for habitat conditions.

Hydrologic Models

Hydrologic models have long been used to determine discharges for various recurrence intervals. Models are particularly applicable where gages are nonexistent, limited, or do not reflect current conditions. Models provide the ability to estimate existing as well as future rainfall runoff patterns for a variety of conditions. The accuracy of models is dependent on calibration data, which may not be available. However, if the issues that are to be addressed are comparative in nature rather than absolute, the importance of calibration is diminished. Brief statements on the use of the models are provided in the following paragraphs.

The rational method (rational formula) is one of the easiest models to implement. It is can be used for drainage areas up to 80 ha (200 acres). Use of the rational formula on larger drainage areas requires sound engineering judgment to ensure reasonable results. The hydrologic assumptions underlying the rational formula include constant and uniform rainfall over the entire basin and a rainfall duration equal to the time of concentration. If a basin has more than one main drainage channel, if the basin is divided so that hydrologic properties are significantly different in one section versus another, if the time of concentration is greater than 60 min, or if storage is an important factor, then the rational method is not appropriate.

The National Resource Conservation Service (NRCS) (formally SCS) TR-55 method (SCS 1986) provides a graphical method for computing peak discharges of drainage basins with areas ranging from 4.0 ha (10 acres) up to 800 ha (2,000 acres, 3.1 square miles). The TR-55 method is segmental (i.e., flow time is computed by adding the travel times for the overland, shallow concentrated (rill), and channel segments). TR-55 considers hydrologic parameters such as slope, roughness, losses, rainfall intensity, soil type, land use, and time. Some hydrologists have stated that TR-55 tends to produce conservatively high estimates of peak flows. TR-55 should be used with caution when the design is highly sensitive to the computed peak flow values. Although TR-55 has fewer assumptions than the rational formula, it also assumes that rainfall is uniform over the entire basin. Additional assumptions include:

a. Basin drained by a single main channel or by multiple channels with times of concentration within 10 percent of each other.

b. Time of concentration between 0.1 and 10 hr.

c. Storage in the drainage area is less than 5 percent of the runoff volume and does not affect the time of concentration.

d. A single composite curve number can accurately represent the watershed runoff characteristics.

The HEC-1 model is a rainfall-runoff model developed by the U.S. Army Corps of Engineers, Hydrologic Engineering Center (USAEHEC 1981). It can be used with basins of almost any size and complexity. HEC-1 is designed to simulate the surface runoff resulting from precipitation over a watershed by representing that watershed as an interconnected system of components. These components consist of surface runoff, stream channels, and reservoirs. Each component is represented by a set of parameters, which specify its characteristics, and the mathematical relations, which describe its physical processes. The end result of the HEC-1 modeling process is the computation of runoff hydrographs for the subbasins and stream channels. The program is composed of five basic submodels as illustrated in Figure 18.

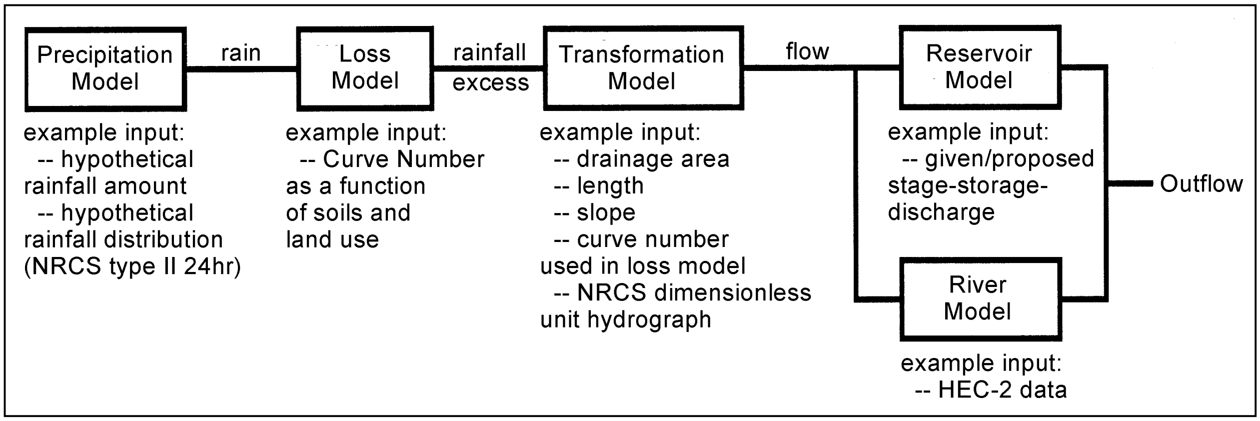

Figure 18. Submodels of HEC-HMS modeling process

HEC-1 assumes that the rainfall is spatially uniform over each subbasin modeled. NRCS rainfall time distributions, loss methods, dimensionless unit hydrographs, and the lag equations often are used; however, careful consideration must be given to the assumptions and limitations underlying these methods. For example, the NRCS has published an upper limit on basin size for the NRCS lag equation of 800 ha (2,000 acres, 3.1 square miles) (SCS 1985). The upper limit on basin area for the NRCS loss method (i.e., runoff curve number) is not well established; however, a limit of 52 km^2 (20 square miles) has been suggested. These limitations may be overcome by subdivision of the watershed and appropriate routing. Various GIS packages can be used as an interface to HEC-1. These GIS techniques systematize the computation of the physiographic and hydrologic parameters required by HEC-1. Similar hydrologic models include TR-20 and HEC-HMS.

Channel-Forming Discharge Concept

Natural alluvial streams experience a wide range of discharges and adjust their shape and size during flow events that have sufficient energy to mobilize either the stream's bed or banks. However, hydraulic design has been attempted using only a single representative discharge for many stream restoration projects. Using a representative or channel-forming discharge may be appropriate for determining initial or preliminary design channel dimensions, but the difficulty in the determination of the channel-forming discharge and the uncertainty related to the concept itself makes its sole use untenable for reliable and effective hydraulic design.

The channel-forming discharge concept is based on the idea that for a given alluvial channel geometry there exists a single steady discharge that given enough time would produce channel dimensions equivalent to those produced by the natural long-term hydrograph. This discharge therefore dominates channel form and process and may be used to make morphological inferences. Although conceptually attractive, this definition is not necessarily physically feasible because bank line vegetation, bank stability and even the bed configuration would be different in a natural stream than in a stream with a constant discharge.

The channel-forming discharge concept is not universally accepted. However, most river engineers and scientists agree that the concept has merit, at least for perennial and ephemeral streams in humid environments and perennial streams in semiarid environments. For channels in arid environments where runoff is generated by localized high-intensity storms and the absence of vegetation ensures that the channel will adjust to each major flood event, the channel-forming discharge concept is generally not applicable.

Care must be exercised in applying the channel-forming discharge procedure, particularly in unstable channels and those that have experienced catastrophic events during the period of record because flow-frequency and sediment-transport relations may have changed or be changing with time as the channel adjusts. Results may, therefore, represent a condition that does not accurately depict present flow and sediment-transport conditions.

Until the 1960s it was widely assumed that floods of great magnitude but low frequency controlled channel form because of the nonlinear relationship between discharge and sediment transport capacity. Sediment transport increases exponentially with discharge. This view was challenged by Wolman and Miller (1960) who demonstrated that in most streams over an extended period of time the total amount of sediment transported by a discharge of a given magnitude depends not only on its transport capacity, but also its frequency of occurrence. Thus, although extremely large events can produce spectacularly high sediment loads, they happen so infrequently and last such a short time that their overall contribution to the total sediment movement during a long period is relatively small. Small events also make a small contribution to the total sediment moved because their high frequency of occurrence is offset by their very low sediment transport capacity. It follows from this logic that flows of both moderate magnitude and moderate frequency are responsible for the greatest amount of sediment movement. Wolman and Miller defined moderate frequency as events occurring at least once each year or two and in many cases several or more times per year.

Assigning a single value to this theoretical channel-forming discharge is problematic. The following deterministic approximations for channel-forming discharge have been suggested as follows:

a. The natural bankfull channel discharge.

b. A discharge based on statistical return intervals.

c. The effective discharge or that discharge which, over time, does the most work and transports the most sediment.

Systematic methodologies for determining each of these approximations for the channel-forming discharge are presented in the following sections.

Bankfull discharge

The bankfull discharge is the maximum discharge that the channel can convey without overflowing onto the floodplain. This discharge is considered to have morphological significance because it represents the breakpoint between the processes of channel formation and floodplain formation. Based on both theoretical and empirical arguments, bankfull discharge is generally recognized as being the moderate flow that best fits Wolman and Miller's channel-forming discharge concept for streams in dynamic equilibrium. Leopold, Wolman, and Miller (1964) proposed that bankfull discharge was responsible for maintaining channel shape in natural alluvial channels and therefore was equivalent to channel-forming discharge. However, in an unstable channel that is adjusting its morphology to changes in the hydrologic or sediment regime, bankfull discharge can vary markedly from channel-forming discharge. Therefore, the expression "bankfull discharge" should not be used to refer to "channel-forming discharge," but should be reserved to refer only to the maximum discharge that the channel can convey without overflow onto the floodplain.

Bankfull discharge is determined first by identifying bankfull stage and then determining the discharge associated with that stage. Identifying the relevant field features that define the bankfull stage can be problematic. Many field indicators have been proposed, but none appear to be generally applicable or free from subjectivity (Williams 1978). The most common definition of bankfull stage is the elevation of the active floodplain (Wolman and Leopold 1957; Nixon 1959). Another common definition of bankfull stage is the elevation where the width to depth ratio is a minimum (Wolman 1955; Pickup and Warner 1976). This definition, diagramed in Figure 19, is systematic and relies only on accurate field surveys. In some cases the highest elevation of channel bars may be used as an indicator of bankfull stage (Wolman and Leopold 1957). Woodyer (1968) defines the bankfull stage of streams having several overflow surfaces as the elevation of the middle bench. Wolman (1955) combines the width to depth ratio criterion with identifying a discontinuity in the channel boundary such as a change in its sedimentary or vegetative characteristics. Schumm (1960) defined bankfull stage as the height of the lower limit of perennial vegetation, primarily trees. Similarly, Leopold (1994) states that bankfull stage is indicated by a change in vegetation, such as herbs, grasses, and shrubs. Given the number of criteria in common use to define bankfull stage and the considerable experience required to apply them, it is not surprising that there can be wide variability in field determination of bankfull stage.

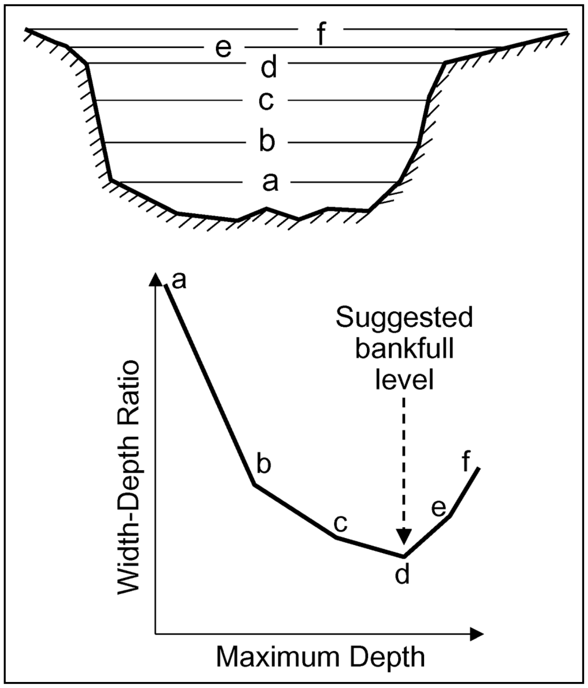

Figure 19. Bankfull depth using width-depth ratio (after Knighton 1984)

The field identification of bankfull indicators is often difficult and subjective (Knighton 1984). The stream reach should be identified as stable and alluvial before field personnel attempt to identify bankfull stage indicators. If the project reach is unstable or nonalluvial, it may be possible to find indicators of bankfull stage in stable alluvial reaches upstream or downstream on the same stream. The process of identifying bankfull indicators is often an iterative and subjective process that involves a great deal of judgment.

If a reach is not stable and not alluvial, indicators of bankfull stage will be unreliable. Some examples are given as follows:

a. If a reach is nonalluvial, then sediment transport capacity normally exceeds sediment supply, and deposits would be missing or underdeveloped. Using underdeveloped deposits as bankfull indicators would result in too low a channel-forming discharge. Deposits could also be relics of extreme flood events, in which case they would normally give too high a channel-forming discharge.

b. If the channel is degrading, then sediment transport capacity exceeds sediment supply, and the observations above for the nonalluvial channel hold true. In addition, since the bed of the channel is lowering, former floodplain deposits are being abandoned (they are in the process of becoming terraces.) Using these deposits as indicators would give too high a channel-forming discharge.

c. If the channel is aggrading, the in-channel deposits could be incorrectly mistaken for bankfull stage indicators. Since the bed of the stream is rising, using the existing floodplain as an indicator would give too low a discharge. (The floodplain will aggrade as well, but usually at a slower rate than the channel.)

Confusion often occurs when criteria suggest a bankfull stage at an elevation that is not close to the top of either bank. This condition suggests that the channel is not in equilibrium, that the existing channel geometry may not be stable, and that the channel-forming discharge would be poorly approximated by the bankfull discharge. Since stream restoration is most often practiced in unstable channels and watersheds (instability is often the reason for restoration), field determination of bankfull stage may be impractical or impossible. In fact, attempting to determine a channel-forming discharge from an unstable stream is in conflict with the theoretical premise that is the basis for the channel-forming discharge concept.

Once bankfull stages are estimated for a reach of the stream, then bankfull discharge can be estimated. Ideally, the discharge associated with bankfull stage can be determined from a stage-discharge rating curve based on measured data at the project site. When floodplain conveyance is significant with respect to channel conveyance, there will be a distinct break in the stage-discharge rating curve at bankfull stage as shown in Figure 20. The data scatter in Figure 20 occurs because stage is not a unique function of discharge in alluvial streams. It is therefore necessary to estimate a rating curve through the data scatter. It is best to consider that the bankfull discharge will have a range rather than a single discrete value. Uncertainty associated with the stage-discharge relationship is addressed in EM 1110-2-1619. In cases where floodplain conveyance is not significant with respect to channel conveyance, there may not be a distinct break in the stage-discharge rating curve (Figure 21). In this case the bankfull discharge may not have as much morphological significance as when floodplain flow is significant. Lacking gage data at the site, a stage-discharge rating curve can be determined from a backwater analysis. Ideally, the downstream starting water-surface elevation will be based on data from a gaging station. The accuracy of this rating curve will depend on the uncertainties associated with assigned hydraulic roughness coefficients and the cross-section geometry. Uncertainty is greatest when the stage-discharge rating curve is estimated from a single cross section. In this case both hydraulic roughness and energy slope must be assigned. It is best if the determination of bankfull stage occurs over a reach length of at least one wavelength or 10 channel widths. An example of a comparison of bankfull stage and a computed water-surface elevation is shown in Figure 22. Note in Figure 22 that bankfull stage is taken to be at the bottom of the top-of-bank data scatter because this represents the elevation at which flow onto the floodplain begins. Also note that considerable variability in bankfull stage could be estimated if only a single top-of-bank point were used in the analysis. The hydraulic engineer determines what method is best suited to compute the bankfull discharge from the bankfull stage indicators. For example, backwater computations may be required in some cases, while normal depth computations will be sufficient in others.

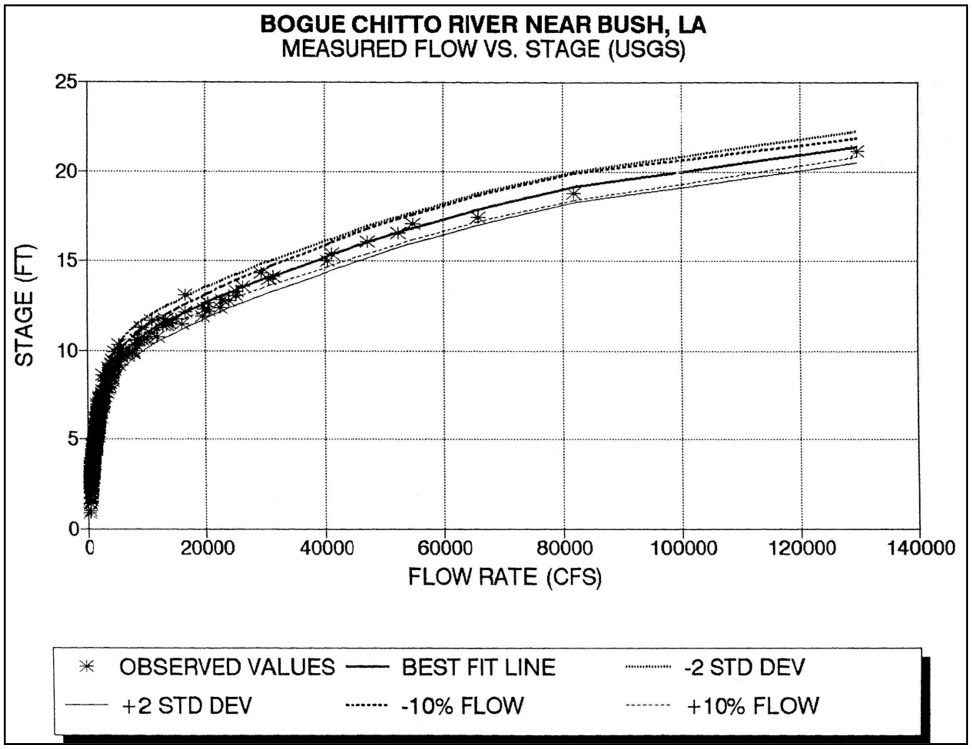

Figure 20. Stage-discharge rating curve Bogue Chitto River near Bush, LA

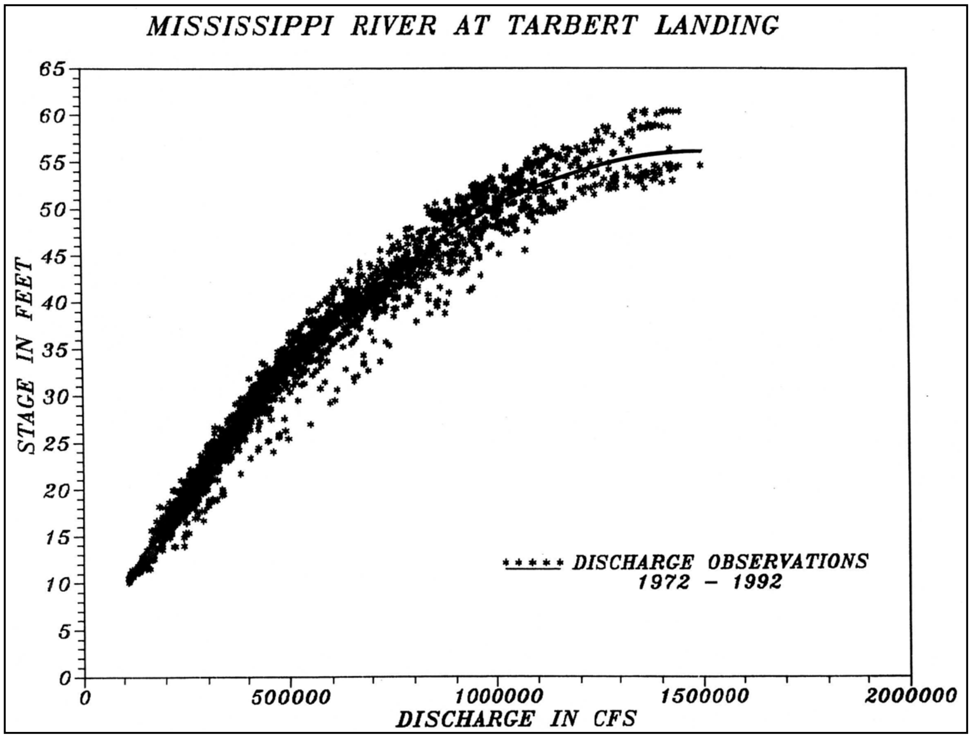

Figure 21. Stage-discharge rating curve Mississippi River at Tarbert Landing, MS

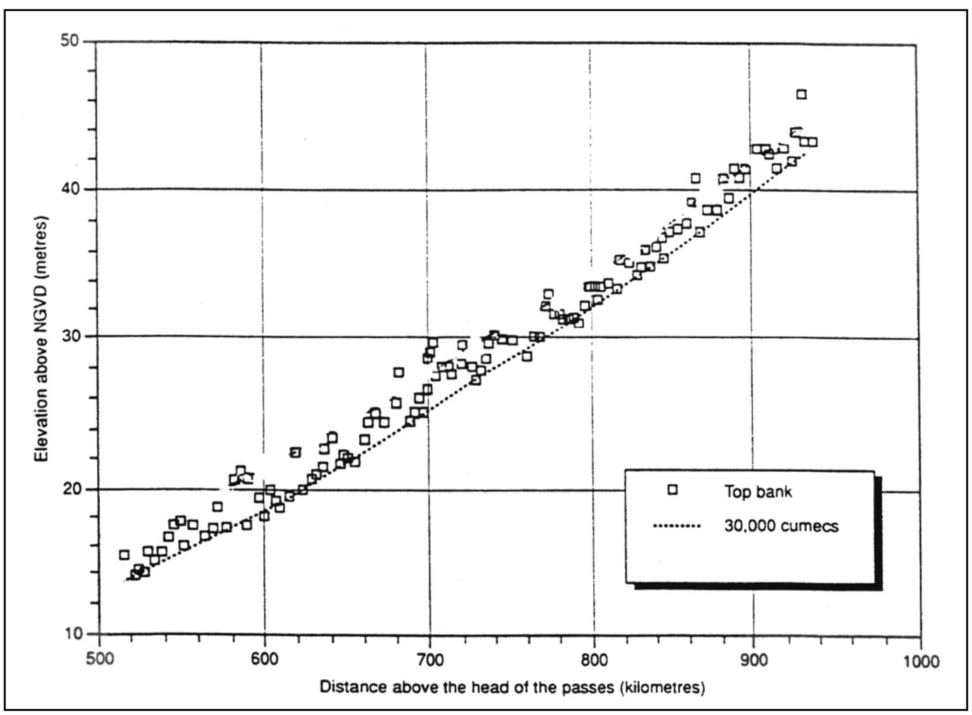

Figure 22. Long-channel variation in bank top elevations: Lower Mississippi River (Biedenharn and Thorne 1994)

The following guidelines are provided relative to field determination of bankfull discharge and use of bankfull discharge as the channel-forming discharge:

a. Bankfull discharge is geomorphologically significant only in stable alluvial channels. Therefore, the reach where bankfull stages are determined should be stable and the streambed should be mobile at bankfull flow.

b. The estimates of bankfull discharge most appropriately used to determine channel dimensions for the main channel are those based upon top-of-bank indicators. A stage identified by the edge of the active channel, the beginning of woody vegetation, or the top of channel bars may have value for designing those particular features in a restored channel, but should not be used for establishing the bank height of a stable channel. Only bankfull discharges that are top-of-bank discharges are morphologically significant in establishing the channel-forming discharge.

c. An exception to the preceding rule is in a stable and alluvial incised stream that has formed a new floodplain within the incised channel. In this case, the top of the high bank is now an abandoned floodplain or terrace, and there should be newly formed top-of-bank features within the older incised channel. However, it is important to remember that the new floodplain may not yet be fully formed; that is, the channel may not be stable (it may still be aggrading). This would give misleading values for the bankfull discharge.

d. Assuming that the bankfull discharge for one reach of a stream is the same as the bankfull discharge in another reach may not be appropriate. The location of the break between the channel and the floodplain is influenced by many factors, including (but not limited to) the following:

(1) confinement of the floodplain

(2) hydrologic regime

(3) sediment supply

(4) bed and bank sediment size and cohesiveness

(5) size and type of vegetation on the floodplain and within the channel

(6) controls on channel width, slope and alignment

For example, the bankfull discharge taken from a reach with a narrow floodplain may be inappropriate for use on another reach on the same stream, which has a wide floodplain.

Discharge for a specific recurrence interval

Due to difficulties in the identification of bankfull stage and discharge, many researchers have related the channel-forming discharge to a specific recurrence interval discharge. In these studies the researchers have typically studied stable streams where bankfull stage could readily be determined and where stream gages were located nearby. Under these conditions, bankfull discharge and channel-forming discharge were assumed equivalent and most of the literature addressing specific return interval discharges uses the two terms interchangeably. This can be confusing, and it should be remembered that these studies are actually comparing two methods for approximating the channel-forming discharge, and not actually comparing an approximation method to the true value.

In general, bankfull discharge in stable channels is often assumed to correspond to an annual flood recurrence interval of approximately 1 to 2.5 years and the 1.5-year recurrence flood has been shown to be a representative mean of many streams (Leopold 1994). Wolman and Leopold (1957) suggested that the bankfull discharge had a recurrence interval of 1 to 2 years. Dury (1973) concluded that the bankfull discharge is approximately 97 percent of the 1.58-year discharge or the most probable annual flood. Hey (1975) showed that for three British gravel bed rivers, the 1.5-year discharge provided a water-surface elevation that passed through the scatter of bankfull discharges measured along the course of the rivers. Richards (1982) suggests that in a partial duration series, bankfull discharge equals the most probable annual flood, which has a 1-year return period. However, there are many instances where the channel-forming discharge does not fall within the 1 to 2.5-year range. Recurrence interval relations are intrinsically different for channels with flashy hydrology than for those with less variable flows. For instance, Williams (1978) clearly showed that out of 35 floodplains he studied in the United States, the bankfull discharge varied between the 1.01- and 32-year recurrence interval, and that only about a third of those streams had a bankfull discharge recurrence interval between 1 and 5 years. In a similar study, Pickup and Warner (1976) determined that bankfull recurrence intervals ranged from 4 to 10 years. Because of such discrepancies, many have concluded that recurrence interval approaches tend to generate poor estimates of bankfull discharge. Hence, field verification is recommended to insure that the selected discharge reflects morphologically significant features.

Effective discharge

The effective discharge is defined as the mean of the discharge increment that transports the largest fraction of the annual sediment load over a period of years (Andrews 1980). The effective discharge incorporates the principle prescribed by Wolman and Miller (1960) that the channel-forming discharge is a function of both the magnitude of the event and its frequency of occurrence. An advantage of using the effective discharge is that it is a calculated value not subject to the problems associated with determining field indicators. It is calculated by integrating the flow-duration curve and a bed-material-sediment rating curve. A graphical representation of the relationship between sediment transport, frequency of the transport, and the effective discharge is shown in

Figure 23. The peak of curve C from Figure 23 marks the discharge that is most effective in transporting sediment, and therefore it is hypothesized that it does the most work in forming the channel.

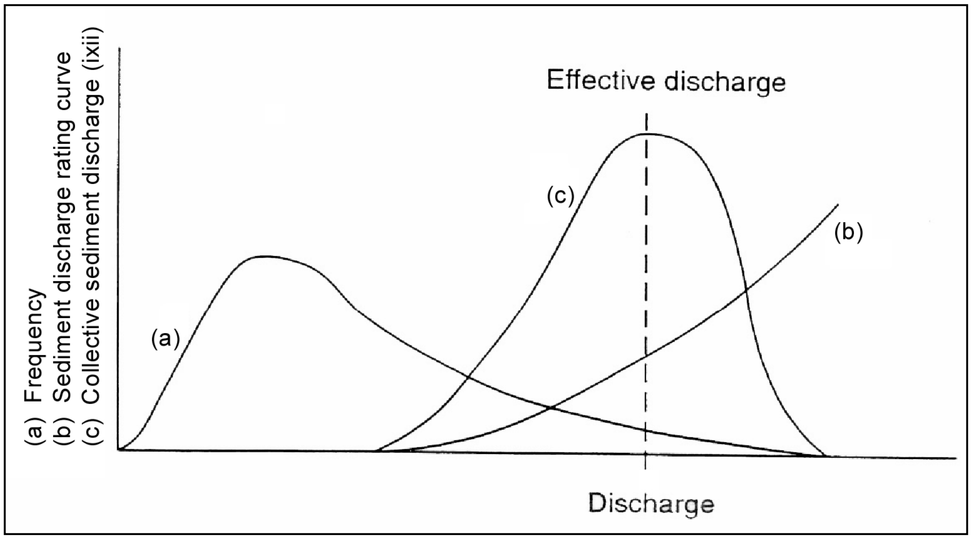

Figure 23. Derivation of total sediment load-discharge histogram (c) from flow frequency (a) and sediment load rating curves (b)

In various types of stable alluvial streams researchers have demonstrated the equivalence between bankfull and effective discharges (Andrews 1980; Carling 1988; Hey 1997). However, the effective and bankfull discharges are not always equivalent as reported by Benson and Thomas (1966); Pickup and Warner (1976); Webb and Walling (1982); Nolan, Lisle, and Kelsey (1987); and Lyons, Purcherelli, and Clark (1992). This suggests that the effective discharge may not always be an adequate surrogate for the channel-forming discharge. However, it may simply reflect uncertainties in (a) the determination of the bankfull elevation, (b) the calculation of the bankfull discharge corresponding to that elevation, (c) the calculated sediment transport rating curve and/or (d) the inherent uncertainties in the effective discharge calculation.

The recommended procedure to determine the effective discharge is executed in three steps. They are as follows:

a. The flow-duration curve is derived from available stream gage data.

b. A bed-material-sediment rating curve is constructed from sediment data or calculated using a bed-material sediment transport equation.

c. The flow-duration curve and the bed-material-sediment rating curve are integrated to produce a sediment load histogram that displays sediment load as a function of discharge for the period of record. The histogram peak is the effective discharge increment.

Calculating a flow-duration curve involves selecting the type of discharge data to be used and a method for subdividing the observed range of discharge into classes. The period of record should be at least 10 to 15 years. In many cases, mean-daily discharges are used because these data are readily available from the U.S. Geological Survey (USGS) and others. However, except for large rivers, mean-daily flows tend to be low-flow biased because they mask the effects of short-duration peak flows on sediment transport. Discharges representing shorter time periods than a day, such as the 15-min data collected by the USGS, provide a more accurate means of establishing a sediment transport rating relation. These data, although superior for a broader size range of streams and rivers, are not readily available, but may sometimes be obtained via special request. The discharge data is typically divided into about 25 class intervals of equal size, although the appropriate number of intervals may vary. Effective discharge calculations require the use of arithmetic intervals for the discharge class. This is different from calculating a flow duration curve where logarithmic intervals are frequently employed. One indicator that additional intervals may be needed is when the discharge mode occurs in the lowest discharge class, which frequently occurs in small, flashy streams.

A bed-material-sediment rating curve, showing sediment concentration as a function of water discharge, can be calculated or determined from measured data. If measured suspended data are used, the wash load component should be removed from the total concentration. If sediment data are not available, bed-material sediment transport rates can be calculated from a variety of sediment transport functions. Frequently, the logarithms of sediment concentrations are plotted against the logarithms of discharge and regressed to create a simple rating relation. However, power functions derived this way are often inadequate to define the transport relation because they overestimate transport at high flows. In addition, depending on how the bed-material varies with discharge, and when and how the bed-material gradation data used in the sediment transport equation were determined, the predicted transport can also be overestimated at low discharges. This necessitates using two or three linear segments or a curved rating.

The sediment load histogram is developed by multiplying the frequency (in percent) in each discharge class by the bed-material sediment transport rate corresponding to the mean of the class interval. The mean of the class interval with the greatest transport load is the effective discharge. In some cases, however, there may not be a single class interval representing a maximum. Instead, the peak average transport rate may spread across a range of classes, indicating that there is no single effective discharge but that significant geomorphic work is performed by a range of flows.

Since channel instability is the result of an imbalance in sediment supply and transport capacity, the greatest advantage of using effective discharge in restoration design lies in the fact that it requires quantification of the sediment transport capacity of a channel for a given hydrologic regime. Various channel geometries can be examined for their competence to transport the incoming sediment load, facilitating comparison of permutations of channel dimensions in order to optimize sediment transport efficiency within logistical constraints. This information is also useful when predicting the impact of alteration of watershed conditions with respect to sediment loads (e.g., upstream dam removal) or hydrology (e.g., urbanization) on channel stability.

Examples of channel-forming discharge representations

Using data collected from 57 stable sand bed rivers in the United States, Thorne and Soar (2000) compared the bankfull discharge with other representations of channel-forming discharge. The bankfull discharge was taken as the best representation of channel-forming discharge because the rivers were stable. Of course, the bankfull discharge, in this case, was calculated from measured hydraulic dimensions, slope estimated from topographic maps and field estimates of hydraulic roughness and must therefore also be considered an approximation of the channel-forming discharge.

The effective discharge is compared to the calculated bankfull discharge in Figure 24. This figure confirms the results of earlier research that the effective discharge is significantly lower than the bankfull discharge in most cases, particularly at low discharges, and only approximates the bankfull discharge at high discharges in a small proportion of streams. In general, the effective discharge provides an adequate lower bound for the range of bankfull discharges.

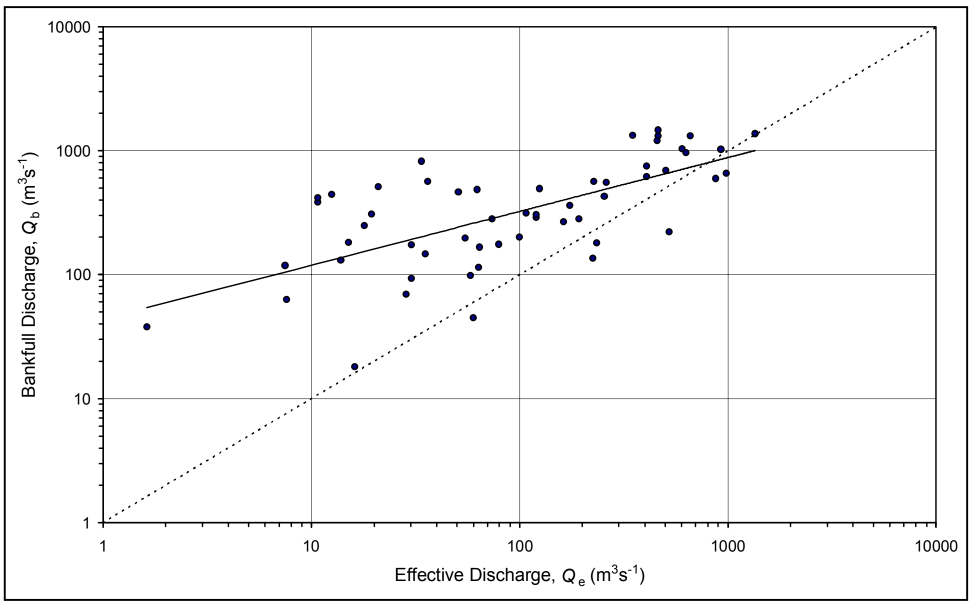

Figure 24. Relationship between effective discharge, Q_e, and bankfull discharge, Q_b, for 57 U.S. sand bed rivers. Solid line is the best-fit power relationship. Dotted line is equality

In contrast, as shown in Figure 25, the 2-year flow event is greater than the bankfull discharge in most cases and provides an adequate upper bound to the range of bankfull discharges. The best-fit line in Figure 25 is linear at the 95 percent significance level and represents bankfull discharges at approximately 60 percent of 2-year flow over the range of the data.

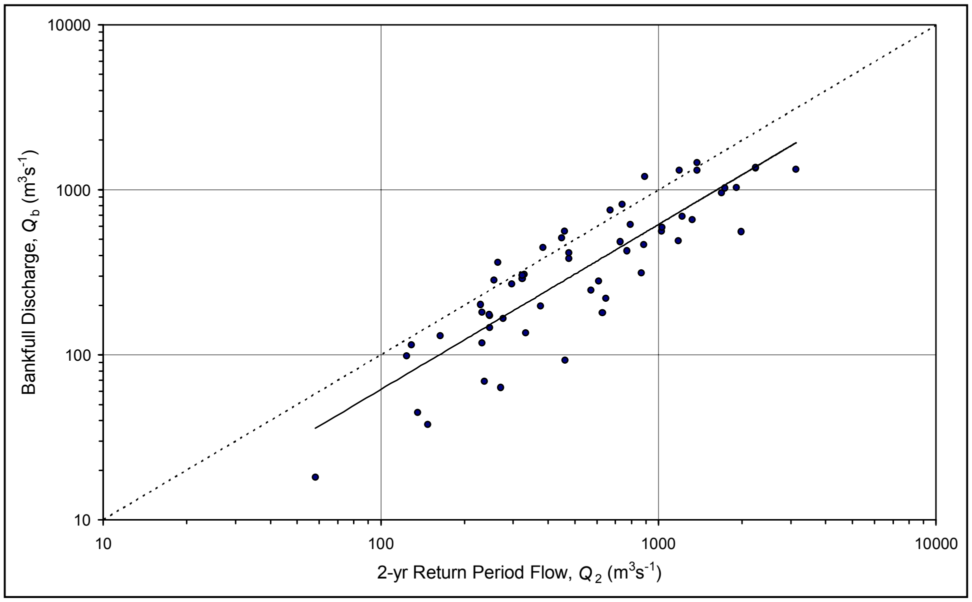

Figure 25. Relationship between the 2-year return period flow, Q_2, and bankfull discharge, Q_b, for 57 U.S. sand bed rivers. Solid line is the best-fit power relationship. Dotted line is equality

The effective discharge is statistically defined as the steepest gradient on the cumulative sediment frequency curve. Thorne and Soar (2000) hypothesized that the median discharge on this curve might have greater morphological significance than the effective discharge. The median discharge is defined as the upper limit of the range of discharges that transport 50 percent of the average annual bed material load (Q_{e50}). Although providing a closer association with bankfull discharge, the median flow with respect to sediment transport, Q_{e50}, underestimates the bankfull discharge in most cases (Figure 26). However, the 75 percent flow with respect to sediment transport, Q_{e75}, provides a better relationship with bankfull discharge with an r^2 value of 0.82 for the best-fit line (Figure 27). Furthermore, there is no statistical difference between the best-fit line in Figure 27 and the line of perfect agreement at the 95 percent significance level. The Q_{e75} discharge corresponds in many cases to either a high in-bank flow or a flow that just overtops the bank, and could provide engineers with a useful alternative to the effective discharge as a surrogate for the channel-forming discharge in stable sand bed rivers.

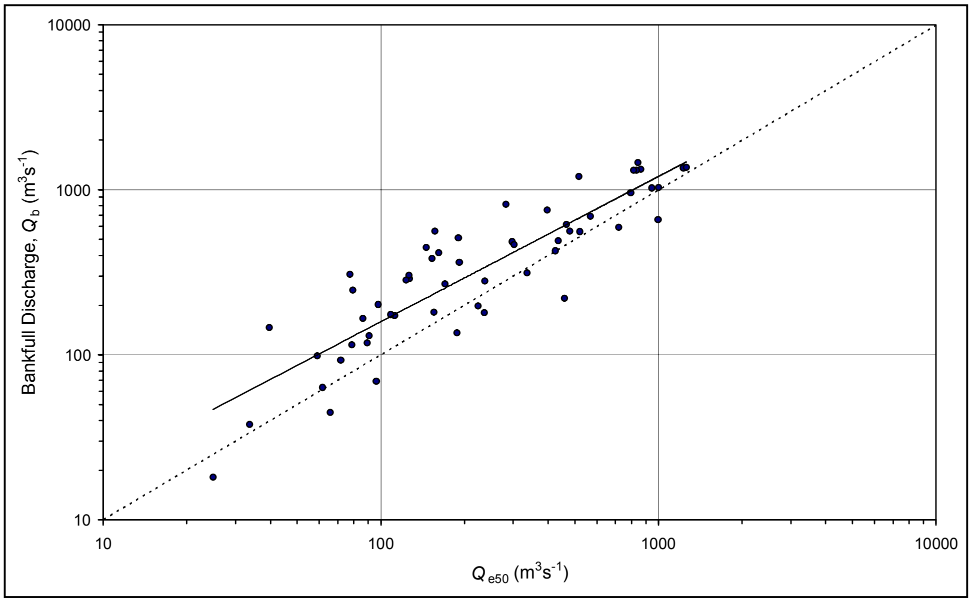

Figure 26. Relationship between the discharge marking the upper limit of the range of discharges that transport 50 percent of the average annual bed material load, Q_{e50}, and bankfull discharge, Q_b, for 57 U.S. sand bed rivers. Solid line is the best-fit power relationship. Dotted line is equality

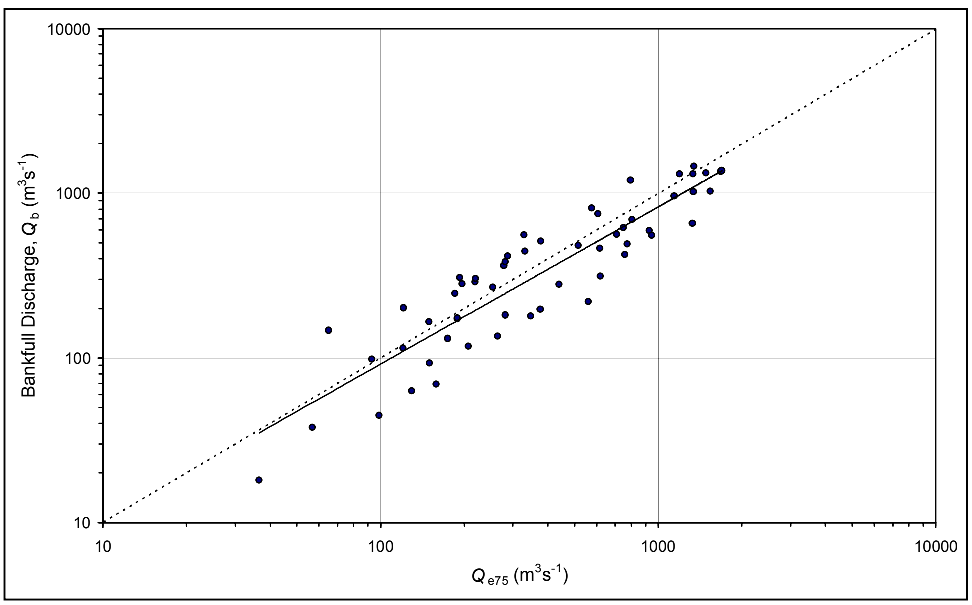

Figure 27. Relationship between the discharge marking the upper limit of the range of discharges that transport 75 percent of the average annual bed material load, Q_{e75}, and bankfull discharge, Q_b, for 57 U.S. sand bed rivers. Solid line is the best-fit power relationship. Dotted line is equality

Channel-forming discharge related to drainage area

Use of regional regression curves for determining channel-forming discharge as a sole function of the drainage area is not recommended. Drainage area is only one of many parameters affecting runoff.

Within physiographically similar watersheds it may be useful to develop a channel-forming discharge versus drainage area curve for use in that watershed. Emmett (1975) developed such a curve for the Salmon River in Idaho (Figure 28). Emmett chose stable channel reaches for his study and assumed that bankfull discharge was equivalent to channel-forming discharge. Although the regression line fits the data in a visually satisfactory fashion, it should be noted that for a drainage area of about 181 km^2 (70 square miles), the bankfull discharge varied between about 8.5 m^3/s and 25.5 m^3/s (300ft^3/s and 900 ft^3/s). This large range should not necessarily be attributed to errors in field measurements, but rather to the natural variation in bankfull discharge with drainage area.

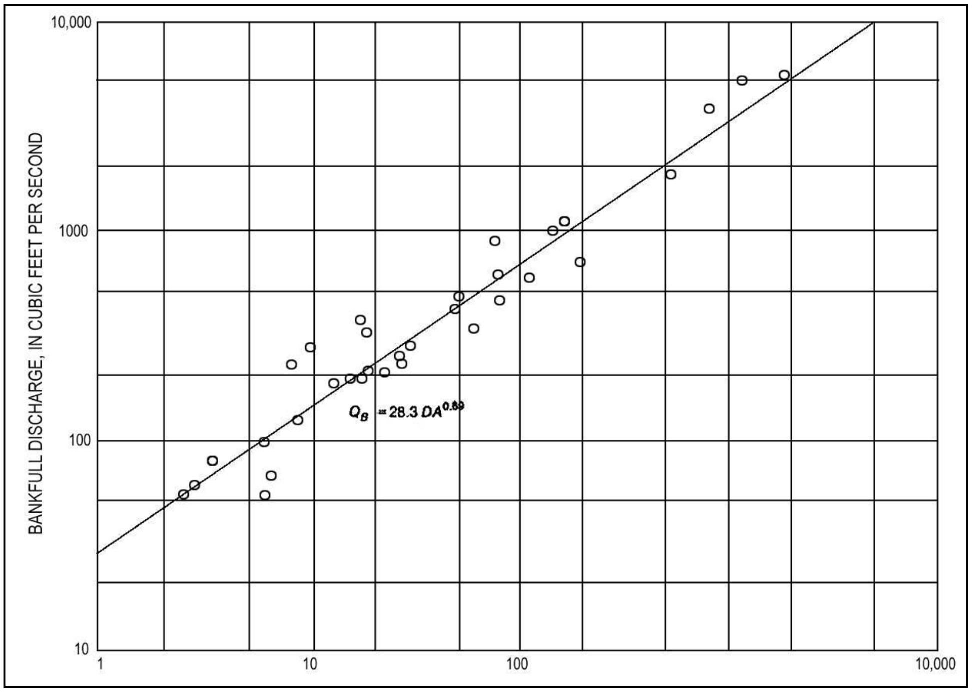

Figure 28. Bankfull discharge as a function of drainage area (Emmett 1975)

Channel-Forming Discharge Summary

Channel forming discharge can be estimated using a prescribed methodology. One such deterministic discharge is the bankfull discharge. Another deterministic discharge used to represent the channel-forming discharge is a specified recurrence interval discharge, typically between 1 and 3 years. The third is effective discharge.

All three methodologies for estimating channel-forming discharge present challenges. In practice, problems often arise when attempting to identify bankfull stage in the field. Although several criteria have been identified to assist in field identification of bankfull stage, ranging from vegetation boundaries to morphological breaks in bank profiles, considerable expertise is required to apply these in practice, especially on streams which have in the past undergone aggradation and degradation. Recurrence intervals for channel-forming discharge are generally in the range of 1 to 3 years, but have been shown to vary widely (4 to 10 years) for different types of streams. Calculation of effective discharge requires hydrologic and sediment data. Without nearby gage data, effective discharge calculations require use of an assumed hydraulic roughness and selection of a reliable sediment transport equation. In light of these challenges, it is recommended that all three methods be used and cross-checked against each other to reduce the uncertainty in the final estimate of the channel forming flow.

Stormwater Management

Stormwater management can be an important component of a broad restoration program in urban and suburban watersheds. Stormwater management can provide for channel stability as well as habitat benefits. There are a variety of stormwater management techniques that are in use today. An appropriate technique should be selected based on a firm understanding of the technique's limitations and capabilities and the hydrologic effects of urbanization. The hydrologic implications of various management practices need to be taken into consideration. This section outlines some of the more common techniques and offers general guidelines regarding selection criteria.

General hydrologic effects of urbanization and stormwater management

The primary hydrologic result of urbanization is an increase of runoff from a rainfall event of a given recurrence interval. Urbanization results in an increase in the impervious area of the watershed. Impervious areas such as parking lots, roads, and roofs increase the rainfall excess by reducing the volume of rainfall that can be absorbed through infiltration. Gutters, culverts, and storm sewers also reduce the travel time across the watershed, which increases the rate of rise of the runoff hydrograph. In addition, as an area is developed, natural ponding areas are reduced which further increases the rate of rise and total volume of the urban runoff hydrograph. Only in rare cases, such as the development of poorly tilled cropland into large lot residential areas, would runoff volume decrease as the watershed developed.

A large watershed can typically be broken into three areas relative to a project location as shown in Figure 29.

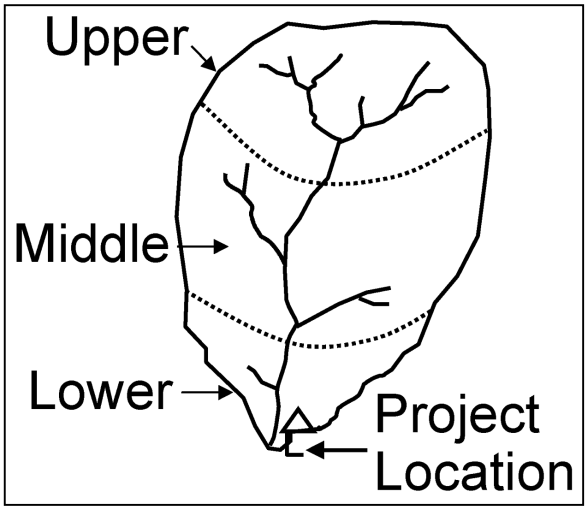

Figure 29. Schematic of a watershed relative to project location

The upper portion of the watershed is typically very sensitive to development. Stormwater management should focus on long detention times in this area to prevent the upper watershed peak flows from coinciding with the peaks in the lower portion of the watershed. Measures that decrease travel time within reaches such as piping and channelization can have detrimental affects downstream for similar reasons. Since the base flows are naturally lower in this upper portion, infiltration measures are especially important to maintain base flow for habitat purposes.

In the middle portion of the watershed, the detention should be intermediate in nature. The goal for stormwater management in this area is to delay flows long enough to allow runoff from the lower portion to clear the watershed. However, the delay should not be significant enough to cause an overlapping with the peak flows from the upper portion of the watershed.

In the lower area, stormwater management that relies on long detention times should be avoided since it may result in peak runoff from this lower area being delayed to coincide with the peak runoff from the upper watershed. As a result, this lower area is typically less sensitive to development.

Watersheds and development patterns are unique. It is recommended that a basin hydrologic model be developed and used in conjunction with stormwater planning to avoid adverse interactions between stormwater management measures.

The four basic types of stormwater management ponds are listed in the following sections. New structures often make use of features from all four.

Conventional dry ponds

Since the 1970s, many state governments have required that the hydrologic effects of urbanization be mitigated through the implementation of stormwater management practices primarily of the form of stormwater ponds. Stormwater ponds are designed to reduce the effects of development on nuisance level flooding. The ponds are directed towards maintaining the post development peak discharge of the 2-year and 10-year storm events and have been locally very effective for this design goal. While stormwater ponds have met with success in reducing the peaks of storm events, conventional dry ponds do little to reduce the overall quantity of runoff. It has only been in recent years that the use of stormwater ponds for water quality enhancement and channel-forming flows has been implemented. In addition, research has suggested that if stormwater ponds are designed without consideration to their relationship in the watershed, their interaction may result in an increase of peak discharge over what would have occurred if they had not been constructed at all (Ferguson 1991). As a result, it is advisable to develop a watershed hydrologic model if stormwater management is a significant portion of a watershed study so that the impacts of existing and proposed stormwater ponds on the watershed can be determined.

A conventional flood reduction stormwater management pond can adversely affect stability of the downstream channel. Figure 30 illustrates an idealized effect of a stormwater pond. In this example, the pond maintains the peak velocities of post development conditions at the predevelopment level. While this might indicate that flooding would not be exacerbated, a geomorphic or stability assessment is required to determine if the channel stability is adversely affected. For example, if the erosion threshold velocity is *V2*, then postdevelopment conditions with the pond should have minimal effect on the channel morphology. However, if the erosion threshold velocity is *V1*, then the pond can increase channel erosion. Since channel erosion is time dependent, a pond can make downstream channel erosion worse over conditions that existed without a pond.

Many stormwater management plans recommend a series of stormwater ponds throughout the watershed. This is schematically illustrated in Figure 31. The use of a number of ponds at the upper portions of the watershed will allow for a more uniform control of the entire watershed hydrology. It is typically easier to achieve multiple objectives (flood reduction, stability, and ecological) with multiple ponds. However, it is recommended that a watershed model be used for the planning and permitting of these features in order to avoid adverse interactions between the ponds. A drawback to the use of multiple ponds is the typical increase for maintenance over what is typically required for a single larger structure. A benefit is that since these are typically smaller structures, the dam safety requirements are typically less than for larger dams and thus they are simpler to design and construct.

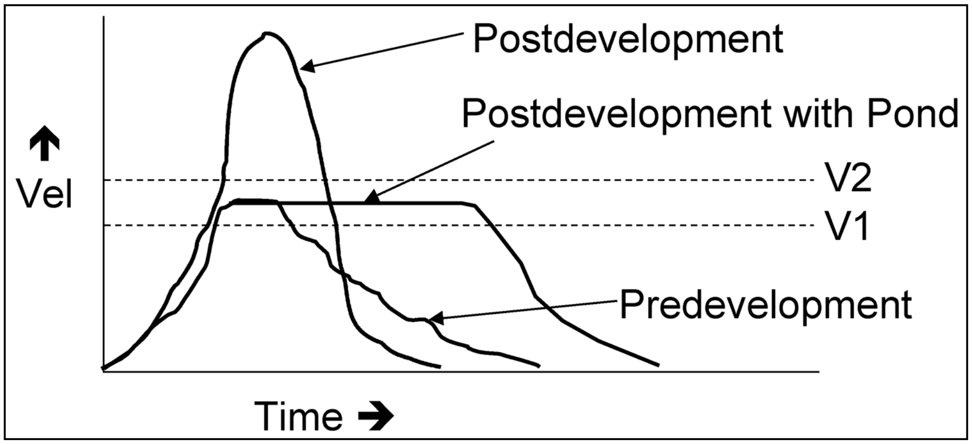

Figure 30. An idealized effect of stormwater pond on channel velocities

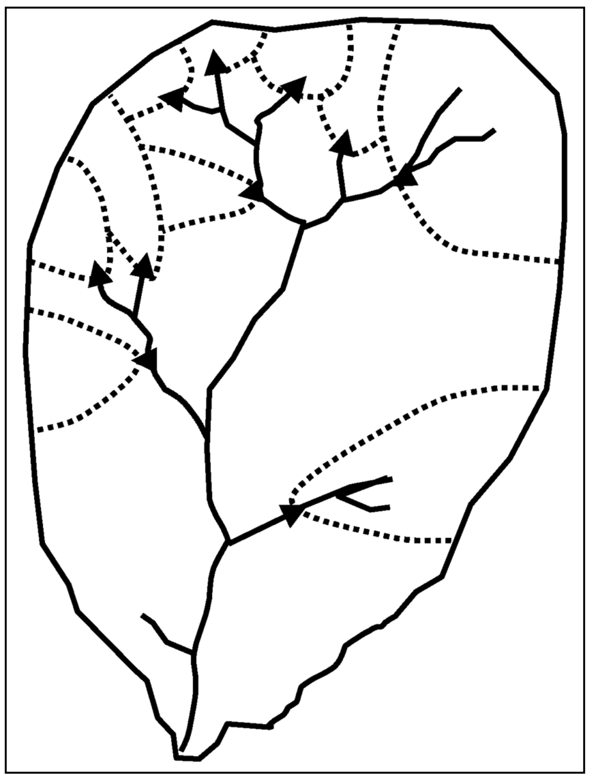

Figure 31. Schematic of a watershed relative to multiple small ponds

Retrofitting older storm water ponds to provide additional features such as those documented herein can often provide significant benefits. Existing stormwater ponds throughout the Northeast are being retrofitted to provide such benefits. Utilizing the existing stormwater drainage system and increasing the storage volume of an existing pond can be a cost-effective approach to stream restoration in urban and suburban watersheds.

Extended detention ponds

Extended detention ponds can provide both water quality benefits and reduce erosive flows. The most common design storms are the 1-year rainfall event or the event that generates 0.5 in. of runoff. The first 0.5 in. of runoff is considered to provide a “first flush” of the watershed and contains a significant concentration of pollutants. The 1-year event is also considered for erosion control. The design storms are detained for 12 to 24 hr as measured between the centroid of the inflow to the centroid of the outflow hydrograph. This results in a longer detention time and a decrease in the peak discharge over what would have occurred without the pond. The water quality benefits are provided by detaining water for enough time to allow sediments (and their attached pollutants) to settle to the bottom of the pond. The stream stability benefits are based on the premise that the increased volume of runoff from the developed watershed is offset by a reduced peak discharge.

As noted to be the case with dry ponds, if stormwater ponds are designed without consideration to their relationship in the watershed, their interaction may result in an increase of peak discharge over what would have occurred if they had not been constructed at all. Maintenance requirements should be considered due to the settling out of suspended sediments. To facilitate maintenance, a sediment forebay is recommended for these systems. Effects of the structure on fish passage as well as thermal loading to the stream should also be considered. Temperature of water stored in detention ponds typically increases with time and may adversely affect cold water fisheries downstream.

Wetland-pond systems

Wetland-pond systems are used to provide aesthetic, habitat, and water quality benefits. Often, large systems include nature and fitness trails. Habitat benefits can be provided with high and low marshes, nesting islands, and planting diversity. It is important to note that since these features are a sink for a variety of pollutants, the choice of planted species is more limited than in conventional wetland creation sites. Water quality enhancement is a result primarily of the settlement of pollutant-laden sediment, and physical filtration of particulate matter as well as nutrient uptake.

As with any shallow impoundment, a drawback for the use of wetland-pond systems is primarily thermal loading to downstream reaches. Effects of the structure on fish passage as well as public safety should also be considered. Maintenance can be more extensive due to the settling out of suspended sediments. If they are not maintained, they can become a source of pollutants during dry weather. To facilitate maintenance, a sediment forebay is recommended for these systems.

Infiltration basins and bioretention

Infiltration designs are often a preferred first choice since they seek to mimic predevelopment hydrology. They provide quasihabitat benefits through increased base flow and water quality benefits though filtration. However, they are limited to areas that have well drained soils and often require large areas. A sediment forebay is recommended since many of the infiltration designs that are currently in use are prone to failure by clogging.

Bioretention projects typically involve the use of shallow ponding areas and infiltration. The use of mulching and vegetation reduces the possibility of clogging and failure of the infiltration components of the bioretention systems. Because they are relatively small, they can be incorporated into the landscaping plans of almost any site. The primary benefit of this type of project is improved water quality and the maintenance of base flow. Bioretention and infiltration designs typically do not affect runoff during larger events.

Stormwater management guidance

There are a wide variety of reports, technical papers, and manuals that address different aspects of stormwater management design and usage (American Society of Civil Engineers 1993; McCuen and Moglen 1988; Moglen and McCuen 1988; Ferguson 1994; and Schueler 1994). State and local governments are often in the forefront of the development of design guidance (Maryland Department of the Environment, 1984 and 1987; and Prince Georges County, MD 1997).

4 Stability Analysis

Stability analyses are necessary for stream restoration projects. The purpose of the stability analysis is to identify the dominant fluvial processes in the stream system. Knowledge of dominant channel processes allows prediction of the proposed project's impact on channel morphology and channel stability and the effect the natural processes will have on the functionality of the project. An accurate assessment of the stability of various stream reaches and the types of instability occurring in the stream system (aggradation, degradation, planform instability, etc.) is the foundation for the designer's understanding of the watershed's dominant physical processes.

The most basic form of stability analysis is the assessment of bed stability - the determination of whether the channel bed is aggrading, degrading, or stable. Other aspects of stability assessment are bank stability, planform stability, historic or future changes in hydrology or sediment inflow, and changes in channel width or cross section. This chapter will discuss the methods available for assessing channel stability. An example scope of work for a stability analysis is given in Appendix F.

A channel is considered stable when the prevailing flow and sediment regimes do not lead to aggradation or degradation or to changes in the channel cross-sectional geometry over the medium to long term. It is important to recognize that short-term changes in sediment storage, channel shape, and planform are both inevitable and acceptable in natural channels with unprotected bank lines. Evaluation of stability can be undertaken at various levels, ranging from geomorphic assessments based on qualitative methods to quantitative techniques using numerical data and analytical techniques. There are three levels of stability assessment ranging from empirical reconnaissance-level methods to more process-based analytical techniques (Figure 32).

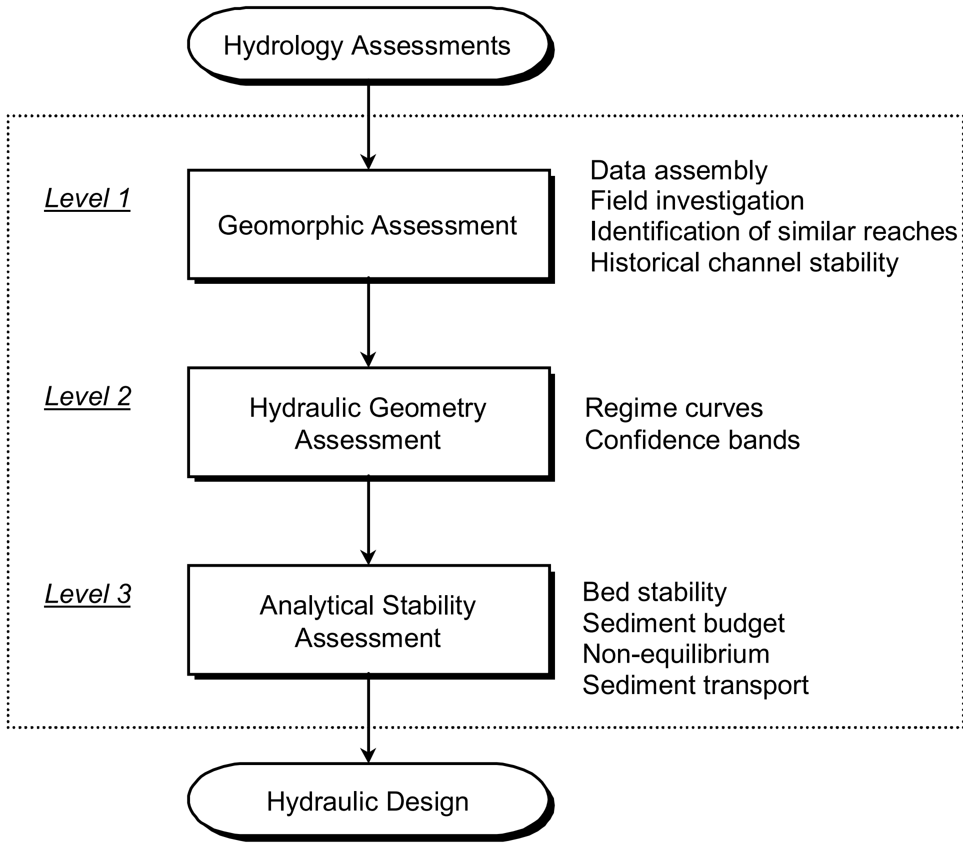

Figure 32. Levels of stability assessment

The appropriate level of detail for a particular evaluation depends on the status of the study, the perceived seriousness of potential problems, the scale of the project and the resources available (EM 1110-2-1418). This study procedure is consistent with the staged sediment study approach outlined in EM 1110-2-4000. In the Corps of Engineers, all three levels of the stability assessment are the functional responsibility of the Hydraulic and Hydrologic Engineering Sections and should be performed by an engineer with experience in river engineering and geomorphology.

Geomorphic Assessment

The geomorphic assessment provides the process-based framework to define past and present watershed dynamics, develop integrated solutions, and assess the consequences of remedial actions. This is an essential part of the design process whether planning bank protection for a single stream bank or attempting to develop a comprehensive plan for an entire watershed. A geomorphic assessment may be divided into the following three components: (a) data assembly; (b) field investigation; and (c) identification of geomorphologically similar reaches. Channel classification is also discussed under this heading.

Data assembly

The first step in the geomorphic assessment is to gather and compile existing data. Historical data are used to identify trends, to provide information on rates of landform change in the watershed, and to help the engineer determine land use impacts upon current conditions. Data requirements depend upon project objectives and watershed characteristics. Guidelines for data collection are provided in EM 1110-2-4000, EM 1110-2-1418, and Biedenharn et al. (1997).

Field investigations

Field reconnaissance is undertaken to gather data and make observations leading to an understanding of the active processes and condition of the stream. It is critical that experienced personnel conduct this effort. Field reconnaissance is used to describe the geomorphological landform of study reaches and to identify potentially destabilizing phenomena based on reach-scale evidence of erosion, sediment storage, and deposition. Basic information on how to conduct field investigations to collect data for channel stability assessment is contained in the following publications: EM 1110-2-4000; EM 1110-2-1418; Biedenharn et al. (1997), which contains detailed discussion on field equipment and a description of features to look for in the field; and Thorne (1993).

Collection of field data can be aided with the use of appropriate field assessment data sheets. Example data sheets are provided in Appendixes B and C. These sheets are comprehensive and should be adapted to specific study needs. Guidance for carrying out detailed reconnaissance surveys is given by Downs and Thorne (1996); Thorne, Simon, and Allen (1996); and Thorne (1998). The level of effort required to conduct a field reconnaissance varies depending on conditions. It is recommended that a consistent technique be utilized and that it be tailored to the watershed conditions and the study goals. It is also recommended that a trial run be conducted with a formulated field sheet to assess time requirements and assessment coverage before initiation of a large watershed-level field effort.

Field assessments are best made during low-water conditions and during the dormant season when the banks can be more readily examined. However, it is important to recognize that conditions may be different at high flows. For safety and logistical reasons, field work is best accomplished by teams of at least two people. Field work (particularly in urban areas) may raise significant health and safety issues. Potential hazards include crime, needles, and exposure to raw sewage and waterborne pathogens such as hepatitis. It is recommended that a minimal team consist of a biologist who is familiar with characteristics of aquatic and riparian habitat of the study area and an engineer who is experienced in hydraulics, hydrology, geomorphology, and sediment transport. Field work goes much better if at least one member of the team is familiar with the area. Inspections at bridge crossings should be treated with caution since bridges are frequently placed at constrictions and/or at bedrock outcrops - locations that may not be characteristic of the stream as a whole. However, valuable indicators of stream stability can be observed at bridges and other points where infrastructure crosses the stream. In assessing streams in the field, it is important to keep in mind that a channel typically has four degrees of freedom: width, depth, slope, and planform.

During the field reconnaissance at least the following basic information should be collected:

a. Descriptions of watershed development and land use, floodplain characteristics, channel planform, and stream gradient.

b. Assessment of historical conditions. This can be obtained via interviews with knowledgeable landowners.

c. Measurements of low-flow and bankfull channel dimensions and channel slope in critical reaches. Identification of terraces and active floodplains.

d. Characterization of the channel bed. Determine if it is bedrock, erodible cohesive material, armored or alluvial. Determine the gradation of any armor layer and collect bed-material samples of the substrate layer. Guidelines for collection of bed material samples are given in Appendix D.

e. Descriptions of riverbank profiles, bank materials, and evidence of bank instability.

f. Descriptions of point bars, pools, riffles, bed instability, and evidence of sedimentation processes.

g. Observations of impacts due to channel alterations and evidence of stream recovery.

h. Descriptions of channel debris and bed and bank vegetation.

i. Preliminary stream restoration alternatives should be identified so that information can be gathered on possible constraints such as access, utilities, staging areas.

j. Photographic records of critical stream and watershed characteristics.

There are many possible indicators of stream stability. A range of field indicators within a watershed is given in Table 1. It is important to recognize that these are not absolutes and that items listed as possible indicators of instability may occur in natural and/or stable streams and vice versa. Therefore it is important that those conducting the field assessment be experienced in the accurate interpretation of the results of stream reconnaissance.

Table 1 Possible Field Indicators of River Stability/Instability	
Evidence of Degradation	Terraces (abandoned floodplains) Perched channels or tributaries Headcuts and knickpoints Exposed pipe crossings Suspended culvert outfalls and ditches Undercut bridge piers Exposed or "air" tree roots Leaning trees Narrow/deep channel Banks undercut, both sides Armored bed Hydrophytic vegetation located high on bank
Evidence of Aggradation	Buried structures such as culverts and outfalls Reduced bridge clearance Presence of midchannel bars Outlet of tributaries buried in sediment Sediment deposition in floodplain Buried vegetation Perched main channel Significant backwater in tributaries Uniform sediment deposition across the channel Hydrophobic vegetation located low on bank or dead in floodplain
Evidence of Stability	Vegetated bars and banks Limited bank erosion Older bridges, culverts and outfalls with bottom elevations at or near grade Mouth of tributaries at or near existing main stem stream grade No exposed pipeline crossings

It is important to recognize the possible pitfalls of field assessments. These include observer bias, temporal limitations, and spatial limitations. Issues related to observer bias can be partially overcome with the consistent use of trained personnel. This practice will minimize relative differences between observations. Temporal bias can be minimized by examination of historical records, but these may be incomplete. Having the field team walk a continuous reach of stream can reduce spatial bias. Field investigation should extend both upstream and downstream of the project reach, and ideally should be conducted at several different periods of the year.

During the stream reconnaissance, it is important to locate and observe both stable and unstable areas within the particular study reach. By observing the areas that have the worst problems, one will be able to establish the upper limits of erosion, sedimentation, and/or flooding. It is equally important to visit reaches of the system where these problems are either not as apparent or absent. This will allow the engineer to define a total envelope of values associated with the study area and to understand the variability of the physical characteristics of the various reaches in the stream.

Identification of geomorphologically similar reaches

The information gathered in the data assembly and field investigation should be used to divide the channel into geomorphologically similar reaches. When establishing reach limits, consideration should be given to: differences in channel slope, tributary locations, presence of geologic controls, planform changes, location of channel control structures (grade control structures, dams, culverts, etc.), changes in bed material size, major sediment sources (gravel mines, sediment laden tributaries, etc.), changes in channel evolution type, or other significant hydrologic or geomorphic changes. Initial reach limits may be made early during the field investigation, but may be refined following more detailed analyses.

Assessment of reach condition

At the conclusion of a field investigation, a summary of channel stability in each reach is assessed. This summary may include the use of general typing and scoring techniques related to the existing condition of individual reaches. The many techniques available range in complexity and required effort. The choice of an assessment technique should be made with consideration of the study goals after the field investigations have been performed. An example of basic typing is as shown in Table 2.

Table 2
Reach Condition Assessment

Condition	Bed	Bank
Stable	The channel bed is as close to a stable condition as can be expected in a natural stream. If the reach exhibits signs of local bed scour or deposition with a low rate of change, it would fall into this category.	The channel banks are as close to a stable condition as can be expected in a natural stream and appear to have a low potential to erode. Banks are predominantly covered with extensive vegetation, boulders, or bedrock formations. If the reach exhibits signs of local bank erosion within an allowable rate of change, it would fall into this category.
Moderately stable	The channel bed in the reach is in a moderately stable condition. However, the reach may be in transition. Reaches where the bed is experiencing bed aggradation or degradation at a low rate of change would fall into this category. In addition, moderate to high local bed scour or deposition would fall into this category. For example, rapid aggradation immediately above and scour immediately below a minor debris blockage (such as a single tree blocking the channel).	The channel banks in the reach are in a moderately stable condition and exhibit medium erodibility. Banks are partially vegetated with moderately erodible soils. Typically, parallel flows would not result in bank erosion. The reach may be in transition. Reaches with banks that exhibit moderate local bank erosion that does not appear to be spreading would fall into this category. For example, in an otherwise stable reach, a single section of the bank could fall into the stream and result in local, moderate bank erosion.
Unstable	The channel bed in the reach is in an unstable condition. Reaches where the bed is undergoing widespread bed aggradation or degradation at a moderate rate would fall into this category. Moderately scoured reaches or reaches where many of the pools are filled with loose sediment would fall into this category.	The channel banks in the reach are predominantly unstable. Reaches where the banks are experiencing widespread erosion at a moderate rate would fall into this category. Reaches where the channel banks are undergoing local bank erosion at a high rate of change and where the erosion is not likely to be self healing would also fall into this category.
Very Unstable	The channel bed in the reach is in a very unstable condition. Typically the channel shows no signs of approaching equilibrium with the current shape and planform. Reaches where the bed is undergoing widespread aggradation or degradation at a high rate would fall into this category. Severely scoured reaches would fall into this category. Reaches where all of the pools are filled with loose sediment would also fall into this category.	The channel banks in the reach exhibit high erodibility and do not have any controls that would restrict extensive changes in planform or shape. Riparian root masses are not present to slow rapid bank retreat. Any parallel or impinging flows would cause extensive bank erosion. Reaches with near vertical to overhanging banks.

Channel typing and classification

Channel typing or classification is a useful though not essential step in channel assessment. A channel can be described in detail without selecting a classification system and assigning the stream reach to a certain class. Typing or classification is useful if one is developing or using hydraulic geometry relations with separate regression equations for different types of streams. Such relationships should result in regression equations with better accuracy and less uncertainty.

Determining a channel type relies on developing a channel description based primarily on observation. The channel description includes parameters such as channel and floodplain geometry, bed and bank material, planform, vegetation, bed forms, evidence of aggradation or degradation, grade control, alluvial or threshold conditions, etc. Channel typing is an elementary level of stream classification, using generic terms. For instance, a stream may be typed as a meandering sand bed channel.

Channel classification involves the selection of a classification system, normally developed by a specific person (e.g., Brice 1984 or Schumm 1977), and the categorization of a channel into a specific class based on factors and measurements such as planform and planform features, dominant mode of sediment transport, entrenchment ratio, sinuosity, etc. There are numerous stream classification systems. Some of the most widely used are described in EM 1110-2-1418 and in the Federal Interagency Stream Restoration Working Group (1998). Some limitations of stream classification systems include the following:

a. The classification is a "snapshot" of the existing condition of the stream and does not give any information about trends, such as whether a stream is stable, aggrading, degrading, or approaching a critical geomorphic threshold.

b. Water quality or the biological health of a stream cannot be determined from a geomorphic classification system.

c. The classification is a generalization of stream behavior, which the individual stream may conform to well or poorly.

Channel evolution models differ from classification systems in that they are used to predict sequential stages in channel response. For example, the incised-channel evolution model developed by Schumm, Harvey, and Watson (1984) predicts the sequence of changes which will occur in a channel as a headcut moves upstream. The model stages are shown in EM 1110-2-1418, Figure 2-23. Simon (1989) has developed a similar model. Channel evolution models can be used to predict trends (aggradation, degradation, and channel widening) at a project site, and to prioritize restoration work along a stream channel.

Regime-type relationships that express bankfull width as a function of bankfull discharge can be used to provide initial predictions for stable dimensions of restored channels. However, the equations are valid only for the stream type and range of parameters from which they were derived. Hence, when designing a stable channel it is essential to apply only the morphological equations appropriate to the stream type of the target restored channel. In the geomorphological stability assessment the existing channel stream type should be determined and an appropriate target stream type should be recommended based on characteristics from stable reference-reach sites. Classification of rivers might be used as a basis for typing the stream. There are many different methods of classifying alluvial rivers at the reach scale, ranging from simple descriptions to more comprehensive systems (see Federal Interagency Stream Restoration Working Group 1998). According to Thorne (1997), "the action of the driving variables of water and sediment inputs on the boundary conditions presented by the floodplain topography, bed sediments, bank materials and riparian vegetation produces the characteristic channel morphology of an unconfined alluvial stream." More comprehensive typologies are limited in practice because they require strong geomorphic insight and understanding to apply consistently and usefully (Thorne 1997) and in many cases there are insufficient morphological equations to match the number of subcategories. On this basis, it is recommended that channels should be typed according to the nature of bed sediments and bank characteristics and the typing used to guide engineers in choosing appropriate hydraulic geometry equations for use in stability assessments and channel restoration design.

In summary, data obtained during the field investigation and historical data collection can be used to determine the target stream type, in terms of boundary sediments, riparian vegetation and meander pattern. In many cases, the type and density of bank vegetation will be different from that present in the reference reaches due to ecological, aesthetic, and recreational objectives. It is important that target vegetation is identified prior to channel design as it influences flow resistance. Otherwise the stability status of the restored channel could be affected.

Methods for assessing historical channel stability

The analysis of historical data from stream gages, surveys, and mapping can give useful information about channel stability, any aggradation /degradation trends, rates of lateral movement, and planform changes. The review of aerial photographs taken at different time periods is a useful starting point. These are normally available for any site, even when gage data or historic surveys are absent. The use of historic data has some potential pitfalls, however, especially when comparing surveys performed several years apart, or gage data with gaps in the record. For example, the fact that the existing thalweg is lower than the historic thalweg normally indicates that degradation is the dominant process, but it does not always indicate that the stream is currently degrading. The stream may have degraded to a point below the existing streambed, reversed its trend of instability and then aggraded so that the existing dominant process is aggradation (Schumm, Harvey and Watson 1984). Results of the historical data analysis should be compared to both the results of the field investigation and the analytical stability assessment before reaching final conclusions.

Specific gage analysis

If gage data are available, one of the most useful tools available to the engineer and geomorphologist for assessing the historical stability of a river system is the specific gage record. A specific gage record is a graph of stage for a specific discharge at a particular gaging location plotted against time. A channel is considered to be in equilibrium if the specific gage record shows no consistent increasing or decreasing trends over time, while an increasing or decreasing trend is indicative of aggradation or degradation, respectively.

The first step in a specific gage analysis is to establish the stage-discharge relationship at the gage for the period of record being analyzed. A rating curve is developed for each year in the period of record. A regression curve is then fitted to the data and plotted on the scatter plot. Once the rating curves have been developed, the discharges to be used in the specific gage record must be selected. This selection will depend largely on the objectives of the study. It is usually advisable to select discharges that encompass the entire range of observed flows. A plot is then developed showing the stage for the given flow plotted against time.

Specific gage records are an excellent tool for assessing the historical stability at a specific location. However, specific gage records indicate only the conditions in the vicinity of the particular gaging station and do not necessarily reflect river response upstream or downstream of the gage. Therefore, the specific gage record should be coupled with other assessment techniques in order to assess reach conditions, or to make predictions about the ultimate response of a river.

Comparative surveys and mapping

One of the best methods for directly assessing channel changes is to compare both channel thalweg and cross sections. Thalweg surveys are taken along the channel at the lowest point in the cross section. Comparison of several thalweg surveys taken at different points in time allows the engineer and geomorphologist to chart the change in the bed elevation through time and track the migration of headcuts or aggradation zones through the system. Cross-section surveys provide information about channel widening or narrowing.

There are certain limitations that should be considered when comparing surveys on a river system. When comparing thalweg profiles it is often difficult, especially on larger streams, to determine any distinct trends of aggradation or degradation if there are deep scour holes, particularly in bendways. The existence of very deep local scour holes may completely obscure temporal variations in the thalweg. This problem can sometimes be overcome by eliminating the pool sections and focusing only on the crossing locations, thereby allowing aggradation or degradation trends to be more easily observed. Reliable survey comparisons can be made only if the surveys are homologous in rivers and streams that have significant bed form movement and/or seasonal variations in sediment transport.

While thalweg profiles are a useful tool it must be recognized that they reflect only the behavior of the channel bed and do not provide information about the channel as a whole. For this reason it is usually advisable to study changes in the cross-sectional geometry. Cross-sectional geometry refers to width, depth, area, wetted perimeter, hydraulic radius, and channel conveyance at a specific cross section.

If channel cross sections are surveyed at permanent monumented range locations, then the cross-sectional geometry can be compared directly for different time periods. At each range, the cross section plots for the various time periods can be overlaid and compared. When available cross sections are not located by permanent monuments it is often advisable to compare reach average values of the geometric parameters. This requires the study area to be divided into distinct reaches based on geomorphological characteristics. Next the cross-sectional parameters are calculated at each cross section and averaged for the entire reach. Then the reach average values can be compared for each survey period. Cross-sectional variability between bends (pools) and crossing (riffles) can obscure temporal trends, so it is often preferable to use only cross sections from crossing reaches when analyzing long-term trends of channel change.

Comparison of time sequential maps or aerial photographs can provide insight into planform evolution, and change or instability of the channel. Rates and magnitude of channel migration (bank caving), locations of natural and manmade cutoffs, and spatial and temporal changes in channel width and planform geometry can be determined from analysis of historical information. With this type of data, channel response to imposed conditions can be documented and used to substantiate predictions of future channel response to a proposed alteration. Contemporary planform data can be obtained from aerial photos, maps, or from field investigations.

Hydraulic Geometry Assessment

Background

A common component of empirical approaches to stable channel design rests on downstream hydraulic geometry analysis. This approach employs a statistical treatment of data sets linking flow regime, sediment characteristics and resulting channel forms under dynamically stable conditions. Hydraulic geometry theory is based on the concept that a river system tends to develop in a way that produces an approximate equilibrium between the channel and the inflowing water and sediment load (Leopold and Maddock 1953). The stable channel does not change significantly in profile, cross section, or planform characteristics over the long term. Stable does not mean static: a stable channel may be actively meandering. Since many natural channels are stable over a wide range of flows, the empirical hydraulic geometry relations used to describe them have been of great interest to river engineers. Hydraulic geometry relations typically correlate an independent or driving variable, such as discharge or drainage area, to dependent variables such as width, depth, slope, and velocity. These relations are empirically derived, and their development requires a relatively large amount of data. EM 1110-2-1418 discusses the historic development, limitations, and application of hydraulic geometry relations. The development of hydraulic geometry relations and their use for stability assessment will be discussed in the following section.

Developing hydraulic geometry relations

The development of hydraulic geometry relations for a watershed or region is not a trivial task. It is best performed by engineers and geomorphologists with extensive experience in the region. Some excellent examples of regional hydraulic geometry studies are Emmett (1975); Charlton, Brown, and Benson (1978); Bray (1982); and Hey and Thorne (1986). Hydraulic geometry data should be collected in stable, alluvial reaches. Channel dimensions are typically treated as dependent variables and are best determined from field surveys. Discharge is typically the independent variable.

Hydraulic geometry relations can be developed for a project reach, a stream, a watershed, or a physiographic region. The various sources of data are listed and described as follows in order of preference:

a. Given the natural variation of stream and watershed characteristics, the preferred source of data for a project reach is the reach itself. This choice may not be feasible, either because the reach is not stable and alluvial, or because the reach has been altered.

b. The second preferred choice is data from the same stream at stable, alluvial reaches.

c. The third choice is data from other streams in the project watershed, although care must be taken to ensure that data are acquired from portions of the watershed with physiographic conditions similar to those of the project reach.

d. The fourth choice is relations developed for a different watershed in a similar physiographic region.

e. Generalized relations, or relations developed for other parts of the country, are a last choice, and should be evaluated on a case-by-case basis. For example, relations developed for the Piedmont region of Virginia may be transferable to the Piedmont region of Maryland. But relations developed for the glaciated areas of northern Pennsylvania would probably not be transferable to the nonglaciated areas of the same state. The use of hydraulic geometry relations outside the area in which they were developed is discussed in more detail in the following paragraphs.

Choice of independent variables

The fundamental assumption of hydraulic geometry theory is that the shape of a channel can be related to measurable or predictable hydraulic parameters. Therefore, cross-sectional form is inherited from the imposed natural sequence of water and sediment inflows and boundary conditions. As the discharge usually explains most of the variance in geometry, bankfull width, depth, and velocity are normally plotted as dependent variables against discharge. Although drainage area is often used as an independent variable (due to its ease of measurement), it is merely a surrogate for discharge, and may be poorly correlated with bankfull discharge within a watershed. While these relationships may be used to provide rough estimates of channel dimensions at ungaged sites, they should be applied with caution if used to design stable channels. The choice between drainage area and discharge as the independent variable also depends on the processes occurring in the watershed. For instance, in an urbanizing watershed, the relationship between discharge and drainage area will vary both spatially and over time, making drainage area a poor choice for an independent variable.

Use of stream typing systems to refine hydraulic geometry relations

In general, data sets used in hydraulic geometry analysis are regionally-based and apply to a particular locality rather than a stream type. Consequently, applying the resultant morphological equations beyond the parent region must be exercised with extreme caution. Alternatively, hydraulic geometry relations developed for various subsets of streams within a classification system based on bed and bank sediment and vegetation characteristics could reasonably be expected to have less scatter since some of the secondary factors are taken into account.

Transfer of hydraulic geometry relations from one watershed to another

The transfer of hydraulic geometry relations developed for one watershed to another watershed should be performed with care. The two watersheds should be similar in historical land use, physiography, hydrologic regime, precipitation, vegetation, etc. For example, relations developed for pristine watersheds should not be transferred to urban watersheds. Relations developed for areas with snowmelt hydrology should not be transferred to areas dominated by convective storms. Since discharge is the variable that shapes the channel, relations based on discharge can be transferred with more confidence than those based on drainage area.

Special problems of urbanized streams

Urbanized streams present particular problems in both the development and the application of hydraulic geometry relations. Land use and runoff characteristics usually vary greatly, even within a single watershed. The multiplicity of manmade structures, such as storm sewers, bridge openings, culverts, and stormwater management facilities, changes the amount, duration, and timing of flows. This would be expected to greatly increase data variability. (These factors make discharge more poorly correlated with drainage area, and, hence, would make discharge the better choice than drainage area as an independent variable.) Locating stable, alluvial reaches may be difficult.

Uncertainty in hydraulic geometry relations

A sufficient number of data points must be measured to ensure that the results from hydraulic geometry analysis are statistically valid. For example, if any three or four random data points were used, a different relation could easily be derived. The fewer and more widely scattered the data points, the less confidence one has in any derived trend. Even with quite a few data points in a relatively homogeneous watershed, there is a great deal of scatter in the data due to natural variability.

Natural rivers which are in regime have stable morphologies that broadly conform to regime or hydraulic geometry relationships, linking the dependent parameters of channel form to independent controls of flow regime, boundary materials, and riparian vegetation. However, rivers do not follow regime laws precisely. In fact, every river displays local departures from the expected channel form described by morphological equations and possesses inherent variability in space and time. While it is true that natural channel forms are in general predictable, it is also true that each river is in detail unique. Regime dimensions in the natural domain should be interpreted only as representative reach-average, ideal or target conditions about which channel morphology fluctuates in time and space.

The coefficient of determination, r^2, in hydraulic geometry analysis numerically represents the amount of variation that can be explained by the selected independent variable. The lower the r^2 value, the less useful the relation is (and the wider the scatter in the data). The natural variability of data in a relatively homogeneous watershed such as the upper Salmon River watershed (Emmett 1975) underlines the importance of viewing the data used to develop the curve (not just the curve itself), along with statistical parameters such as r^2 values and confidence limits. Equations given without plotted data points or statistical parameters should be verified for applicability.

Statistical confidence bands can be used effectively to introduce nonuniformity into restored channel designs and have been applied for this purpose by Soar and Thorne (2001). Advanced texts on probability and statistics describe standard methods of computing correlation coefficients and setting confidence limits on data (e.g., Myers 1990; Graybill and Iyer 1994).

Application of hydraulic geometry relations to assess channel stability

Hydraulic geometry analysis can be used in a geomorphic assessment of the study reaches to provide semiquantitative information on channel stability and sensitivity to change. The hydraulic geometry observed for the existing channel may be compared to that predicted for a stable channel in a reference reach using new or existing equations and associated bands of uncertainty. If the data for the project reach fall outside the 95 percent confidence band applied to the reference hydraulic geometry data, then there is reason to believe that the project reach in question may be unnatural or unstable. However, this method should be used only to provide an indication of stability because data points that lie far from the best-fit regression line could be influenced by other factors such as geology, land use, or vegetation that are not common to the rest of the data set.

The use of hydraulic geometry relations and confidence bands to assess the stability of a given channel reach requires that the watershed and stream channel characteristics of the reach in question are not dissimilar to the reference data set used to develop the hydraulic geometry relations and that the data scatter is known, so that confidence bands can be derived. When applying this stability assessment, the two most reliable hydraulic geometry equations are those expressing bankfull width as a function of bankfull discharge, for different types of bed and bank characteristics, and meander wavelength as a function of bankfull width. These relationships exhibit the least variability as opposed to other combinations of the dependent and independent variables (for examples see Hey and Thorne 1986 and Williams 1986).

In summary, the application of downstream hydraulic geometry relationships requires that the actual data be plotted and the statistical coefficients calculated. Hydraulic geometry relations and associated confidence bands can be used as a preliminary guide to indicate potential stability or instability in stream reaches, but these indications should be checked using other techniques due to the wide natural variability of the data.

Analytical Stability Assessment

Observations and hydraulic geometry relations may be used to identify possible stability problems, but analytical methods are required to determine the magnitude of a stability problem. An analytical stability analysis requires calculation of hydraulic parameters such as velocity and shear stress for the range of natural discharges. The hydraulic resistance of the channel boundary is determined from field observations and measurements. Sufficient field sampling of the streambed should be conducted to determine the spatial variability, size, and gradation of the bed material. Sediment inflow is estimated from measured data or by calculation.

Hydraulic calculations

Hydraulic parameters can be determined using normal depth assumptions or by a more rigorous backwater analysis. The SAM hydraulic design package (Thomas et al. 2000) can be used to average hydraulic parameters if normal depth assumptions are adequate. There are several available computer programs, including HEC-2, HEC-RAS, WSPRO and HY-22 if a gradually varied, steady flow assumption is more appropriate.

Normal depth. The SAM hydraulic design package is available to calculate normal depth in a compound channel with variable hydraulic roughness. Several complex channel compositing schemes are available for the calculation. Channel hydraulic parameters are calculated separate from overbank hydraulic parameters and effective channel hydraulic parameters are calculated for use in sediment transport relationships. Hydraulic roughness can be varied across the cross section and different roughness equations can be used for different portions of the cross section.

Reliability of the normal depth calculation is directly related to the reliability of the input data. Sound engineering judgment is required in the selection of a representative cross section. The cross section should be located in a uniform reach where flow is essentially parallel to the bankline (no reverse flow or eddies). This typically occurs at a crossing or riffle. Determination of the average energy slope can be difficult. Thalweg slopes and low-flow water-surface slopes may not be representative of the energy slope at morphologically significant flows. Slope estimates should be made over a significant length of the stream (a meander wavelength or 20-channel widths). Hydraulic roughness must be estimated based on field observations and measurements. Several techniques are recommended in EM 1110-2-1601 and the SAM Users Manual (Thomas et al. 2000).

Backwater analysis. Hydraulic models are used to calculate water-surface profiles, flow and lateral velocity distributions, flow regimes, and scour potential. For stream restoration projects that are likely to involve revisions to the Federal Emergency Management Agency's (FEMA) flood insurance rate maps, selection of the hydraulic model should be coordinated carefully with FEMA. Some standard hydraulic models are discussed in the following paragraphs.

HEC-RAS. HEC-RAS (USACE, HEC 2001) is the recommended model for performing hydraulic calculations for steady, gradually varied (over distance), one-dimensional, open channel flow. HEC-RAS includes a culvert module that is consistent with HDS-5 and HY-8. The bridge hydraulics algorithms now include the WSPRO models. HEC-RAS applies conservation of momentum, as well as energy and mass, in its hydraulic analysis. HEC-RAS includes all the features inherent to HEC-2 and WSPRO plus several friction slope methods, mixed flow regime support, automatic n value calibration, ice cover, quasi 2-D velocity distribution, and superelevation around bends.

HEC-2. HEC-2 (USACE, HEC 1990) performs hydraulic calculations for steady, gradually varied (over distance), one-dimensional, open channel flow. One of HEC-2's technical limitations is the normal bridge routines and standard-step backwater computations use energy conservation only. Conservation of momentum is used only in the special bridge routines when there are bridge piers.

WSPRO. The WSPRO computer program was developed by the U.S. Geological Survey (USGS) and is comparable to HEC-2, except for the fact that WSPRO had special subroutines for analysis of water-surface profiles at bridge locations. All of these WSPRO subroutines have been incorporated into

HEC-RAS. The current version of WSPRO must be used with caution since it has known bugs and is no longer being supported.

HY-22. HY-22 is a small tool kit of relatively simple computer programs for performing the hydraulic analyses described in the "Urban Drainage Design Manual," Hydraulic Engineering Circular No. 22, FHWA (Brown 1996). HY-22 includes pavement drainage, open channel hydraulics, critical depth computation, computation of storage volume, and simple reservoir routing.

Bed stability

After hydraulic parameters have been calculated for a range of discharges, it is important to determine the discharge at which the streambed begins to move. This can be accomplished using the threshold criteria described in EM 1110-2-1418. This step is especially important in a channel with an armor layer. Sediment transport capacity dramatically increases when the armor layer is disrupted or destroyed and the coarse material becomes thoroughly mixed with the substrate material. Stability of vegetated or gravel banks can be determined using allowable velocity methods or shear stress methods. A mobile streambed is not necessarily unstable, but mobile beds require a higher level of analysis to determine stability.

Sediment rating curve analogy analysis

The sediment-rating curve analogy analysis is a relatively simple technique that can be used to assess the sediment transport characteristics of an existing or proposed stream project. The basic approach is to assess the sediment transport character of a study reach by comparing its sediment transport capacity to that of its supply reach. If the supply reach is not fully alluvial, a reference reach may be used as a surrogate for the supply reach. The sediment rating curve analogy analysis is suitable for streams where the sediment supply is not limited, that is where the stream is alluvial. It is generally not suitable for threshold streams.

This qualitative technique does not require stream gage data or sediment gage data. It does require an estimate of the supply reach grain size distribution, an estimated range of peak flows, and a description of hydraulic characteristics of both the study and supply reaches. Hydraulic information can be based on normal depth calculations or hydraulic modeling. Peak flows can be estimated using regional regression curves or hydrologic modeling. Sediment transport capacity is calculated for a range of discharges in both the existing and supply reaches. By comparing the sediment rating curves of the two reaches, an estimate can be made of the sediment transport capacity of the study reach relative to the capacity of the sediment supply reach. This is illustrated in Figure 33.

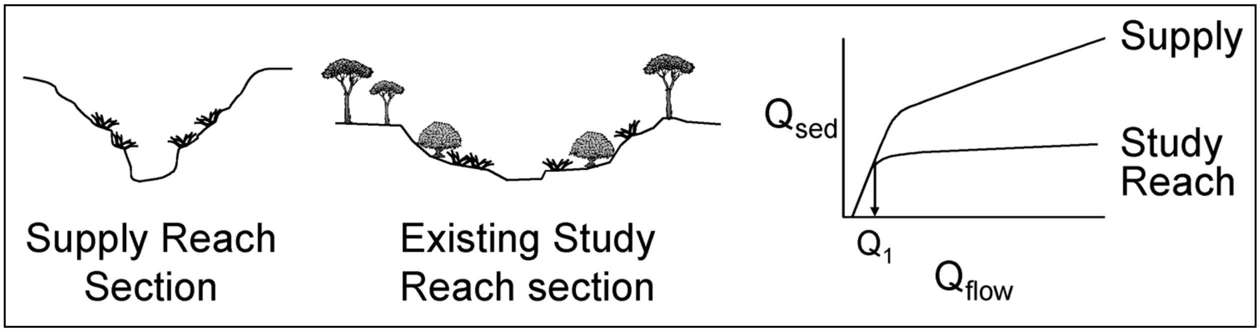

Figure 33. Sediment rating curve analogy analysis of existing conditions

The comparison of the supply reach and study reach sediment rating curves shown in Figure 33 indicates that there is a strong possibility that the existing study reach is depositional for flows above Q_1. This estimated condition should be checked by field observations to detect evidence of an aggradational trend. To improve channel stability the sediment rating curve for the project channel should be as close as possible to the supply reach sediment rating curve.

Sediment budget analysis

Channel stability is ultimately determined by the ability of the channel to pass the incoming sediment load while not scouring its bed. If sediment transport capacity is less than sediment supply then the channel will aggrade. On the other hand if the capacity is greater than the supply and the bed is alluvial then the channel will degrade. A determination of the potential for aggradation or degradation in a channel reach requires an assessment of the reach-scale sediment budget. The sediment budget compares the quantity of sediment transported into the reach with the sediment transport capacity of the reach. This is accomplished using the magnitude and frequency of all sediment-transporting flows. The following steps are recommended for conducting a sediment budget analysis.

a. Assemble information about the stream. This includes geometric, sedimentation, and hydrologic information. Missing data may be filled in from detailed site reconnaissance completed during a geomorphic assessment.

b. Calculate hydraulic parameters for a typical or average reach for a range of discharges. This range should extend from the average annual low flow to the peak of the design flood. Average hydraulic parameters can be determined from HEC-2 results using SAM.m95; or from normal depth calculations for a designated typical cross-section geometry using SAM.hyd.

c. Select an appropriate sediment transport function for the study reach. This can be achieved by comparing calculated sediment transport to measured data, taking care to ensure that bed-material load is being compared. When no data are available, one may rely on experience with similar streams in the region. SAM.aid will designate the best sediment transport equation for rivers with similar hydraulic and sediment characteristics.

d. Calculate three sediment transport rating curves for the existing channel in the assessment reach, upstream of the assessment reach (the supply reach), and downstream from the assessment reach. Sediment transport rating curves should also be determined for any tributaries that might be affected by the assessment reach.

e. Calculate sediment yield for the supply reach and the assessment reach and the downstream reach using the flow-duration sediment discharge rating curve method. This should be done using a flow-duration curve to obtain average annual sediment yield and a flood hydrograph to obtain yield during a flood event.

f. Calculate trap efficiency by comparing the supply reach and assessment reach sediment yields. Also calculate trap efficiency for the assessment reach compared to the downstream receiving channel. A positive trap efficiency indicates deposition and a negative value indicates erosion. If the assessment reach is stable the trap efficiency is near zero.

The preferred method for calculation of average annual sediment yield is the flow-duration sediment-discharge method described in EM 1110-2-4000, Chapter 3. This method requires sufficient gage data to develop the flow-duration curve and requires either measured bed-material load data or calculation of a sediment-discharge rating curve using an appropriate sediment transport relationship.

Often sufficient gage data are not available to calculate a flow-duration curve for the project reach. In these cases, there are two approaches that can be used to compute average annual sediment yield. The first is to synthesize a flow-duration curve using the drainage-area flow-duration curve method or the regionalized duration method (Appendix A), and then use standard methods to compute sediment yield. The second approach is to compute sediment yields for hydrographs of various frequencies and then weight them according to their probability of occurrence. This is not a frequently used method but is discussed in Appendix E because of its usefulness for certain applications.

Nonequilibrium sediment transport

HEC-6 (USACE, HEC 1993) is a one-dimensional moveable boundary open channel flow numerical model designed to simulate and predict changes in river profiles resulting from scour and deposition over moderate time periods, typically years, although applications to single flood events are possible. A continuous discharge record is partitioned into a series of steady flows of variable discharge and duration. For each discharge, a water-surface profile is calculated, providing energy slope, velocity, depth, and other variables at each cross section. Potential sediment transport rates are then computed at each section. These rates, combined with the duration of the flow, permit a volumetric accounting of sediment within each reach. The amount of scour or deposition at each section is then computed and the cross-section geometry is adjusted for the changing sediment volume. Computations then proceed to the next flow in the sequence and the cycle is repeated using the updated cross-section geometry. Sediment calculations are performed by grain size fractions, allowing the simulation of hydraulic sorting and armoring.

HEC-6 is a powerful tool that allows the designer to estimate long-term response of the channel to a predicted series of water and sediment supply. The primary limitation is that HEC-6 is one-dimensional, i.e., geometry is adjusted only in the vertical direction and average hydraulic parameters are assumed in the computations. Changes in channel width or planform cannot be simulated.

Integration and application of results

The final part of a stability assessment of a channel system is accomplished by integrating the information from all the available analyses. Analysis using each of the geomorphic tools discussed previously may yield a verdict of aggradation, degradation, or dynamic equilibrium with respect to the channel bed, and stable or unstable with respect to the banks. Often the individual assessments produce contradictory results. For instance, the field investigations might indicate that a channel reach is vertically stable, but the empirical relationships and SAM results indicate that the channel should be degrading. In this case one would have to assign a level of confidence to the various components based on the reliability and availability of the data, and one’s own experience with each tool in order to reconcile these contradictory results. Once again, it is obvious that there is no cookbook answer, and that sound judgement based on insight and experience must always be incorporated when making a stability assessment.

The information gained from the channel stability assessment can be applied to determine potential evolutionary trends in the stream system. This is dependent on having a clear understanding of the dominant geomorphic processes at work in the watershed, and a conceptual model of how the stream system reacts to imposed changes. For example, the incised channel evolution model (Schumm, Harvey, and Watson 1984) is a conceptual model of the reaction of a stream system to a base-level lowering without changes in the upstream land use or sediment supply. In the channel evolution model, the evolution of the stream (at any one point) follows a predictable series of stages. In watersheds where this model applies, the engineer can predict the future evolution of various channel reaches, based on an assessment of the current channel condition gained from the channel stability assessment. However, in many watersheds, the effects of base level lowering may be combined with other perturbations, such as increased runoff or decreased sediment supply and this will cause a more complicated response in the stream system than is described in the incised channel evolution model. The engineer should attempt, as much as possible, to develop a conceptual understanding which explains the historic and future evolution of the streams within the study watershed.

The scope of a hydraulic analysis of a stream restoration project will vary depending on the stage of the planning process and the magnitude of channel instability problems. Appendix F is an example scope of work for a stability analysis that might be conducted early in the planning process to define dominant geomorphological processes.

5 Hydraulic Design Methodology

Design Discharges

In order to design a stream restoration project with long-term stability which is sustainable without the need for maintenance dredging or grade control, it is necessary to evaluate the full range of flows that will affect the channel. A stream restoration project usually has several design flows selected to meet various objectives. A narrow deep channel may be designed for lower dry-season flows (base flow) to meet habitat requirements during biologically critical periods. Channel dimensions for the main channel may be selected to convey a flow crucial to channel stability (channel-forming discharge), while project features, such as bank protection and habitat enhancement structures, may be designed to withstand a significant flood event, normally a 10 percent chance exceedance discharge or larger. The appropriate types of design discharges for different project elements are discussed in the following sections. Although a particular project may not require the use of all of these flows for design, the engineer should still consider how the project will perform during low, intermediate, and high flows.

Design discharge for low flows

Normally, biological objectives drive project design for low flows. For instance, under many hydrologic regimes, summer low flows are often a critical period for fish, and project goals may include narrowing the low-flow channel to provide the increased depths necessary to support the fish population. Design flows may also be necessary to provide the depths and velocities essential for fish spawning or fish passage during other critical times of the year. Coordination with study team biologists is essential to make sure an appropriate flow (or range of flows) is selected. Design of a low-flow channel may be required as part of a channel modification. The 7-day annual low flow is often used for critical low-flow design. Guidance for flow depths and velocities required or tolerated by a wide variety of fish species can be found in Bell (1986) and Morrow and Fischenich (2000).

Main channel discharge

If the stream channel is realigned or reconstructed, a suitable design discharge must be selected for an initial estimate of reach-averaged stable channel dimensions. This is normally larger than the one-year frequency event. Issues related to the selection of a channel-forming discharge for stable channel design are covered in Chapter 3. However, project constraints may not allow for a channel that carries only the channel-forming discharge. For instance, a channel that is larger than the regime channel may be required for flood conveyance. In such cases, a compound channel may be designed with a main channel that has other than ideal dimensions. Constraints that influence the design discharge for the main channel may also include the capacity of the upstream and downstream channels, utilities or rights-of-way that limit width, slope or alignment, and flooding concerns. It should be noted that stable channel design includes the evaluation of sediment transport capacity for a range of flows (not just the design discharge) to determine long-term maintenance requirements and whether the project is likely to aggrade or degrade significantly in the future.

A single channel-forming discharge can be estimated by determining the bankfull flow, calculating the effective discharge or selecting a specific recurrence interval discharge. However, inspection of a natural channel reveals that variability is inherent to natural fluvial systems. Hence, when designing channels that are intended to replicate natural channel features, but also remain stable over long periods of time, it is important to establish both the degree of local morphological diversity expected for the channel and its stability over a range of discharges.

After a preliminary design has been prepared, channel stability checks may include simulation of sediment transport under either selected hydrologic events or the entire flow-duration curve for the available period of record. This type of analysis will indicate whether the channel will experience unacceptable levels of scour or deposition during discharges above and below the design flow, and whether aggradational or degradational trends will be significant within the life span of the project.

Habitat and hydraulic structure design discharge

Constraints such as floodplain development or flood damage reduction requirements mean that successful stream restoration often includes bank protection, grade control, and in-stream construction of habitat features. Living plant materials are often used in association with inert materials, such as timber or rock, and manufactured products. To accomplish a reasonably self-sustaining holistic ecosystem, a combined technology approach is required. Sound physical principles and well established engineering formulae are used in the analysis and design of both soft and hard features.

A significant flood event (normally no smaller than the 10-year frequency discharge) is used to size stone and compute scour depths. The goal is that the hard project features will withstand a flood of this magnitude without major damage, movement, or flanking. Impacts that might reasonably be expected during a flood event would be deposition of sediment and debris; combined with local and general scour, erosion and stone movement; and destruction of vegetation. Often in urban settings and flood damage reduction channels the one-percent chance exceedance discharge is used to size stone and compute scour depths. In addition, the impact of the project on flood elevations and conveyance must be evaluated. Often the impact on the water-surface profile for the one-percent chance exceedance discharge must be submitted as part of the project's permitting requirements. It may also be necessary to compute the impact of the project on more frequent flood events, or for a larger event.

Threshold Channels

As defined herein, a threshold channel is a channel in which channel boundary material movement is not a stability issue during the design flow. The term threshold is used because the channel geometry is designed such that applied forces from the flow are below the threshold for movement of the boundary material. This class of stream includes cases where the streambed is composed of very coarse material or erosion resistant bedrock. Streams where the boundary materials are remnants of processes no longer active in the stream system may be threshold streams. Examples are streambeds formed by high runoff during the recession of glaciers or dam breaks and streams armored due to reduction in the upstream sediment supply and degradation. Fine sediment may pass through threshold streams as throughput or wash load. In general, this sediment should not be considered part of the stream boundary for stability design purposes even if there are temporary deposits on the streambed at low flow. However, throughput or wash load may be an environmental issue. Threshold channels do not have the ability to adjust their geometry, as do alluvial channels, because the material forming the channel boundary is unerodible under the normal range of flows, and there is no significant exchange of material between the sediment carried by the stream and the bed. At flows larger than the design flow or during extreme events, threshold channels may become destabilized for short periods, with harmful morphological impacts and this possibility must be borne in mind.

There is not always a clear distinction between threshold and alluvial channels. One reach of the stream may be alluvial while another has the characteristics of a threshold channel. A threshold stream reach can be changed to an alluvial reach by flattening the slope. A stream may be alluvial at low discharges when there is an adequate sediment supply and then act like a threshold channel at high discharges. If an armor layer is present, a stream may be a threshold channel at low flows and on the rising limb of a flood hydrograph, but an alluvial channel at high flows when the armor layer is mobilized, and on the falling limb of the flood hydrograph when sediment is being deposited. It is therefore important to evaluate channels through their entire flow range to determine how they will react to natural inflow conditions and how their stability status may change as a function of discharge.

Hydraulic design methods for threshold channels are well established and available from several sources. Maximum permissible velocity methods are applicable for a variety of boundary materials and guidance can be found in

EM 1110-2-1601 and EM 1110-2-1418. The U.S. National Resource Conservation Service (NRCS) has developed allowable velocity design procedures for drainage channels (USDA 1977 and EM 1110-2-1418). The U.S. Bureau of Reclamation developed the tractive force method (Lane 1955; and EM 1110-2-1418) for design of irrigation canals, primarily with gravel beds. Threshold design methods do not provide unique solutions for channel dimensions of width, depth, and slope. However, this limitation is not critical to the hydraulic design in terms of stability because the boundary is immobile.

The concepts of channel-forming discharge and hydraulic geometry are generally not applicable to threshold channels because these channels do not adjust their dimensions to the natural runoff hydrograph.

Theoretical threshold-channel design methods have been developed that consider the lateral turbulent diffusion of downstream momentum in a cross section with a laterally-uniform bed gradation (Parker 1978; Parker 1979; Ikeda, Parker, and Kimura 1988; Ikeda and Izumi 1990; and Diplas and Vigilar 1992). These methods are discussed by the ASCE Task Committee on Hydraulics, Bank Mechanics, and Modeling of River Width Adjustment (1998).

Threshold methods are also used to design stream features such as bank toe protection, riffles, and deflector dikes. The Corps' riprap design procedure (EM 1110-2-1601) is appropriate for design of these features. This procedure allows for use of rounded stone as well as angular stone more commonly used in flood-control projects.

Alluvial Channels

Alluvial streams have bed and banks formed of material transported by the stream under present flow conditions. There is an exchange of material between the inflowing sediment load and the bed and banks of the stream. Alluvial channels adjust their width, depth, slope and planform in response to changes in water or sediment discharge.

The hydraulic design methodology described herein is intended for cases where an historically stable channel has been realigned creating instability, or where hydrologic and/or sediment inflow conditions have changed so much that the channel is currently unstable. A stream is defined as stable when it has the ability to pass the incoming sediment load without significant degradation or aggradation and when its width, depth, and slope are fairly consistent over time. Bank erosion and bankline migration are natural processes and may continue in a stable channel. When bankline migration is deemed unacceptable, then engineering solutions must be employed to prevent bank erosion. Bank protection technology is not addressed in this report, but a review of issues and design considerations can be found in Biedenharn et al. (1997).

Alluvial channel design variables

The hydraulic design variables of width, depth, slope, and planform are dependent variables in an alluvial channel. Their magnitudes are determined by the independent variables of sediment inflow, water inflow, and bank composition. The downstream water-surface elevation is an independent variable that could have a significant effect on the dependent variables in some cases. Boundary resistance along the channel banks and sometimes along the bed can be both dependent and/or independent depending on local circumstances. The hydraulic design methodology provides a method for determining the magnitude of the dependent variables given the magnitudes of the independent variables.

The design philosophy is to employ the best available physically based methodologies to determine the design variables. Average magnitudes for width, depth, and slope are determined first. The initial or preliminary average channel geometry is initially determined using a single channel-forming discharge. Later a full range of discharges is used to evaluate the channel design, and the initial design may be adjusted. Analytical techniques are employed to ensure that the combination of design variables are compatible. With three unknowns, three equations are required to determine the magnitude of each design variable. A hydraulic resistance equation, such as Manning's equation, can be one design equation. A sediment transport equation, such as Brownlie's equation, can be the second design equation. Resistance and sediment transport equations are well established and can be used with a reasonable level of confidence in the design process. One additional equation is needed. Four alternatives are considered herein for this third equation: (a) analogy methods, (b) hydraulic geometry relationships, (c) constraint of one of the variables, or (d) adopting an extremal hypothesis.

When channel width is not constrained by rights-of-way limitations, the preferred method for determining one of the unknown dependent variables is to apply geomorphic principles such as an analogy method or a hydraulic geometry relationship. Several techniques are available.

Analogy methods. If the existing channel is stable in the project reach an attempt should be made to retain the same channel geometry in the restored channel.

If the channel is unstable in the project reach, a design top width for the stable project channel can be determined by assigning a measured average top width from a reference reach. The reference reach must be stable and alluvial and have the same channel-forming discharge and boundary conditions as the project reach. The reference reach may be upstream and/or downstream from the project reach, or in a different but physiographically similar watershed. The bed and banks in the project and reference reaches must be composed of similar material, and there should be no significant hydrologic, hydraulic, or sediment differences between the reaches. This technique is inappropriate for streams where the entire fluvial system, or a significant part of it, is in disequilibrium.

An alternative to the reference reach approach is to reconstruct the channel to a stable predisturbance width and planform. This is feasible if historical width and planform information can be determined from mapping, aerial photos, and/or soil borings. However, this technique is inapplicable if the watershed water and sediment runoff characteristics have changed over time, as the historically stable channel form will no longer be stable in the current watershed context.

Hydraulic geometry methods. Hydraulic geometry theory is based on the concept that a river system tends to develop in a predictable way, producing an approximate equilibrium between the channel and the inflowing water and sediment (Leopold and Maddock 1953). The theory typically relates a dependent variable, such as width or slope, to an independent or driving variable, such as discharge or drainage area. Hydraulic geometry relations are sometimes stratified according to bed material size, bank vegetation, or bank material type. Hydraulic geometry relationships are developed from field observations at stable and alluvial cross sections. These relationships were originally used as descriptors of geomorphologically adjusted channel forms. As design tools, hydraulic geometry relationships may be useful for preliminary or trial selection of the stable channel width.

A hydraulic geometry relation for width can be developed for a specific river, watershed, or for streams with similar physiographic characteristics. Data scatter is expected about the developed curve even in the same river reach. An example of a hydraulic geometry relationship between bankfull discharge and bankfull water surface width developed for a mountainous watershed can be found in Emmett (1975). He collected data at 39 gaging stations in the Salmon River Drainage Basin, ID. The relationship between bankfull discharge and bankfull width is shown in Figure 34. Emmett's mean regression line had a regression coefficient (r^2) of 0.92. Nevertheless a wide range of bankfull widths were found for any specific bankfull discharge. This range does not necessarily indicate instability or different physiographic conditions (Emmett gave no indication in his report that any of his sites were unstable) but rather the wide range of possible stable widths for a given channel-forming discharge. The data scatter in Figure 34 also demonstrates the importance of using confidence bands with hydraulic geometry relationships in geomorphologic stability assessment.

It follows that the more dissimilar the stream and watershed characteristics are, the greater the expected data scatter is. It is important to recognize that this scatter represents a valid range of stable channel configurations due to variables such as geology, vegetation, land use, sediment load and gradation, and runoff characteristics. The composition of the bank is very important in the determination of a stable channel width. It has been shown that the presence and percentage of cohesive sediment in the bank and the amount of vegetation on the bank may significantly affect the stable alluvial channel width.

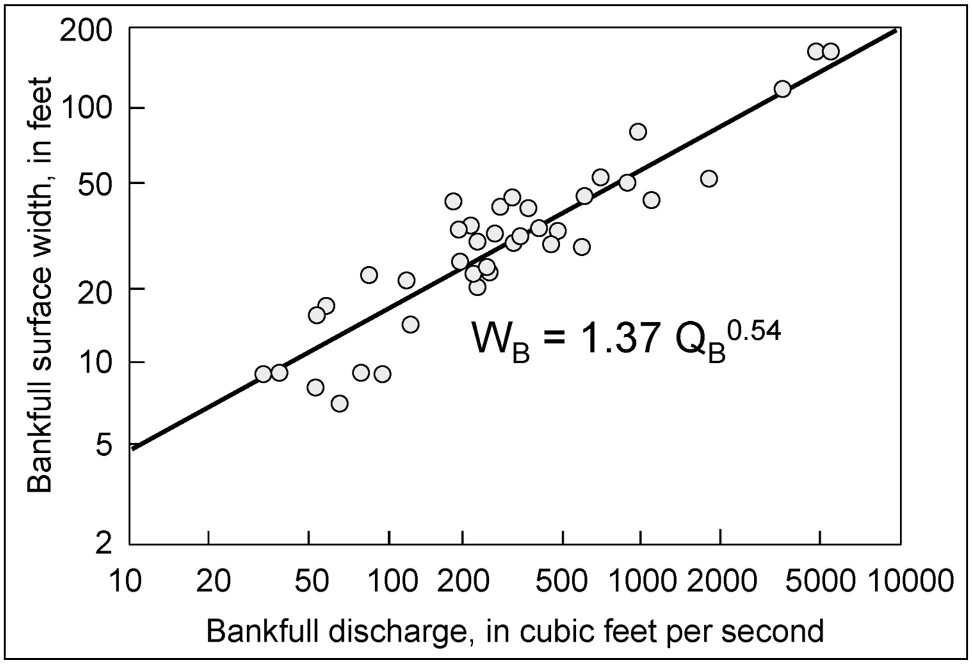

Figure 34. Hydraulic geometry relationship for width for the Upper Salmon River Basin, ID (Emmett 1975)

A regional slope-drainage area hydraulic geometry relationship can be developed for physiographically similar watersheds. An empirical regional stability relationship that defines the stable channel slope (equilibrium slope) as a function of drainage area (a surrogate for discharge) has been developed for several watersheds in north Mississippi (Figure 35). Channel slopes were measured in the field at several locations where stable reaches could be identified. Drainage area was determined from topographic maps. The equilibrium slope was used to set the slope between grade control structures in unstable reaches. The slope-drainage area curve can be a valuable relationship for initial understanding of stream morphology in an unstable watershed. However, the relationship is empirical and extrapolation to other watersheds, or the same watershed during a different time period, is risky. Constant field verification is necessary for continued value.

When a hydraulic geometry relationship is to be used for a channel restoration design it is best to use one developed from stable alluvial reaches of the project stream. It is required that the stable reaches used to develop the relationship have similar physiographic conditions to each other and the project reach. If there are no stable reaches or if the range of discharges is insufficient, other streams or tributaries in the same watershed may be used to develop the hydraulic geometry relationship. The third choice is to use regional relationships developed for other watersheds in the same physiographic region. In all cases it must be remembered that data used to develop hydraulic geometry relationships should come from stable reaches and that the watersheds and channel boundary conditions should be similar in the project channel.

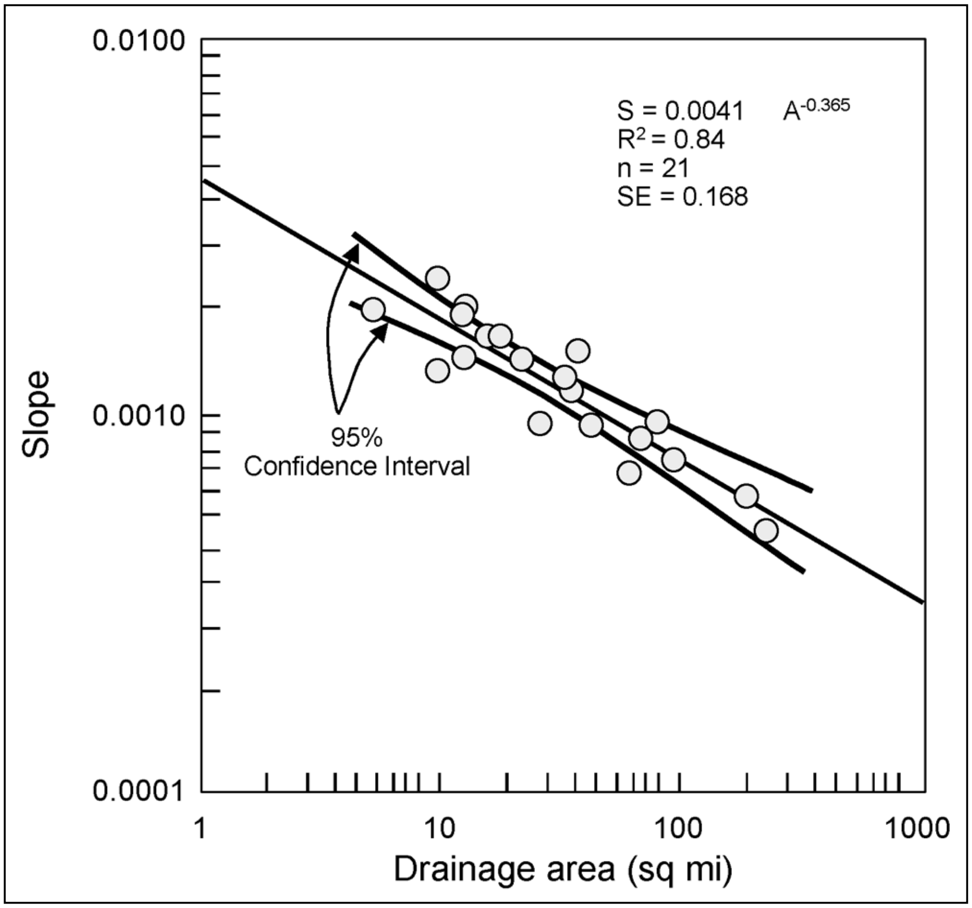

Figure 35. Equilibrium channel slope versus drainage area for Hickahala Creek, Batupan Bogue, and Hotopha Creek, MS

Lacking data to develop more reliable hydraulic geometry relationships, generalized width predictors for various stream types with different bank characteristics have been developed and are presented in Figures 36 through 43. These predictors include confidence limits and may be used for general guidance when stream or watershed specific data cannot be obtained.

a. Hydraulic geometry for meandering sand bed rivers

Hydraulic geometry width predictors (Figure 36) were developed from data collected from 58 meandering sand bed rivers in the United States (Soar and Thorne 2001). Sufficient data were collected to determine both bankfull discharge and effective discharge. Data were collected from stable reaches, so bankfull discharge should be the most reliable approximator for the channel-forming discharge. In many of these meandering sand bed rivers, the effective discharge was significantly less that the bankfull discharge. For design purposes, the bankfull discharge was used to define the width predictor. The data were divided into two sets: type T1 where there was less that 50 percent tree cover on the banks (Figure 37) and type T2 where there was greater than 50 percent tree cover on the banks (Figure 38). All sites were treelined to some degree, therefore the predictors should not be used for grasslined or thinly vegetated banks. The percentage of silt and clay in the banks was not found to be significant in affecting width for these rivers, possibly because the root-binding properties of the trees were more significant in stabilizing the bank than cohesive forces.

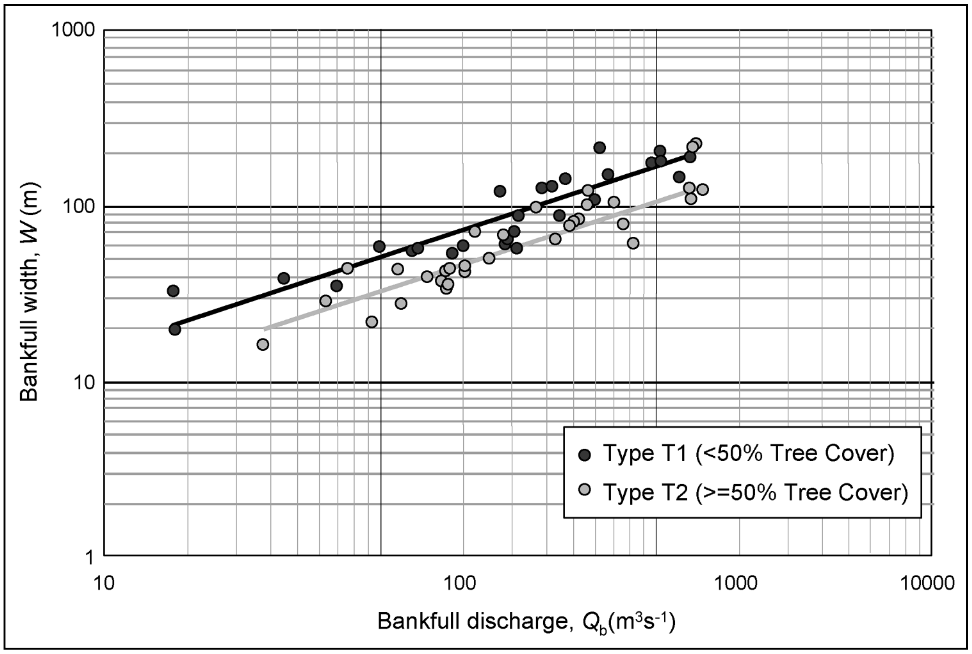

Figure 36. Best-fit hydraulic geometry relationships for width for U.S. sand bed rivers with banks typed according to density of tree cover

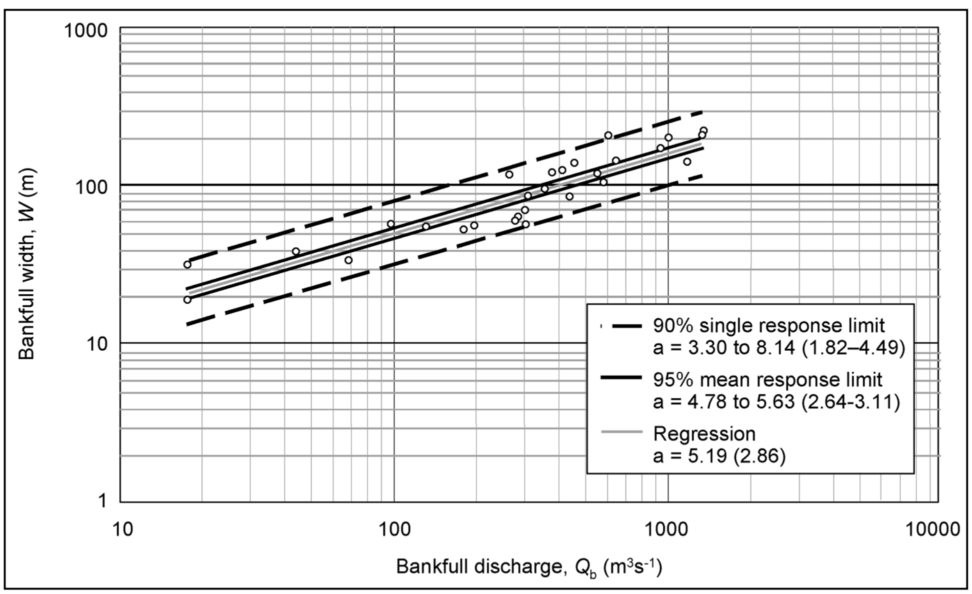

Figure 37. Confidence intervals applied to the hydraulic geometry equation for width based on 32 sand bed streams with less than 50 percent tree cover on the banks (T1). S.I. units, m and m^3/sec (English units, ft and ft^3/sec)

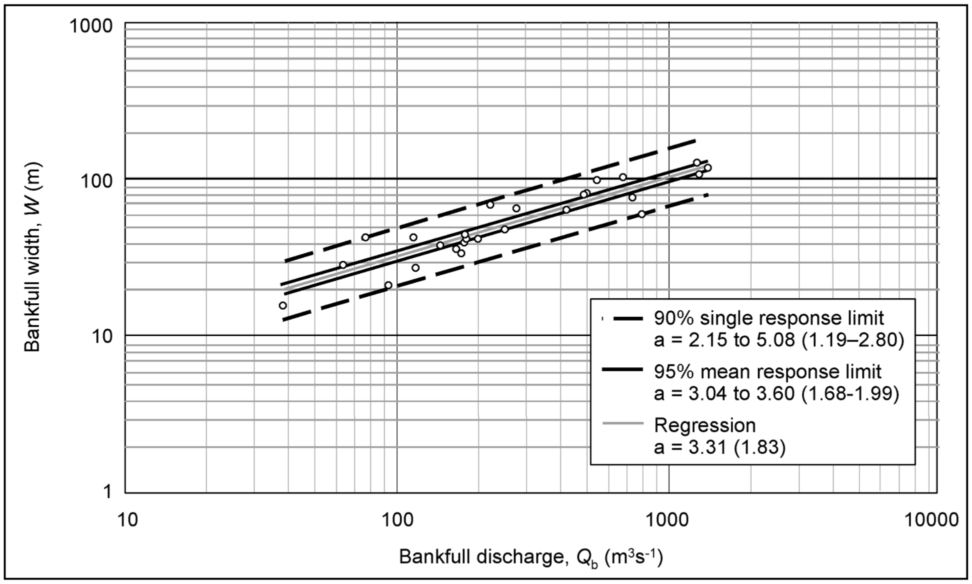

Figure 38. Confidence intervals applied to the width hydraulic geometry equation based on 26 sand bed rivers with at least 50 percent tree cover on the banks (T2). S.I. units, m and m^3/sec (English units, ft and ft^3/sec)

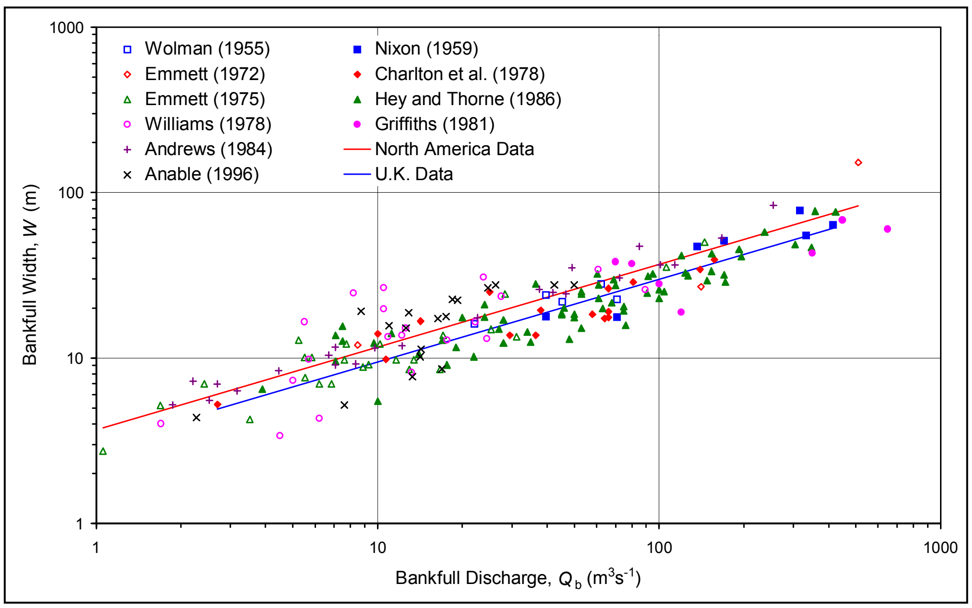

Figure 39. Downstream width hydraulic geometry for North American gravel bed rivers, $W = 3.68\ Q_b^{0.5}$, and U.K. gravel bed rivers, $W = 2.99\ Q_b^{0.5}$

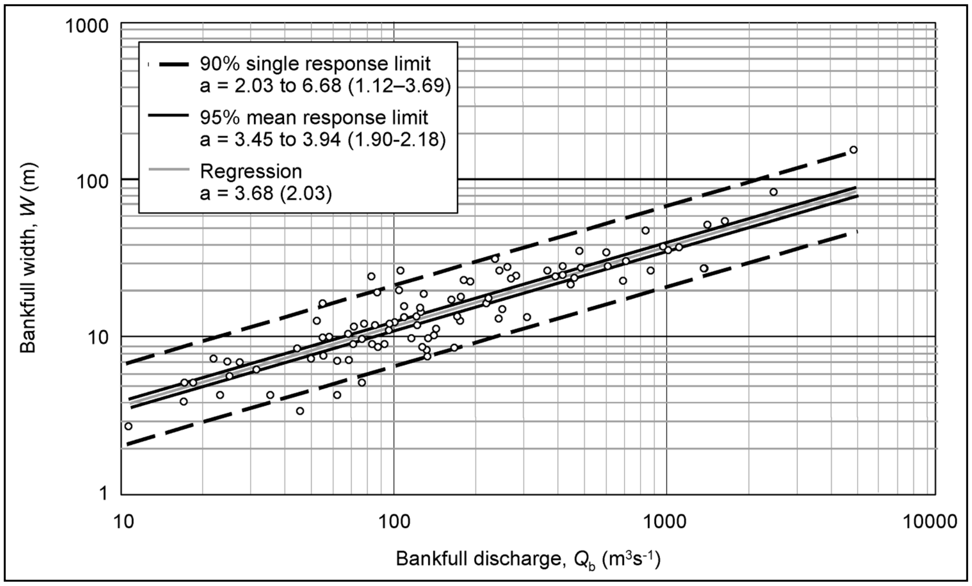

Figure 40. Downstream width hydraulic geometry for North American gravel bed rivers, $W = a\ Q_b^{0.5}$ with confidence bands. Based on 94 sites in North America. S.I. units, m and m^3/sec (English units, ft and ft^3/sec)

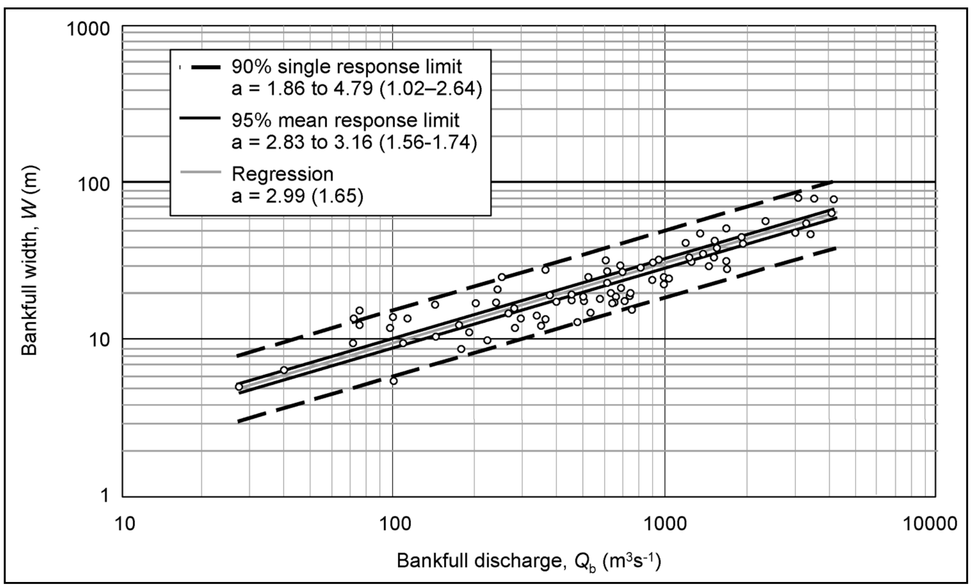

Figure 41. Downstream width hydraulic geometry for United Kingdom gravel bed rivers, $W = a\ Q_b^{0.5}$ with confidence bands. Based on 86 sites in the United Kingdom. S.I. units, m and m^3/sec (English units, ft and ft^3/sec)

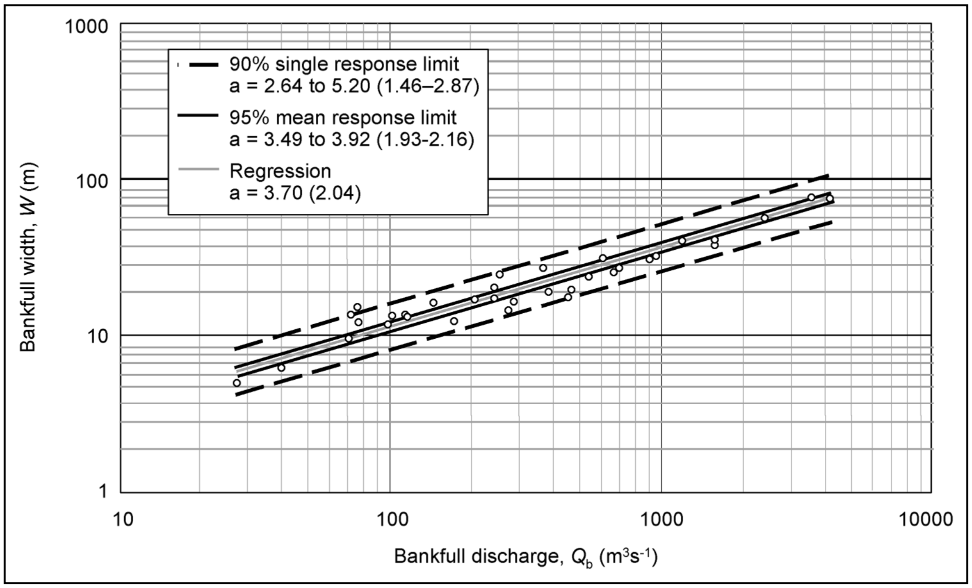

Figure 42. Downstream width hydraulic geometry for United Kingdom gravel bed rivers, $W = a\ Q_b^{0.5}$ with confidence bands. Based on 36 sites in the United Kingdom with erodible banks. S.I. units, m and m^3/sec (English units, ft and ft^3/sec)

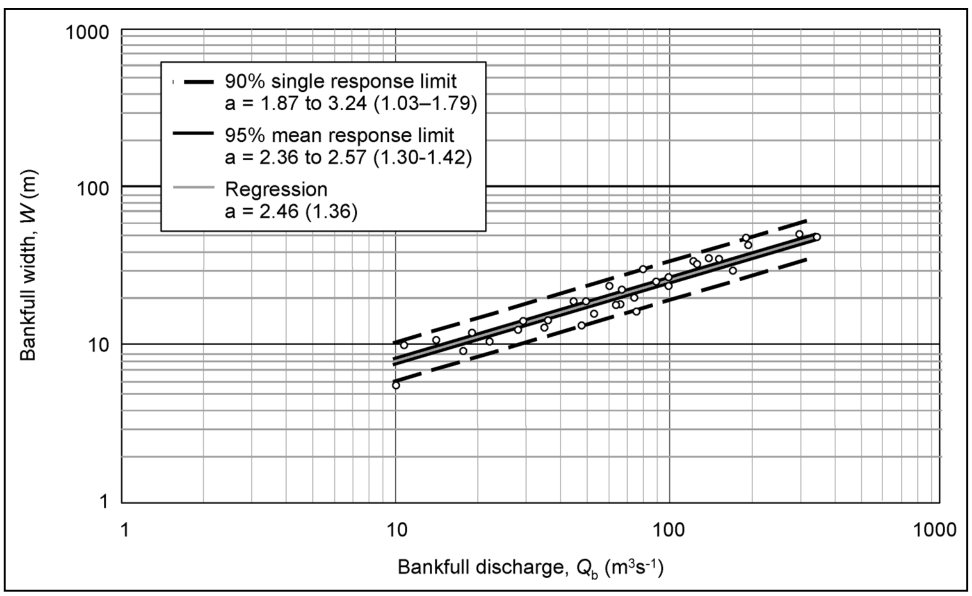

Figure 43. Downstream width hydraulic geometry for United Kingdom gravel bed rivers, $W = a\ Q_b^{0.5}$ with confidence bands. Based on 43 sites in the United Kingdom with resistant banks. S.I. units, m and m^3/sec (English units, ft and ft^3/sec)

The hydraulic geometry width predictor is expressed by the general equation:

$$W = a\ Q^b$$

Where W is the channel top width, Q is the channel-forming discharge, and values for the coefficient a and the exponent b are given in Table 3. The hydraulic geometry width predictors each include two sets of confidence bands. The 95 percent mean response limit provides the band in which one can be 95 percent confident that the mean value of the width will occur. This is the confidence interval for the regression line. This provides the range of average values of width that can be expected for a given discharge. The 90 percent single response limit provides the envelope curves that contain 90 percent of the data points. This is the confidence interval for an individual predicted value. This provides the engineer with the range of possible widths that have been observed to correspond to a given discharge. The confidence interval on an individual predicted value is wider than the confidence interval of the regression line since it includes both the variance of the regression line plus the squared standard deviation of the data set. While the equations given in Table 3 may be used for preliminary design purposes, they are subject to several limitations. In the absence of stage-discharge relationships at each site, the equations are based on flow resistance considerations. As cross-sectional geometry was used to calculate discharge, discharge is not truly independent of width in this analysis. Furthermore, only one cross section was measured at each site in order to maximize the size of the data set and identification of the bankfull reference level, although based on field experience and geomorphic criteria, is always subject to a degree of uncertainty. These factors contribute to the observed variability in the width relationships. Finally, small rivers are not well represented in the data set and should not be applied when discharge is less than 17 m^3s^{-1} in type T1 channels and less than 38 m^3s^{-1} in type T2 channels.

Table 3
Hydraulic Geometry Width Predictors For Sand Bed Channels

Data Source	Sample size	*a*	90% single response limit for *a*	95% mean response limit for *a*	*b*	r^2
All sand-bed streams	58	4.24 (2.34)	2.34-7.68 (1.29-4.24)	3.90-4.60 (2.15-2.54)	0.5	0.76
Type T1: <50% tree cover	32	5.19 (2.86)	3.30-8.14 (1.82-4.49)	4.78-5.63 (2.64-3.11)	0.5	0.87
Type T2: >%50% tree cover	26	3.31 (1.83)	2.15-5.08 (1.19-2.80)	3.04-3.60 (1.68-1.99)	0.5	0.85

Note: r^2 refers to linear regression equations (not given) where *b* was variable. Exponent *b* was found not to be statistically different from 0.5 which was chosen for convenience. S.I. units m and m^3/sec (English units ft and ft^3/sec) $W = a\,Q^b$

b. Hydraulic geometry for gravel bed rivers.

A review of the published gravel bed stream data and hydraulic geometry width predictors for North American and British streams (Soar and Thorne 2001) revealed that North American gravel bed rivers are generally wider than those found in the U.K., assuming discharge and other conditions are equal. North American data used to develop the hydraulic geometry relationship included data from Brandywine Creek in Pennsylvania (Wolman 1955), Alaskan streams

(Emmett 1972), Upper Salmon River in Idaho (Emmett 1975), Colorado, New Mexico, Oregon, Pennsylvania, Tennessee, Utah, West Virginia, and Wyoming (Williams 1978), Alberta, Canada (Annable 1996), and the Rocky Mountain region of Colorado (Andrews 1984). United Kingdom data included data from Nixon (1959), Charlton, Brown, and Benson (1978), and Hey and Thorne (1986). The hydraulic geometry relationships are shown in Figure 39. The difference in these regression curves cannot satisfactorily be explained using the site descriptions given in original publications. A possible explanation is that the U.K. sites have on the average more resistant banks than the North American sites. Another plausible explanation is that width in mobile-gravel bed streams varies with flow variability and the North American sites on the average may be more flashy. Still another possibility is that the North American sites may be more active, that is have a higher concentration of sediment transport. Further research is required to validate these hypotheses.

The hydraulic geometry width predictors for United Kingdom and North American gravel bed streams are presented with confidence bands in Figures 40 and 41, respectively. Exponents and coefficients for the hydraulic geometry equation are given in Table 4. The gravel bed river data comprise a wide range of bank material types (e.g., cohesive, sand, gravel, and composite banks of various strata). However, different width-discharge relationships based on different types of bank material could not be derived for the North American river data from the limited information available. There were sufficient data available from the UK gravel bed rivers to develop distinct width predictors based on erodible banks (low density of trees) and resistant banks (high density of trees). These are presented in Figures 42 and 43. These hydraulic geometry relations may be used for preliminary design purposes, recognizing that considerable variability may occur for areas different from the streams used in the development of the equations.

Table 4
Hydraulic Geometry Width Predictors for Gravel Bed Channels

Data Source	Sample size	*a*	90% single response limit for *a*	95% mean response limit for *a*	*b*	r^2
All North American gravel bed streams	94	3.68 (2.03)	2.03-6.68 (1.12-3.69)	3.45-3.94 (1.90-2.18)	0.5	0.80
All United Kingdom gravel bed streams	86	2.99 (1.65)	1.86-4.79 (1.02-2.64)	2.83-3.16 (1.56-1.74)	0.5	0.80
<5% tree or shrub cover, or grass-lined banks (UK streams)	36	3.70 (2.04)	2.64-5.20 (1.46-2.87)	3.49-3.92 (1.93-2.16)	0.5	0.92
≥5% tree or shrub cover (UK streams)	43	2.46 (1.36)	1.87-3.24 (1.03-1.79)	2.36-2.57 (1.30-1.42)	0.5	0.92

Note: r^2 refers to linear regression equations (not given) where *b* was variable. Exponent *b* was found not to be statistically different from 0.5, which was chosen for convenience.
S.I. units m and m^3/sec (English units ft and ft^3/sec) $W = a\,Q^b$

Extremal hypotheses. If a reliable hydraulic geometry relationship cannot be determined from field data or when sediment transport is significant, analytical methods may be employed to obtain a range of feasible solutions. Analytical methods employ an extremal hypothesis as a third equation. One extremal hypothesis assumes that a channel will adjust its geometry so that the time rate of energy expenditure is minimized (Chang 1980; Copeland 1994). Another assumes that sediment transport is maximized within the constraints on the system (White, Bettess, and Paris 1982; Millar and Quick 1993). These are equivalent assumptions. Computer programs or look-up charts are required to solve the resistance, sediment transport, and extremal equations simultaneously. The SAM hydraulic design package contains a program to solve these equations using either the Brownlie (1981) resistance and sediment transport equations for sandbed streams, or the Limerinos (1970) resistance equation and the Meyer-Peter and Muller (1948) sediment transport equation for gravel-bed streams.

The advantage of using an extremal hypothesis is that a unique solution can be obtained for the dependent variables of width, depth, and slope. However, extensive field experience demonstrates that channels can be stable with widths, depths, and slopes different from those found at the extremal condition. Also the sensitivity of energy minima or sediment transport maxima to changes in driving variables may be low, so that the channel dimensions corresponding to the extremal value are poorly defined.

Constrained dependent variables. In many cases, project constraints limit the theoretical variability in channel geometry. For example, the channel slope cannot be greater than the valley slope for a long reach. The channel width may be limited by available rights-of-way, or flood risks, and damages may limit allowable depth. For these and many other reasons, the selection of one of the dependent design variables may be based on established project constraints.

Calculation of the remaining unknown design variables. Once one of the dependent design variables is determined, the other two should be calculated using one of several resistance and sediment transport equations available in the literature. Appropriate equations can be chosen from those described in EM 1110-2-1601, EM 1110-2-1418, or the SAM Users Manual (Thomas et al. 2000).

In coarse-bed streams where bed-material sediment transport is small, or in streams with bedrock outcrops or with cohesive beds, threshold design methods may be used to calculate depth and slope. However, in sand bed streams, bed-material sediment transport is typically significant and an analytical procedure that considers both sediment transport and bed form roughness is required.

The stable-channel analytical method in the Corps hydraulic design package SAM may be used to determine the unknown dependent design variables. This method is based on a typical trapezoidal cross section and assumes steady, uniform flow. The method is especially applicable to small streams because it accounts for sediment transport, bed form and grain roughness, and bank roughness. This procedure assumes a fully mobile bed. Details are available in the SAM users manual.

The stable channel analytical method in SAM produces a family of solutions for slope and depth for specified widths for a selected discharge (Figure 44). These curves represent combinations of width, depth, and slope that satisfy the sediment transport and roughness equations. The wide range of possible solutions can be narrowed by the assigned project constraints. For example, a maximum width constraint might be imposed by the available rights-of-way, a maximum depth constraint might be imposed by flood-control considerations, and/or a maximum slope constraint would be imposed by the valley slope. Lacking project constraints a hydraulic geometry relationship with confidence limits for width could be used to select a range of stable slopes and depths, or the extremal assumption can be applied and the unique solution occurs at the minimum slope on the stable channel design curve.

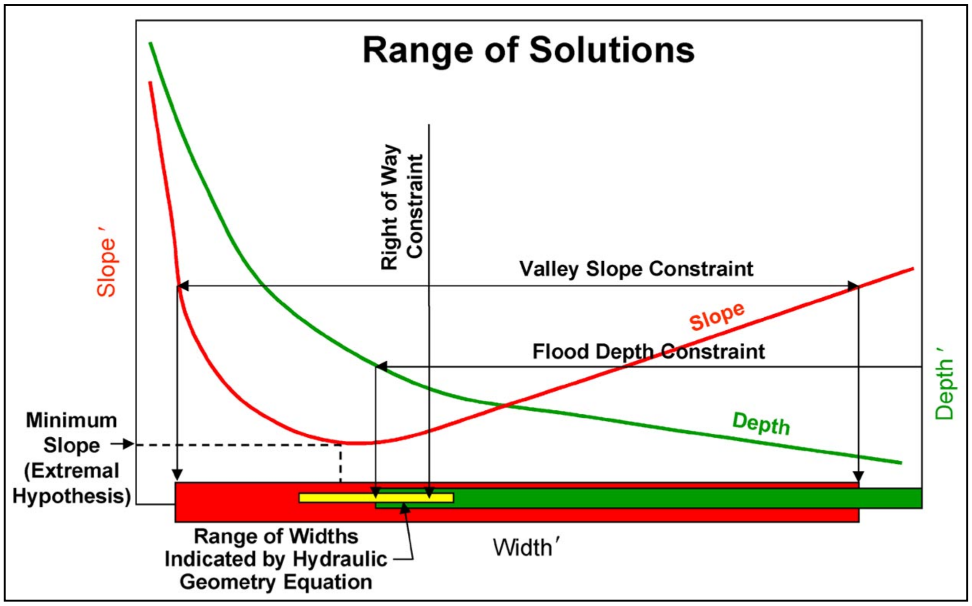

Figure 44. Stability curve from stable channel analytical method

Planform

This step involves determining a meander wavelength, an appropriate channel length for one meander wavelength, and then laying out a planform. Existing methods often rely on the user locating a reference or control reach on either the study stream or another suitable stream from which to develop a template for the meander planform. This may often be problematic due to the nonavailability of a reference reach, subtle but important fluvial, sedimentary or morphological differences between it and the study reach, or restrictions on the rights-of-way, which preclude the introduction of meanders with the same amplitudes observed in the reference reach. Alternatively, meander wavelength can be determined using hydraulic geometry techniques. The most reliable hydraulic geometry relationship is wavelength vs. width. As with the determination of channel width, preference is given to wavelength predictors from stable reaches of the existing stream either in the project reach or in reference reaches. Lacking data from the existing stream, general guidance is available from several literature sources. A composite relationship has been developed by Soar and Thorne (2001) combining nine data sets and 438 sites. Their mean linear regression predictor for wavelength is

$$\lambda = 10.23\,W$$

where: λ is meander wavelength and W is channel width. Definitions of planform descriptive variables are shown in Figure 45. Confidence bands about this equation are shown in Figure 46. The r^2 for the wavelength equation was 0.88 for a linear regression equation with a variable exponent on W. This exponent was found not to be significantly different from 1.0 so the exponent was fixed at 1.0 for convenience. Only sites with sinuosities of at least 1.2 and bankfull widths between 1 m and 1,000 m were used in development of this regression equation. Within these constraints, meander wavelengths range between 10.4 m and 19,368 m and sinuosities range between 1.2 and 5.3. The equation corrected for bias is:

$$\lambda \quad = \quad 11.85\,W$$

An unbiased hydrologic equation for meander wavelength within 95 percent confidence limits on the mean response suitable for engineering design is:

$$\lambda \quad = \quad (11.26 \;\; to \;\; 12.47)\,W$$

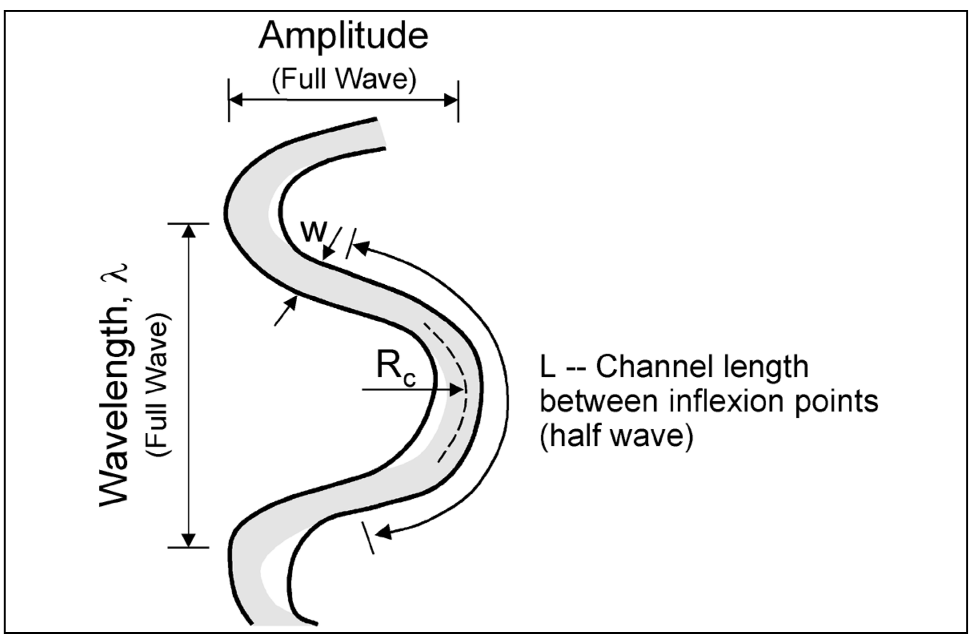

Figure 45. Meander parameters

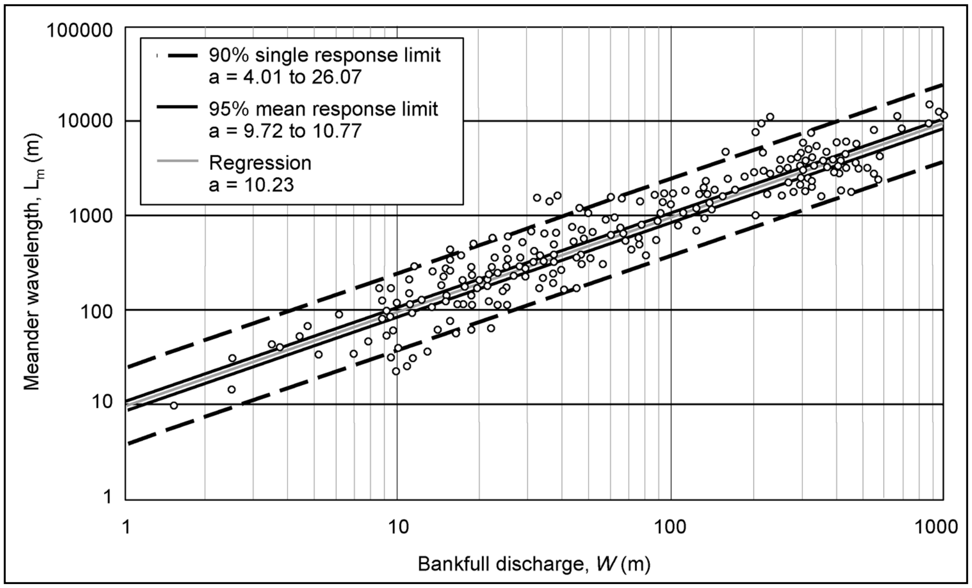

Figure 46. Hydraulic geometry relationship for meander wavelength with confidence intervals, λ = 10.23 W, based on a composite data set of 438 sites

According to Hey (1976) and Thorne (1997), twice the distance between successive riffles (or pools) in a straight channel equals $4\pi W$ (12.57 *W*). This is based on the assumption that the average size of the largest macro-turbulent eddies (or helical flow cell) is half the channel width. The preceding equation shows that the upper range of stable meander wavelengths is numerically very close to this value and similar to the coefficient of 12.34 given by Richards (1982). This corroborates the assertion by Leopold and Wolman (1957, 1960) that the matching of waveforms in bed topography and planform is related to the mechanics of the flow, and in particular to the turbulent flow structures responsible for shaping the forms and features of meandering channels.

The following data sources were used in the development of these equations: Leopold and Wolman (1957) data from U.S. rivers (21 sites); Leopold and Wolman (1960) data compiled from various sources and including rivers in France (1 site), U.S. (34 sites) and one model river (total of 36 sites); Carlston (1965) data from U.S. rivers (29 sites); Schumm (1968) data from midwestern U.S. rivers (25 sites); Chitale (1970) data from large alluvial rivers in Africa (1 site), Canada (1 site), India (16 sites), Pakistan (2 sites) and U.S. (1 site) (total of 21 sites); Williams (1986) data compiled from various sources and including rivers in Australia (2 sites), Canada (7 sites), Sweden (17 sites), Russia (1 site), U.S. (16 sites) and one model river (total of 44 sites); Thorne and Abt (1993) data from various sources including measurements from the Red River 1966 (35 sites) and 1981 (39 sites) hydrographic surveys between Index, Arkansas, and Shreveport, LA, and rivers in India (12 sites), The Netherlands (1 site), U.K. (48 sites) and U.S. (18 sites) (total of 153 sites); Annable (1996) data from streams in Alberta, Canada (30 sites); and Cherry, Wilcock, and Wolman (1996) data from U.S. rivers, predominantly sand bed (79 sites).

Other hydraulic geometry relationships for meander wavelength from the literature are given in Table 5.

Table 5
Hydraulic Geometry Relationships for Meander Wavelength

Author	Equation	Units
Leopold and Wolman (1960)	$\lambda = 10.9\ W^{1.01}$	feet
Inglis (1941)	$\lambda = 6.06\ W^{0.99}$	feet
Yalin (1992)	$\lambda = 6\ W$	length
Dury (1965)	$\lambda = 30\ Q_{bf}^{0.5}$	feet, cfs
Carlston (1965)	$\lambda = 8.2\ Q_{bf}^{0.62}$	feet, cfs
Carlston (1965)	$\lambda = 106.1\ Q_{ma}^{0.46}$	cfs
Schumm (1967)	$\lambda = 1890\ Q_{ma}^{0.34}\ M^{-0.74}$	feet, cfs

Notes: λ = meander wavelength
W = width
Q_{bf} = bankfull discharge
Q_{ma} = mean annual discharge
M = silt-clay factor

The channel meander length is simply the meander wavelength times the valley slope divided by the channel slope.

$$\text{channel meander length} = \frac{\text{wavelength} \times \text{valley slope}}{\text{channel slope}}$$

Once meander wavelength is determined, one way to lay out the planform is to cut a string to the appropriate channel length and lay it out on a map. Another, more analytical approach, is to assume a sine-generated curve for the planform shape as suggested by Langbein and Leopold (1966) and calculate x-y coordinates for the planform. Their theory of minimum variance is based on the hypothesis that the river will seek the most probable path (the path that provides the minimum variance of bed shear stress and friction) between two fixed points. The sine-generated curve is defined in Figure 47 and by the following equation:

$$\Phi = \omega \cos \frac{2\pi s}{M}$$

where:

Φ = angle of meander path with the mean longitudinal axis

ω = maximum angle a path makes with the mean longitudinal axis in radians

s = the curvilinear coordinate along the meander path

M = the meander arc length

The shape parameter, ω, is a function of the channel sinuosity, P, which can approximated by the following equation (Langbein and Leopold 1966):

$$\omega = 2.2 \sqrt{\frac{P-1}{P}}$$

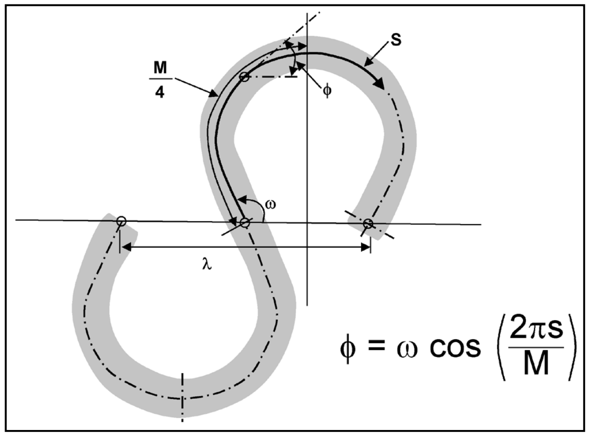

Figure 47. Definition of sine-generated curve

Figure 48 shows how the shape parameter of a sine-generated curve defines the shape of the stream.

Calculation of the points on a sine-generated curve is a rather tedious numeric integration for Φ. However, it can be accomplished using a computer program such as the one in the SAM hydraulic design package. The sine-generated curve produces a very uniform meander pattern. A combination of the string layout method and the analytical approach would produce a more natural looking planform.

The radius of planform curvature is not constant in the sine-generated curve but ranges from a maximum value at the inflexion point to a minimum curvature around the bend apex. The average radius of curvature is centered at the bend apex for a distance of approximately one sixth of the channel meander length.

Most reaches of stable meandering rivers have radius of curvature-to-width ratios between 1.5 and 4.5. Of the 438 sites used to derive the wavelength-width relationship in Figure 44, radius of curvature is recorded for 263 of the sites. This subset was used to develop a cumulative distribution curve of radius of curvature-to width ratios (Figure 49). This figure shows that 33.5 percent, 52.9 percent, and 71.2 percent of the sites have radius of curvature-to-width ratios between 2 and 3, 2 and 4, and 1.5 and 4.5 respectively. The final planform layout should have ratios within the normal range.

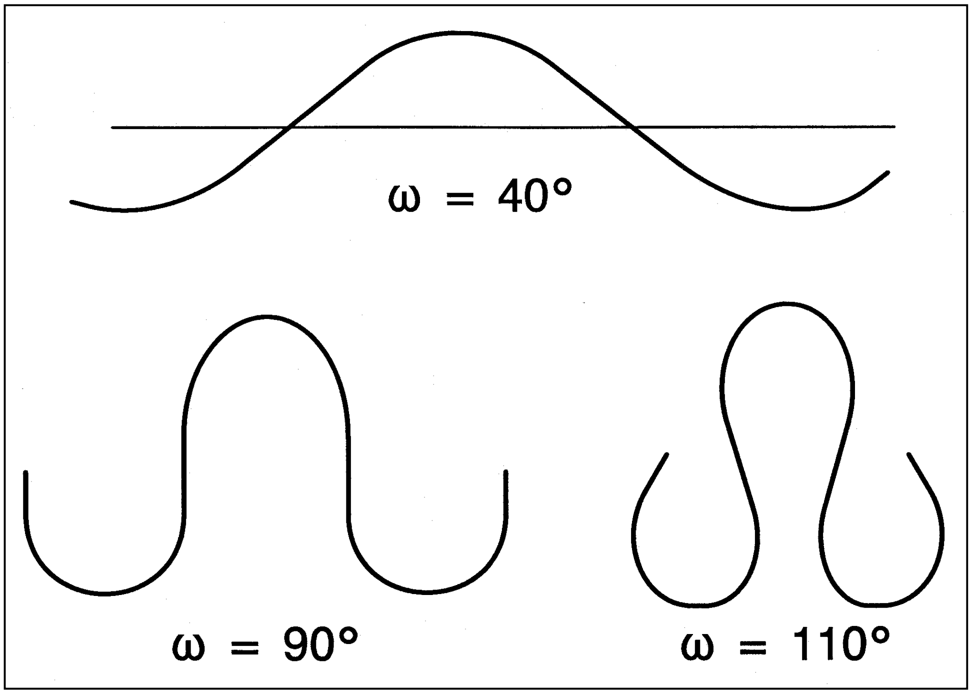

Figure 48. Effect of the shape factor on channel sinuosity with the sine-generated curve (Langbien and Leopold 1966)

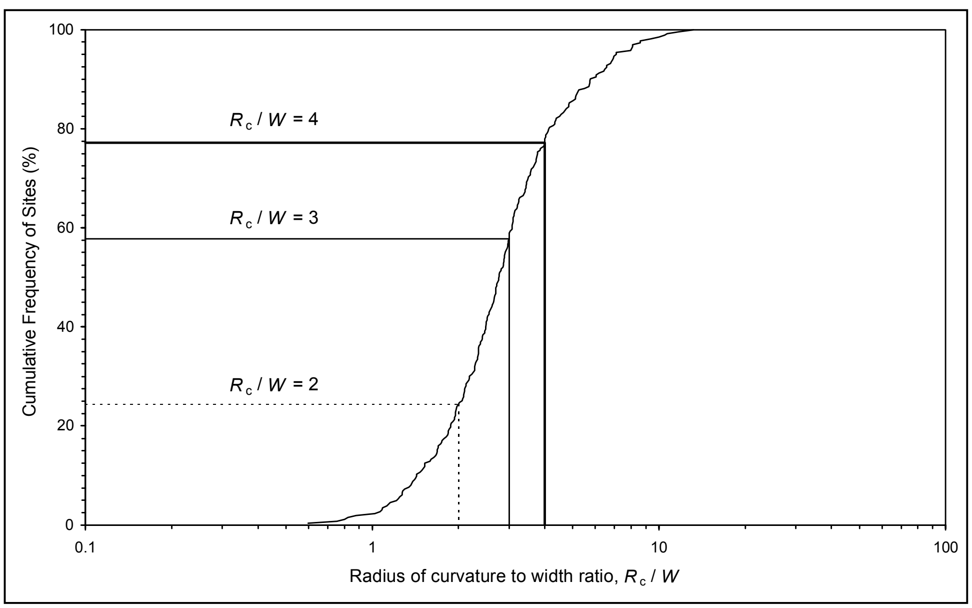

Figure 49. Cumulative distribution of radius of curvature-to-width ratio derived from a composite data set of 263 sites

If the calculated meander length is too great, or if the required meander belt width is unavailable, grade control structures may be required to reduce the channel slope and stabilize the bed elevations.

In streams that are essentially straight (sinuosity less than 1.2) riffle and pool spacing may be set as a function of channel width. The empirical guide of 5 to 7 channel widths applies here (Knighton 1984). Two times this riffle spacing gives the total channel length through one meander pattern.

Natural variability around meander bendways

Thorne (1988) and Soar and Thorne (2001) compiled empirical data sets of cross section and planform dimensions from meander bends in the Red River between Index, AR, and Shreveport, LA. The Red River in this reach is typical of large meandering rivers, having a wide variety of both bend geometries and bank materials. These studies provided a useful baseline database for examining the variability of width around meander bends. Of course, while the Red River in the study reach is representative of meandering rivers in general, if applied elsewhere these equations should be used with caution. In the data set, each bend was classified as one of three types based on the Brice (1975) classification system: equiwidth meanders - denoted as Type-e (T_e) meanders, meanders with point bars - denoted as Type-b (T_b) meanders, and meanders with point bars and chute channels - denoted as Type-c (T_c) meanders.

a. ***Equiwidth meandering.*** Equiwidth indicates that there is only minor variability in channel width around meander bends. These channels are generally characterized by: low width/depth ratios; erosion resistant banks; fine-grain bed material (sand or silt); low bed material load; low velocities; and low stream power. Channel migration rates are relatively low because the banks are naturally stable.

b. ***Meandering with point bars.*** Meandering with point bars refers to channels that are significantly wider at bendways than crossings, with well-developed point bars but few chute channels. These channels are generally characterized by: intermediate width/depth ratios; moderately erosion resistant banks; medium grained bed material (sand or gravel); medium bed material load; medium velocities; and medium stream power. Channel migration rates are likely to be moderate unless banks are stabilized.

c. ***Meandering with point bars and chute channels.*** Meandering with point bars and chute channels refers to channels that are very much wider at bendways than crossings, with well-developed point bars and frequent chute channels. These channels are generally characterized by: moderate to high width/depth ratios; highly erodible banks; medium to coarse grained bed material (sand, gravel, and/or cobbles); heavy bed material load; moderate to high velocities; and moderate to high stream power. Channel migration rates are likely to be moderate to high unless banks are stabilized.

Ranges of physical characteristics pertaining to each of the meander bend types are given in Table 6. Figure 50 provides a definition sketch for channel cross-section geometries and dimensions through a meander.

Table 6
Ranges of Physical Characteristics Found in Different Meander Bend Types Identified from the 1981 Red River Hydrographic Survey Between Index, AR, and Shreveport, LA

	n	S (10^6)	P	W_i / D_m	D_{max} / D_i	R_c / W_i
Type-e	20 (8)	65 to 268 (133 to 268)	1.0 to 2.1 (1.2 to 2.1)	34.2 to 74.1 (38.3 to 74.1)	1.6 to 2.4 (1.7 to 2.4)	0.9 to 9.3 (0.9 to 5.2)
Type-b	34 (19)	76 to 294 (105 to 294)	1.0 to 2.0 (1.1 to 2.0)	36.8 to 121.0 (36.8 to 102.4)	1.5 to 2.6 (1.7 to 2.6)	1.5 to 9.1 (1.5 to 6.1)
Type-c	13 (10)	91 to 201 (91 to 201)	1.1 to 2.3 (1.2 to 2.3)	33.5 to 88.2 (33.5 to 88.2)	1.6 to 2.4 (1.6 to 2.4)	2.2 to 6.8 (2.2 to 5.2)

Note: n = number of meander bends studied; S = water-surface slope; P = sinuosity; W_i / D_m = inflexion point width-to-mean depth ratio; D_{max} / D_i = maximum scour depth in pool-to-mean depth at inflexion point; R_c / W_i = radius of curvature-to-inflexion point width ratio. Values in parentheses refer to meander bends with sinuosity 1.2 or greater.

Two dimensionless parameters can be used to describe the width variability around meander bends based on the enhanced Red River data set. These are the ratio of bend apex width to inflexion point width, W_a/W_i, and the ratio of width at the location of maximum bend pool scour to inflexion point width, W_p/W_i. Theoretically, these parameters adjust according to the degree of curvature and the type of meander bend. To derive new morphological relationships, sinuosity, P, was preferred as the independent variable rather than the radius of curvature-to-width ratio, which would have resulted in width appearing on both sides of the regression equations.

Morphologic relationships for the width ratios as a function of meander type were developed for channels with sinuosities greater than 1.2. This is a commonly accepted threshold between straight channels with only slight sinuosity and meandering channels with moderate to high sinuosity. The bed apex width to inflexion point width ratio, W_a/W_i, was found to be independent of sinuosity. Data are plotted with confidence limits in Figure 51. Values for the ratios for each type of meander bend can be determined from Table 7 and the following equation, where p denotes the level of significance and corresponds to the 100(1-p)% confidence level.

$$\left(\frac{W_a}{W_i}\right)_p = a + u_p \qquad (2)$$

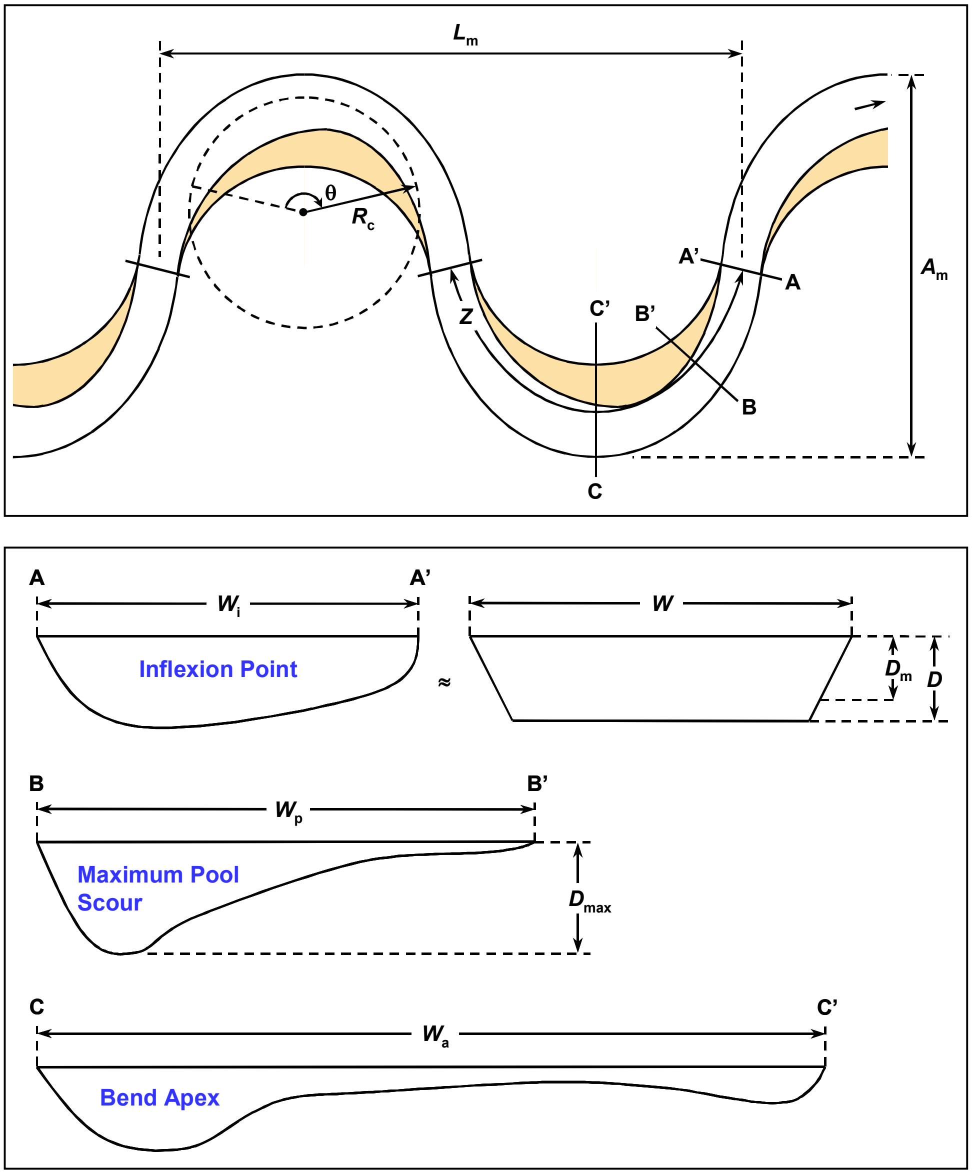

Figure 50. Meander cross-section dimensions for restoration design. Note: Point bars defined by shaded regions; L_m = meander wavelength, Z = meander arc length (riffle spacing); A_m = meander belt width, R_c = radius of curvature; Θ = meander arc angle; W = reach average bankfull width; D = depth of trapezoidal cross section; D_m = mean depth (cross-sectional area / W); D_{max} = maximum scour depth in bendway pool; W_i = width at meander inflexion point; W_p = width at maximum scour location; W_a = width at meander bend apex

Table 7
Constant Values Used to Estimate the Mean Ratio of Bend Apex Width to Inflexion Point Width, W_a/W_i, Within Confidence Bands for Different Types of Meander Bends and for Sites with Sinuosity of at Least 1.2 (Coefficients Pertaining to the 99, 95 and 90 percent Confidence Limits are Given)

	a	$u_{0.01}$	$u_{0.05}$	$u_{0.1}$
Type-e	1.05	0.08 (0.29)	0.05 (0.20)	0.04 (0.16)
Type-b	1.35	0.05 (0.27)	0.04 (0.20)	0.03 (0.16)
Type-c	1.79	0.09 (0.36)	0.06 (0.25)	0.05 (0.20)

Note: Values given refer to mean response confidence limits. Value in parentheses is used to calculate single response confidence limits.

Morphologic relationships for the width ratios as a function of meander type were developed for the ratio of pool width at the location of maximum scour to inflexion point width (W_p/W_i) for channels with sinuosities greater than 1.2. This ratio was also found to be independent of sinuosity. Data and confidence limits are plotted in Figure 52. Values for the ratios for each type of meandering river can be determined from the following equation and Table 8.

$$\left(\frac{W_\mathrm{p}}{W_\mathrm{i}}\right)_p = \mathrm{a} + \mathrm{u}_p$$

Table 8
Constant Values Used to Estimate the Mean Ratio of Pool Width (at Maximum Scour Location) to Inflexion Point Width, W_p/W_i, Within Confidence Bands for Different Types of Meander Bends and for Sites with Sinuosity of at least 1.2. Coefficients Pertaining to the 99, 95, and 90 Percent Confidence Limits are Given.

	a	$u_{0.01}$	$u_{0.05}$	$u_{0.1}$
Type-e	0.95	0.15 (0.56)	0.10 (0.38)	0.08 (0.30)
Type-b	1.15	0.12 (0.64)	0.09 (0.47)	0.07 (0.39)
Type-c	1.29	0.26 (1.07)	0.18 (0.74)	0.14 (0.60)

Note: Values given refer to mean response confidence limits. Value in parentheses is used to calculate single response confidence limits.

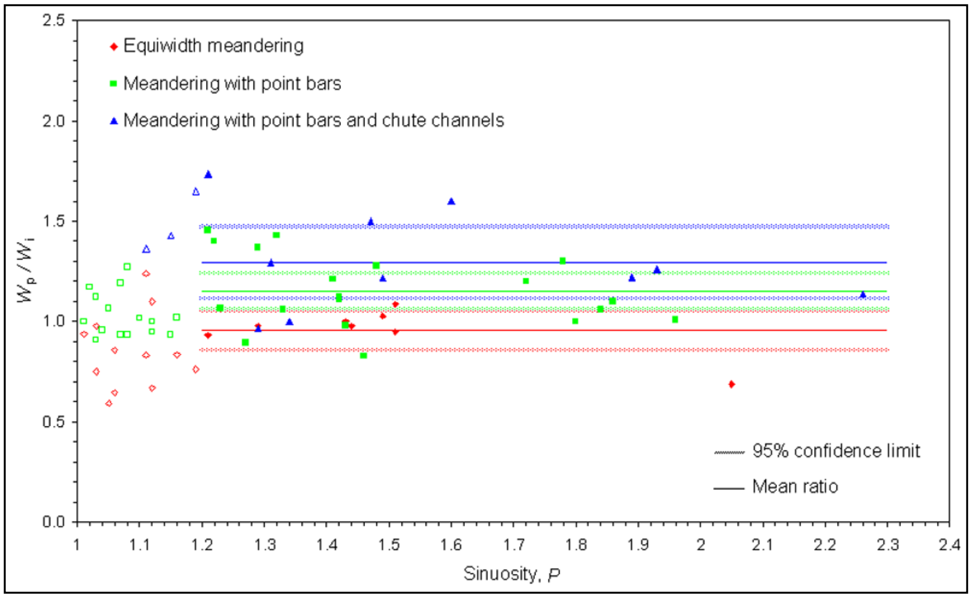

Figure 51. Ratio of bend apex width to inflexion point width, W_a/W_i as a function of meander bend type only, for sinuosities of at least 1.2. Confidence limits of a mean response are shown at the 95 percent level. Source data: 1981 Red River hydrographic survey. Note: Filled symbols = sinuosity of at least 1.2; empty symbols = sinuosity less than 1.2

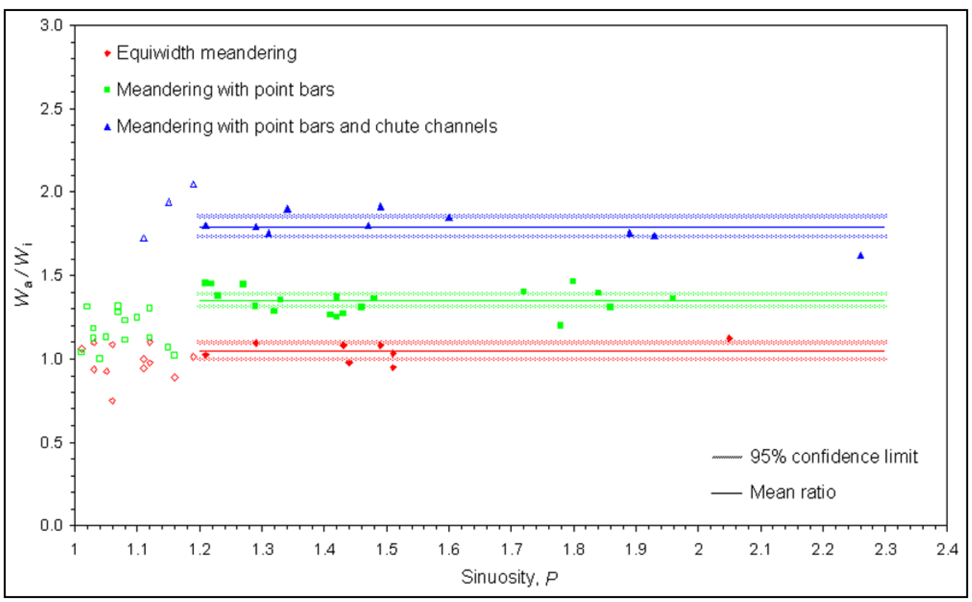

Figure 52. Ratio of pool width (at maximum scour location) to inflexion point width, *Wp/Wi* as a function of meander bend type only, for sinuosities of at least 1.2. Confidence limits of a mean response are shown at the 95 percent level. Source data: 1981 Red River hydrographic survey. Note: Filled symbols = sinuosity of at least 1.2; empty symbols = sinuosity less than 1.2

While the location of meander inflexion points and bend apices are geometrically defined, the location of pools, defined by the position of maximum bend scour, is not only controlled by the meander configuration but by the complex velocity distribution and large-scale coherent flow structures which pulse sediment along the channel to form alternate zones of scour and fill. In natural meanders, the deepest pool is usually located downstream from the bend apex and restoration design should mimic this natural attribute in constructed meanders. The pool location in a meander bend can be represented empirically by a pool-offset ratio, defined as the ratio of the channel distance between bend apex and maximum scour location to the channel distance between bend apex and downstream inflexion point, $Z_{a\text{-}p} / Z_{a\text{-}i}$. The pool–offset ratio was found to be independent of sinuosity. Neither was a distinct relationship found for the different meander types. The range and cumulative distribution function for the pool-offset ratio is shown in Figure 53. The mean value for the ratio was 0.36 and the range was –0.4 to 1.08.

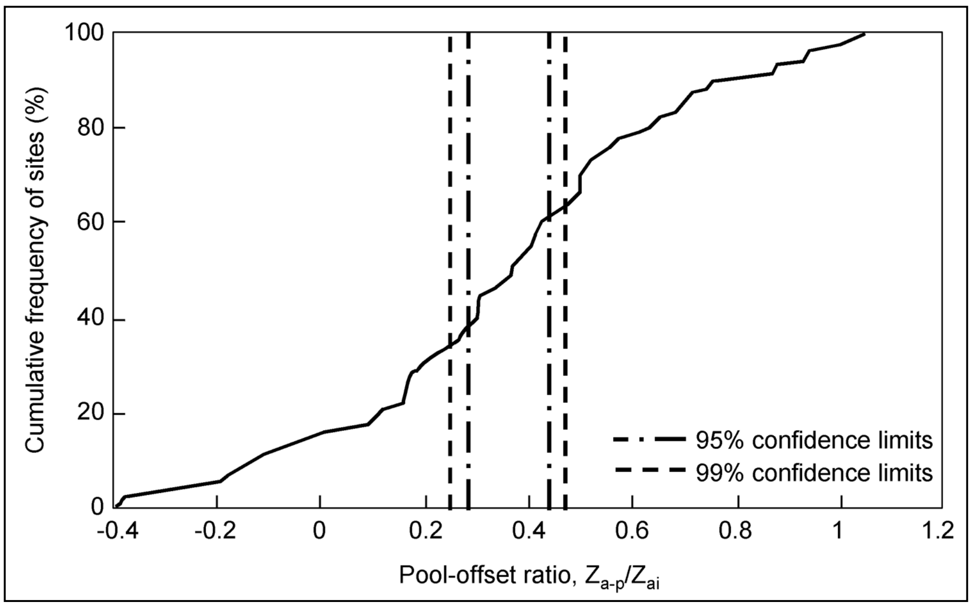

Figure 53. Cumulative distribution of the pool-offset ratio, $Z_{a\text{-}p}/Z_{a\text{-}i}$, for all types of meander bend studied. Confidence limits on the mean response are shown. Source data: 1981 Red River hydrographic survey

Data from a wide range of rivers (Thorne and Abt 1993; Maynord 1996) were used to develop morphological equations for the maximum scour depth in pools. The data were divided into two subsets using a width-to-depth threshold value of 60, which is an approximate modal value. The best-fit morphological relationships are given by Soar and Thorne (2001) as:

$$\frac{W_{\mathrm{i}}}{D_{\mathrm{m}}} < 60 \qquad \frac{D_{\max}}{D_{\mathrm{m}}} = 2 \cdot 14 - 0 \cdot 19 \ln\left(\frac{R_{\mathrm{c}}}{W_{\mathrm{i}}}\right) \tag{3}$$

$$\frac{W_{\mathrm{i}}}{D_{\mathrm{m}}} \geq 60\,,\ \frac{R_{\mathrm{c}}}{W_{\mathrm{i}}} < 10 \qquad \frac{D_{\max}}{D_{\mathrm{m}}} = 2 \cdot 98 - 0 \cdot 54 \ln\left(\frac{R_{\mathrm{c}}}{W_{\mathrm{i}}}\right) \tag{4}$$

A practical safe design curve may then be defined by considering both equations as

$$\frac{D_{\mathrm{max}}}{D_{\mathrm{m}}} = 1 \cdot 5 + 4 \cdot 5 \left(\frac{R_{\mathrm{c}}}{W_{\mathrm{i}}} \right)^{-1} \tag{5}$$

This equation is an asymptotic relationship with a theoretical minimum $D_{\mathrm{max}}/D_{\mathrm{m}}$ of 1.5 representing pool scour depths expected in a straight channel with a pool-riffle bed topography. From this upper-bound relationship, $D_{\mathrm{max}}/D_{\mathrm{m}}$ ranges from 4 to 3 for $R_{\mathrm{c}}/W_{\mathrm{i}}$ between 1.8 and 3. For channels with an $R_{\mathrm{c}}/W_{\mathrm{i}}$ of less than 1.8 pool depth is independent of bend curvature and it is recommended that the dimensionless scour depth should be fixed at 4. All three relationships are portrayed in Figure 54, which shows that this equation is a safe curve for both classes of $W_{\mathrm{i}}/D_{\mathrm{m}}$.

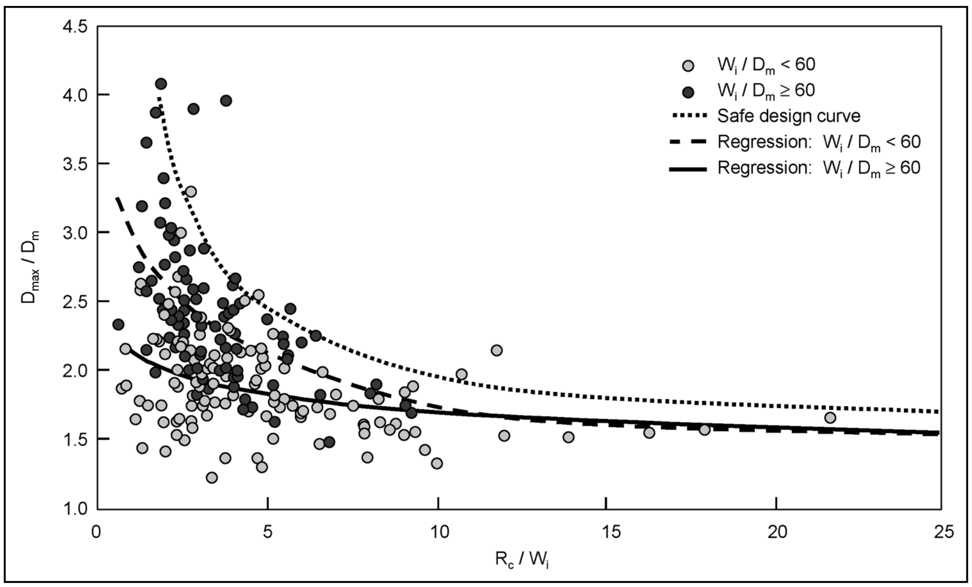

Figure 54. Dimensionless maximum scour depth in meander pools as a function of radius of curvature-to-width ratio. Source data: Thorne and Abt (1993); Maynord (1996)

Practical channel design equations for meander bend geometry

Assuming that confidence is primarily a function of sample size in the analysis of planform width variability, it is possible to derive a mean band of uncertainty, *u*, suitable for all three types of meander bends to provide a set of practical design equations. The cumulative effects of e-type, b-type and c-type bends are represented by the binary parameters, T_{e}, T_{b} and T_{c}, respectively. The value of T_{e} has a value of 1 for all three types of bend and represents the smallest planform width ratio. If point bars are present but chute channels are rare, then T_{b} is assigned a value of 1 and T_{c} is assigned a value of 0. If point bars are present and chute channels are common, then both T_{b} and T_{c} are assigned values of 1. Obviously T_{c} can only be given a value of 1 when T_{b} has a value of 1.

Bend Apex (P>=1.2) $$\frac{W_a}{W_i} = 1 \cdot 05\ T_e + 0 \cdot 30\ T_b + 0 \cdot 44\ T_c \pm u \quad (6)$$

Pool Width (P>=1.2): $$\frac{W_p}{W_i} = 0 \cdot 95\ T_e + 0 \cdot 20\ T_b + 0 \cdot 14\ T_c \pm u \quad (7)$$

For all three bend types and sinuosities greater than 1, the pool offset ratio is given by

Pool-Offset (P>1.0) $$\frac{Z_{a\text{-}p}}{Z_{a\text{-}i}} = 0 \cdot 36 \pm u \quad (8)$$

Values of u refer to confidence limits on the mean response as given in Table 9.

Table 9
Uncertainty, *u*, in Estimates of Width Variability Around Meander Bends and Location of Pools (Values Refer to Confidence Limits on the Mean Response)

Confidence Limits (%)	W_a / W_i	W_p / W_i	$Z_{a\text{-}p} / Z_{a\text{-}i}$
99	0.07	0.17	0.11
95	0.05	0.12	0.08
90	0.04	0.10	0.07

A practical design equation for predicting or constructing maximum scour depths at bends is the upper-bound curve in Figure 54, given by the following equation

$$\frac{D_{\max}}{D_{\mathrm{m}}} = 1.5 + 4.5\left(\frac{R_{\mathrm{c}}}{W_{\mathrm{i}}}\right)^{-1} \quad (9)$$

For sites where active meandering is not permitted, bank protection will be required along the outer bank to prevent erosion. In addition, this equation should be used together with bank stability charts to establish whether bank stabilization against mass failure is also necessary.

Sediment Impact Assessment

The potential success of a river project is often defined in terms of performance based on a single flow event and the sediment load transported by this event. This approach does not account for the potential for instability driven by other flow events in the long-term record. The potential for restoring sediment continuity through the restored reach requires an assessment of the sediment budget, which is determined by the magnitude and frequency of all sediment-transporting flows. To attain geomorphic stability through sediment continuity in the medium- to long-term, the mean annual sediment load for the restored channel (capacity) must match the mean annual sediment load in the supply reach (supply).

On this basis, the sediment impact assessment is a closure loop at the end of the design procedure to: (a) validate the efficacy of the restored channel geometry; (b) identify flows which may cause aggradation or degradation over the short term (these changes are inevitable and acceptable in a dynamic channel); and (c) recommend minor adjustments to the channel design to ensure that dynamic stability will be continued over the medium- to long-term. This can be accomplished using a sediment budget approach for relatively simple projects or by using a numerical model that incorporates solution of the sediment continuity equation for more complex projects. An example of a sediment impact assessment is given in Appendix G.

Adopting this approach should result in a low maintenance channel, with environmental and economic benefits that are sustainable in the long-term. This step is especially important if the restored reach is part of a flood damage reduction project. In such cases it may be necessary to design a channel that is less than ideal in terms of channel stability in order to achieve flood-control benefits. Typically, a compound channel design provides the best combination of benefits.

Topics to report

The following subject areas should be included in a sediment impact assessment report. Some of these items should have been completed early in the study process as part of the geomorphological assessment.

a. The project boundaries and study area boundaries should be identified. The study area should include the area affected by the project. The project's effect on water-surface elevations and sediment transport capacity upstream and downstream of the proposed improvements should be determined. This includes effects the project may have on tributaries, such as headcutting or induced deposition.

b. Available data sources should be identified and the need for additional data collection determined.

c. A site reconnaissance should be conducted to identify the stability of the existing channel as well as existing problems upstream and downstream from the proposed project area. The type of bed-material sediment load should be determined, and bed-material samples collected. Aggradation and/or degradation in the project reach should be noted. The land use in the basin should be noted, especially if there has been any significant changes recently or historically.

d. A brief history of stream behavior in the study reach should be developed. This history should describe aggrading and/or degrading trends, land use changes, behavior of the system during flood events, and historical changes to and by the river system.

e. A sediment budget analysis should be conducted. This is the recommended approach for determining the severity of long-term aggradation or degradation trends, maintenance requirements, reliability during passage of a design flood hydrograph, the need for upstream and tributary control measures to allow for changes in the water-surface elevations due to the project, and the need to make certain the tailwater rating curve is stable.

Sediment budget analysis

The sediment budget analysis is the analytical backbone of the sediment impact assessment. This analysis provides relative stability comparisons for various alternatives, and provides an assessment of the general stability of proposed plans. The level of confidence that can be assigned to the sediment budget approach is a function of the reliability of the available data about the stream and the project. The recommended steps for conducting a sediment budget analysis during the design phase of the study are essentially the same as those used during the geomorphic assessment, as discussed in chapter 4. In the final design phase of the study, sediment yield from the supply reach is compared to sediment yield through the designed channel reach.

Sediment rating curve analysis

The sediment-rating curve analysis discussed as part of the geomorphic assessment in chapter 4 can also be used to evaluate the project design. This qualitative technique does not require stream gage data or sediment gage data. A sediment rating curve is calculated for the proposed and project reaches following the same procedure as is used in the sediment budget analysis. However, instead of gage data, peak flows are used which can be estimated using regional regression or hydrologic modeling. The basic approach is to assess the sediment transport character of a study reach by comparing its sediment transport capacity to that of its supply reach. This approach is illustrated in Figure 55.

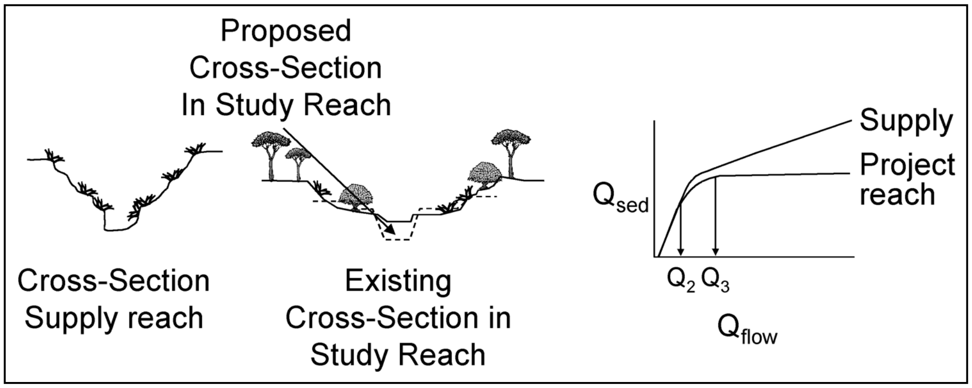

Figure 55. Analogy sediment analysis of proposed project conditions

A comparison of the two sediment-rating curves in Figure 55 indicates that the project reach should be able to transport the incoming sediment load through a discharge of Q_2. Above this discharge, deposition is a possibility with a strong possibility of aggradation occurring above Q_3. These discharges can be compared to the peak discharges of estimated storm frequencies to provide a qualitative estimate of project life. However, since there is no calibration, the actual quantity of deposition cannot be estimated. In addition, this approach does not account for changes in sediment transport capacity, which may occur as sediment deposits in the section and changes its geometry. This technique can be used in conjunction with the sediment budget analysis to assess possible impacts of extreme storm events.

Numerical Sedimentation Modeling

The most reliable way to determine the long-term effects of changes in a complex mobile-bed channel system is to use a numerical model such as HEC-6. River systems are governed by complicated dependency relationships, where changing one significant geometric feature or boundary condition affects other geometric features and flow characteristics both temporally and spatially. Changes at any given location in a stream system are directly related to the inflow of sediment from upstream. This makes the application of the sediment continuity equation essential to any detailed analysis. The most significant of these relationships and the continuity of sediment mass are accounted for in the numerical model approach. The fact that application of a numerical sediment model requires knowledge of sediment transport and river mechanics should not be a deterrent to its use; that knowledge is required for any responsible design work in a river system. It should be expected that an analysis of system response in a complicated system, such as a mobile-bed river system, would require some engineering effort. That effort should be based on analysis of the physical laws that govern the system. The system cannot be expected to adhere to constraints placed on it in violation of natural physical laws, no matter how well intentioned or frugally those constraints were developed. The critical decision with respect to using a numerical model should be based on whether or not significant changes are expected to occur in the system as a result of the proposed design work. In the Corps, this decision typically is reached in the reconnaissance level planning study using the sediment impact assessment approach.

Operation and Maintenance Requirements

Hydraulic design input for assessing operation and maintenance requirements for a stream restoration project are the same as requirements for hydraulic design input for local flood protection projects found in ER 1110-2-1405. Operation and maintenance requirements for a project should ensure the functionality of the project and should be clearly outlined in the project design document and clearly defined in the project Operation and Maintenance manual. These requirements should stand the tests of safety, reliability, functionality, cost-effectiveness and environmental consciousness. Environmental considerations in the conduct of operation and maintenance should be clearly defined in the Operation and Maintenance manual and coordinated in advance with applicable resource agencies to ensure that these requirements can be implemented in a timely manner and also within environmental resource constraints.

Operation should include a monitoring program to ensure that the project behaves as designed. It must be recognized that there are design uncertainties associated with stream restoration design. It may be necessary to make project design adjustments due to unexpected response. It is also possible that an infrequent high flow event may impart severe damage to the project before stabilization measures such as vegetation have had time to become established. The monitoring program should include data collection quantifying changes in average channel dimensions, bank erosion, aggradation or degradation, erosion in the vicinity of structures, and vitality of vegetation.

Quality Management

Stream restoration projects should undergo established quality management processes to ensure that the products being developed meet or exceed expectations.

6 Conclusions

This report presents a new systematic methodology for hydraulic and morphological design of stream restoration projects. The methodology employs both geomorphological principles and analytical engineering formulae. The objective of the methodology is to fit the stream restoration project into the natural system within physical constraints imposed by past development of the floodplain and other project objectives. It is critical that the design process has participation from all the project stakeholders and from a range of scientific disciplines to ensure that the project will meet expectations and objectives.

The basic steps in the hydraulic design methodology are as follows:

a. Define project objectives and constraints in cooperation with stakeholders.

b. Determine the hydrologic regime, including discharge frequencies, flow-duration curves, and channel-forming discharge.

c. Conduct a geomorphological analysis to assess historical channel stability and to determine the dominant geomorphological trends currently active in the watershed.

d. Determine average hydraulic dimensions for a stable main channel using both geomorphological principles and hydraulic formulae. The required dependent design dimensions are width, depth, slope, and planform. Techniques presented in this report include analogy methods, hydraulic geometry methods, and analytical methods that can be facilitated using the SAM hydraulic design package.

e. Conduct a sediment impact assessment to determine the impact that the full range of natural flows will have on project stability. The primary focus of the sediment impact assessment is the sediment budget that compares sediment inflow to the project to sediment transport capacity through the project. The initial design should be refined and modified until input and capacity are closely matched.

The scope of a hydraulic analysis of a stream restoration project will vary depending on the stage of the planning process and the magnitude of channel instability problems. Two examples of stream restoration projects are described in Appendixes F and G. Appendix F is a scope of work for a stability analysis that might be conducted early in the planning process to define dominant geomorphological processes. Appendix G is an example sediment impact assessment using methods described in this report.

References

American Society of Civil Engineers (ASCE). (1993). “Design and construction of urban stormwater management systems - manuals and reports of engineering practice, No. 77,” Prepared by the Task Committee of the Urban Water Resources Research Council of the American Society of Civil Engineers and the Water Environment Federation, New York.

American Society of Civil Engineers (ASCE) Task Committee on Hydraulics, Bank Mechanics, and Modeling of River Width Adjustment. (1998). “River width adjustment, Part I: Processes and mechanics, and Part II: Modeling,” *Journal of Hydraulic Engineering* ASCE, 124(9), 881-917.

Andrews, E. D. (1980). “Effective and bankfull discharge of streams in the Yampa Basin, western Wyoming,” *Journal of Hydrology* 46, 311-330.

Andrews, E. D. (1984). “Bed-material entrainment and hydraulic geometry of gravel-bed rivers in Colorado,” *Bulletin of the Geological Society of America*, 95, 371-378.

Annable, W. K. (1996). *Database of morphological characteristics of watercourses in southern Ontario.* Ministry of Natural Resources, Ontario, Canada.

Bell, M. C. (1986). *Fisheries handbook of engineering requirements and biological criteria*. Fish Passage Development and Evaluation Program, U.S. Army Engineer Division, North Pacific, Portland, OR.

Benson, M. A., and Thomas, D. M. (1966). “A definition of dominant discharge,” *Bulletin of the International Association of Scientific Hydrology* XI, 76-80.

Biedenharn, D. S.; Derrick, D. L.; Elliott, C., and Watson, C. C. (1997). *The WES stream investigation and streambank stabilization handbook*. U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Biedenharn, D. S., Copeland, R. R., Thorne, C. R., Soar, P. J., Hey, R. D., and Watson, C. C. (2000). “Effective discharge calculation: A practical guide,” ERDC/CHL TR-00-15, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Biedenharn, D. S., and Thorne, C. R. (1994). “Magnitude-frequency analysis of sediment transport in the Lower Mississippi River,” *Regulated Rivers* 9, 237-251.

Bray, D. I. (1972). “Generalized regime-type analysis of Alberta rivers,” Ph.D. diss., Department of Civil Engineering, University of Alberta Publication.

Bray, D. I. (1982). “Regime equations for gravel-bed rivers,” *Gravel bed rivers*. R. D. Hey, J. C. Bathurst, and C. R. Thorne, ed., Wiley, Chichester, U.K. 517-542.

Brice, J. C. (1975). “Airphoto interpretation of the form and behavior of alluvial rivers,” Final report to the United States Army Research Office, Durham, under Grant Number DA-ARD-D-31-124-70-G89, Washington University, St. Louis, MO.

Brice, J. C. (1984). “Planform properties of meandering rivers,” *River Meandering, Proceedings of the ’83 Rivers Conference*. ASCE, New Orleans, LA. October 24-26, 1983, C. M. Elliott, ed., 1-15.

Brown, S. A. (1996). “Urban drainage design manual,” Hydraulic Engineering Circular No. 22. FHWA-SA-96-078. Federal Highway Administration, Washington, DC.

Brownlie, W. R. (1981). “Prediction of flow depth and sediment discharge in open channels,” Report No. KH-R-43A, California Institute of Technology, W.M. Keck Laboratory of Hydraulics and Water Resources, Pasadena, CA.

Carling, P. A. (1988). “The concept of dominant discharge applied to two gravel-bed streams in relation to channel stability thresholds,” *Earth Surface Processes and Landforms* 13, 355-367.

Carlston, C. W. (1965). “The relation of free meander geometry to stream discharge and its geomorphic implications,” *American Journal of Science* 263, 863-865.

Chang, H. H. (1980). “Stable alluvial canal design,” *Journal of the Hydraulics Division* ASCE, 106 (HY5), 873-891.

Charlton, F. G., Brown, P. M., and Benson, R. W. (1978). “The hydraulic geometry of some gravel rivers in Britain,” Report IT 180, Hydraulics Research Station, Wallingford, U.K.

Cherry, D. S., Wilcock, P. R., and Wolman, M. G. (1996). “Evaluation of methods for forecasting planform change and bankline migration in flood-control channels,” Final report to U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, Johns Hopkins University, Baltimore, MD.

Chitale, S. V. (1970). “River channel patterns,” *Journal of the Hydraulics Division*, ASCE, 96, 201-221.

Church, M. A., and Kellerhals, R. (1978). “On the statistics of grain size variation along a gravel river,” *Canadian Journal of Earth Sciences* 15, 1151-1160.

Church, M. A., McLean, D. G., and Wolcott, J. F. (1987). “River bed gravels: sampling and analysis,” Sediment Transport in Gravel-Bed Rivers, C. R. Thorne, J. C. Bathurst, and R. D. Hey, ed., John Wiley and Sons Ltd., Chichester, Great Britain, 43-88.

Copeland, R. R. (1994). “Application of channel stability methods – Case studies,” TR HL-94-11, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Copeland, R. R. (1995). “Sediment impact assessment, Carlsbad, New Mexico,” Miscellaneous Paper HL-95-7, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Diplas, P., and Vigilar, G. G. (1992). “Hydraulic geometry of threshold channels,” *Journal of Hydraulic Engineering* ASCE, 118(4), 597-614.

Downs, P. W., and Thorne, C. R. (1996). “The utility and justification of river reconnaissance surveys*,” Transactions of the Institute of British Geographers,* New Series, 21, 455-468. ISSN 0020-2754.

Dury, G. H. (1965). “Theoretical implications of underfit streams,” U.S. Geological Survey, Professional Paper, 452C.

Dury, G. H. (1973). “Magnitude-frequency analysis and channel morphology,” *Fluvial geomorphology: A proceedings volume of the fourth annual geomorphology symposia series.* M. Morisawa, ed., Binghampton, NY, Allen and Unwin, 91-121.

Einstein, H. A. (1950). “The bed-load function for sediment transportation in open channel flows,” Technical Bulletin No. 1026, U.S. Department of Agriculture, Soil Conservation Service, Washington, DC.

Emmett, W. W. (1972). “The hydraulic geometry of some Alaskan streams south of the Yukon River,” U.S. Geological Survey, Open File Report 72-108.

Emmett, W. W. (1975). “The channels and waters of the upper Salmon River, Idaho,” Geological Survey Professional Paper 870A, Washington, DC.

Einstein, H. A. (1950). “The bed-load function for sediment transport in open channel flows,” Technical Bulletin No. 1026, U.S. Department of Agriculture, Soil Conservation Service, Washington DC.

Federal Interagency Stream Restoration Working Group. (1998). *Stream corridor restoration: Principles, processes, and practices* NRCS, Washington, DC. *http://www.usda.gov/stream_restoration*.

Ferguson, B. K. (1991). "The failure of detention and the future of stormwater design," *Landscape Architecture Techniques and Practice – Construction* December.

Ferguson, B. K. (1994). *Stormwater infiltration.* Lewis Publishers, CRC Press, Boca Raton, FL.

Geological Survey Professional Paper 422-H, Washington, DC.

Graybill, F. A., and Iyer, H. K. (1994). *Regression analysis: Concepts and applications.* Duxbury Press, Belmont, CA.

Hey, R. D. (1975). "Design discharge for natural channels," *Science, technology and environmental management*, R. D. Hey and T. D. Davies, ed., Saxon House, 73-88.

Hey, R. D. (1976). "Geometry of river meanders," *Nature* 262, 482-484.

Hey, R. D. (1997). "Channel response and channel forming discharge: literature review and interpretation," Final Report for the U.S. Army Contract Number R&D 6871-EN-01.

Hey, R. D., and Thorne, C. R. (1983). "Accuracy of surface samples from gravel-bed material," *Journal of Hydraulic Engineering* 109, 842-851.

Hey, R. D., and Thorne, C. R. (1986). "Stable channels with mobile gravel-beds," *Journal of Hydraulic Engineering* ASCE, 112, 6, 671-689.

Ikeda, S., Parker, G., and Kimura, Y. (1988). "Stable width and depth of straight gravel rivers with heterogeneous bed materials," *Water Resources Research*, 24(5), 713-722.

Ikeda, S., and Izumi, N. (1990). "Width and depth of self-formed straight gravel rivers with bank vegetation," *Water Resources Research* 26(10), 2353-2364.

Inglis, C. C. (1941). "Digest of answers to the Central Board of Irrigation questionnaire on meandering of rivers with comments on factors controlling meandering and suggestions for future actions," *Central board of irrigation annual technical report, 1939-1940 session.* Publication 24, A. R. Edgecombe, ed., Poona, India, 100-114.

Knighton, A. D. (1984). *Fluvial forms and processes.* Arnold, London.

Langbein, W. B., and Leopold, L. B. (1966). "River meanders - Theory of minimum variance," Geological Survey Professional Paper 422-H, Washington DC.

Lane, E. W. (1955). "Design of stable channels," *Transactions of the American Society of Civil Engineers.* 120, 1234-1279.

Leopold, L. B. (1994). *A view of the river.* Harvard University Press, Cambridge.

Leopold, L. B., and Maddock, T. (1953). "The hydraulic geometry of stream channels and some physiographic implications," Geological Survey Professional Paper 252*,* Washington, DC.

Leopold, L. B., and Wolman, M. G. (1957). "River channel patterns - braided, meandering and straight," Geological Survey Professional Paper 282B*,* Washington, DC.

Leopold, L. B., and Wolman, M. G. (1960). "River meanders," *Bulletin of the Geological Society of America* 71, 769-794.

Leopold, L. B., Wolman, M. G., and Miller, J. P. (1964). *Fluvial processes in geomorphology.* W.H. Freeman & Company, San Francisco.

Limerinos, J. T. (1970). "Determination of the Manning coefficient from measured bed roughness in natural channels," U.S. Geological Survey Water-Supply Paper 1898-B, Washington DC.

Linsley, R. K., Kohler, M. A., and Paulhus, J. L. H. (1975). *Hydrology for engineers.* McGraw-Hill Series in Water Resources and Environmental Engineering, New York.

Maryland Department of the Environment. (1984). *Infiltration design and practice.* Baltimore MD.

Maryland Department of the Environment. (1987). Design procedures for stormwater management detention structures. Baltimore, MD.

Maynord, S. T. (1996). "Toe scour estimation in stabilized bendways," *Journal of Hydraulic Engineering* ASCE, 122(8), 460-464.

McCuen, R. H., and Moglen, G. E. (1988). "Multicriterion stormwater management methods," *Journal of Water Resources Management* ASCE, 114 ,4.

Meyer-Peter, E., and Muller, R. (1948). "Formulas for bed-load transport," *Proc. Second Meeting of the International Association for Hydraulic Research*, Stockholm, Sweden, 39-64.

Millar, R. G., and Quick, M. C. (1993). "Effect of bank stability on geometry of gravel rivers," *Journal of Hydraulic Engineering* ASCE, 119 (12), 1343-1363.

Moglen, G. E., and McCuen, R. H. (1988). "Effects of detention basins on in-stream sediment movement," *Journal of Hydrology* 104.

Myers, R. H. (1990). *Classical and modern regression with applications*. Second ed., Duxburg Press, Belmont, CA.

Nixon, M. (1959). "A study of bankfull discharges of rivers in England and Wales," *Proceedings of the Institution of Civil Engineers* 12, 157-175.

Nolan, K. M., Lisle, T. E., and Kelsey, H. M. (1987). "Bankfull discharge and sediment transport in northwestern California," *Proceedings of the Corvallis Symposium, Erosion and Sedimentation in the Pacific Rim,* International Association of Hydrological Sciences, Publication No. 165, Wallingford U.K. 439-449.

Parker, G. (1978). "Self-formed straight rivers with equilibrium banks and mobile bed, Part I – The sand-silt river," *Journal of Fluid Mechanics* 89, 109-125.

Parker, G. (1979). "Hydraulic geometry of active gravel rivers," *Journal of the Hydraulics Division*, ASCE, 105(9), 1185-1201.

Parker, G. (1990). "The ACRONYM series of Pascal programs for computing bedload transport in gravel rivers," External Memorandum No. M-220, St. Anthony Falls Hydraulic Laboratory, University of Minnesota.

Pickup, G., and Warner, R. F. (1976). "Effects of hydrologic regime on magnitude and frequency of dominant discharge," *Journal of Hydrology* 29, 51-75.

Prince Georges County, Maryland. (1997). *Low impact development design manual.* Department of Environmental Resources, Marlboro, MD.

Richards, K. S. (1982). *Rivers: Form and process in alluvial channels.* Methuen, London.

Scharffenberg, W. A. (2001*). Hydrologic Modeling System HEC-HMS User's Manual,* USACE Hydrologic Engineering Center, Davis, CA.

Schoklitsch, A. (1930). "Handbuch des wasserbaues," Springer, Vienna (2nd ed.) English translation by S. Shulits., "The Schoklitsch bed-load formula," *Engineering*. London, England, June 21, 1935, 644-646; and June 28, 1935, 687.

Schueler, T. (1994). "The importance of impervious," *Watershed Protection Techniques* 1(3).

Schumm, S. A. (1960). "The shape of alluvial channels in relation to sediment type," Geological Survey Professional Paper 352-B, Washington DC.

Schumm, S. A. (1967). "Meander wavelength of alluvial rivers," *Science*, 157, 1549-1550.

Schumm, S. A. (1968). "River adjustment to altered hydrologic regime – Murrumbidgee River and Palaeochannels, Australia," U.S. Geological Survey Professional Paper, 598.

Schumm, S. A. (1977). *The fluvial system*. Wiley, New York.

Schumm, S. A., Harvey, M. D., and Watson, C. C. (1984). *Incised channels, morphology, dynamics and control.* Water Resources Publications, Littleton, CO.

Shulitis, S. (1935). "The Schoklitsch bed-load formula," *Engineering* London, England, June 21, 644-646 and June 28, 687.

Simon, A. (1989). "A model of channel response in disturbed alluvial channels," *Earth Surface Processes and Landforms* 14, 11-26.

Soar, P. J., and Thorne, C. R. (2001). "Channel restoration design for meandering rivers," ERDC/CHL CR-01-1, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Thomas, W. A., Copeland, R. R., McComas, D. N., and Raphelt, N. K. (2000). "User's manual for the SAM hydraulic design package for channels," U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Thorne, C. R. (1988). "Bank processes on the Red River between Index, Arkansas and Shreveport, Louisiana," Final Report to U.S. Army European Research Office, under contract DAJA45-88-C-0018, Department of Geography, Queen Mary College, U.K.

Thorne, C. R. (1997). "Channel types and morphological classification," *Applied fluvial geomorphology for river engineering and management.* C. R. Thorne, R. D. Hey, and M. D. Newson, ed., Wiley, Chichester, U.K., 175-222.

Thorne, C. R. (1998). *Stream reconnaissance handbook*. Wiley, Chichester, U.K.

Thorne, C. R., and Abt, S. R. (1993). "Velocity and scour prediction in river bends," Contract Report HL-93-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Thorne, C. R., Simon, A., and Allen, R. (1996). "Geomorphological river channel reconnaissance for river analysis, engineering and management," *Transactions of the Institute of British Geographers*, New Series, 21, 469-483.

Thorne, C. R., and Soar, P. J. (2000). "Analysis of channels with compound channels for channel restoration design," Final Report submitted to U.S. Army, Research Development and Standardization Group – U.K., London, under contract No. N68171-00-M-5506, Project No. W90C2K-8913-EN01, School of Geography, University of Nottingham, Nottingham, U.K.

U.S. Army Corps of Engineers (USACE). (1982). "Hydraulic Design for Local Flood Protection Projects," ER 1110-2-1405, Washington, DC.

U.S. Army Corps of Engineers (USACE). (1993). "Hydrologic frequency analysis," EM 1110-2-1415, Washington, DC.

U.S. Army Corps of Engineers (USACE). (1994a). “Channel stability assessment for flood control projects,” EM 1110-2-1418, Washington, DC.

U.S. Army Corps of Engineers (USACE). (1994b). “Hydraulic design of flood control channels,” EM 1110-2-1601, Washington, DC.

U.S. Army Corps of Engineers (USACE). (1995). “Sedimentation investigations in rivers and reservoirs,” EM 1110-2-4000, Washington, DC.

U.S. Army Corps of Engineers (USACE). (1996). “Risk-based analysis for flood damage reduction studies,” EM 1110-2-1619, Washington, DC.

U.S. Army Corps of Engineers Hydrologic Engineering Center (USACE, HEC). (1981). “HEC-1 flood hydrograph package, Users manual,” Davis, CA.

U.S. Army Corps of Engineers Hydrologic Engineering Center (USACE, HEC). (1990). “HEC-2 water surface profiles, users manual,” Davis, CA.

U.S. Army Corps of Engineers Hydrologic Engineering Center (USACE, HEC). (1993). “HEC-6 scour and deposition in rivers and reservoirs, users manual,” Davis, CA.

U.S. Army Corps of Engineers Hydrologic Engineering Center (USACE, HEC). (2001). “HEC-RAS river analysis system, users manual,” Davis, CA.

U.S. Army Engineers (USAE). (1991). Tidal Hydraulics, EM 1110-2-1607, Office of the Chief of Engineers, Washington, DC.

U.S. Army Engineers (USAE). (1989). “Sediment investigations of river and reservoirs,” EM 1110-2-4000, Office of the Chief of Engineers, Washington, DC, Change 1.

U.S. Department of Agriculture (USDA). (1977). *Design of open channels.* Technical Release No. 25. Soil Conservation Service, Washington, DC.

U.S. Interagency Advisory Committee on Water Data (USIACWD). (1982). “Guidelines for determining flood flow frequency, Bulletin 17-B of the Hydrology Subcommittee,” Office of Water Data Coordination, U.S. Geological Survey, Reston, VA.

U.S. Geological Survey (USGS). (1993). “Nationwide summary of U. S. Geological Survey regional regression equations for estimating magnitude and frequency of floods for ungaged sites,” Water-Resources Investigations Report 94-4002.

U.S. Soil Conservation Service (SCS). (1985). *National engineering handbook, Section 4, “Hydrology.”* U.S. Department of Agriculture, Washington, DC.

U.S. Soil Conservation Service (SCS). (1986). “Urban hydrology for small watersheds,” Technical Release 55 (TR-55).

U.S. Soil Conservation Service (SCS). (1992). “Computer program for project formulation hydrology,” Technical Release 20 (TR-20).

Watson, C. C., Dubler, D., and Abt, S. R. (1997). “Demonstration erosion control project report, design hydrology investigations,” Fort Collins, Colorado, submitted to U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Webb, B. W., and Walling, D. E. (1982). “The magnitude and frequency characteristics of fluvial transport in the Devon drainage basin and some geomorphological implications,” *Cantena*, 9, 9-23.

White, W. R., Bettess, R., and Paris, E. (1982). “Analytical approach to river regime,” *Journal of the Hydraulics Division* ASCE, 108 (HY10), 1179-1193.

Williams, G. P. (1978). “Bankfull discharge of rivers,” *Water Resources Research* 14(6), 1141-1154.

Williams, G. P. (1986). “River meanders and channel size,” *Journal of Hydrology* 88, 147-164.

Wolman, M. G. (1954). “A method for sampling coarse river-bed material,” *American Geophysical Union Transactions.* 35(6), 951-956.

Wolman, M. G. (1955). “The natural channel of Brandywine Creek Pennsylvania,” Geological Survey Professional Paper 271, Washington, DC.

Wolman, M. G., and Leopold, L. B. (1957). “River flood plains: Some observations on their formation,” Geological Survey Professional Paper 282-C, Washington, DC.

Wolman, M. G., and Miller, J. P. (1960). “Magnitude and frequency of forces in geomorphic processes,” *Journal of Geology* 68, 54-74.

Woodyer, K. D. (1968). “Bankfull frequency in rivers,” *Journal of Hydrology* 6, 114-142.

Yalin, M. S. (1992). *River mechanics*. Pergamon Press, New York.

Yang, C. T. (1973). “Incipient motion and sediment transport,” *Journal of the Hydraulics Division*, American Society of Civil Engineers, 99, (HY10), 1679-1704.

Yang, C. T. (1984). “Unit stream power equation for gravel*,” Journal of the Hydraulics* Division, American Society of Engineers, 110 (12) 1783-1797.

Yevjevich, V. (1972). *Probability and statistics in hydrology*. Water Resources Publications, Fort Collins, CO.

Yuzyk, T. R. (1986). “Bed material sampling in gravel-bed streams, sediment survey section, environment Canada,” IWD-HQ-WRB-SS-86-8, Ottawa, Ontario.

Appendix A
Flow Duration Curves for Effective Discharge Calculation[1]

A standardized procedure is required to ensure that effective discharge calculations are accurate and that results from different sites can be compared. To be practical, the procedure must use only data that are readily available from gaging stations, or that can be synthesized using limited additional computations.

The basic approach is to divide the range of river flows during the period of record into a number of arithmetic classes and then calculate the total sediment quantity transported by each class. This is achieved by multiplying the frequency of occurrence of each flow class by the median sediment load for that flow class (Figure A1). The initial data required are flow duration data and a sediment transport rating curve.

The calculated value of the effective discharge depends to some extent on the steps used to manipulate the input data to define the flow regime and sediment transport function. The procedure described here represents the best practice in this regard, based on extensive first-hand experience in using flow and sediment transport data to determine the effective discharge.

Gaged Sites

The first step in an effective discharge calculation is to group the discharge data into equal arithmetic flow classes and determine the number of events occurring in each class during the period of record. Logarithmic or nonequal width arithmetic classes introduce systematic bias into the calculation of effective discharge and should not be used. Grouping the discharge data is usually accomplished using a flow-duration curve, which is a cumulative distribution function of observed discharges at the gaging station. Figure A2 is an example of a flow-duration curve calculated for the Sevier River, UT. The flow-duration

[1] Extracted from Biedenharn et al. (2000). All references cited in this appendix are listed in the References section following the main text.

curve defines the percentage of time a particular discharge is equaled or exceeded. The frequency of occurrence of each discharge class is calculated from this curve. Three critical components must be considered when developing a flow-duration curve: the number of discharge classes; the time base for discharge averaging; and the length of the period of record. It is important that the historical record is homogeneous, i.e., watershed conditions are unchanged.

Class interval and number of classes

The selection of class interval can influence the effective discharge calculation. Intuitively, it might be expected that the smaller the class interval and, therefore, the greater the number of classes, the more accurate would be the outcome. However, when too small an interval is used, discontinuities appear in the discharge frequency distribution. These in turn produce a rather irregular sediment load histogram having multiple peaks. Therefore, the selected class interval should be small enough to accurately represent the frequency distribution of flows but large enough to produce a continuous distribution.

There are no definite rules for selecting the most appropriate interval and number of classes, but Yevjevich (1972) stated that the class interval should not be larger than $s/4$, where s is an estimate of the standard deviation of the sample. For hydrological applications he suggested that the number of classes should be between 10 and 25, depending on the sample size.

Hey (1997) found that 25 classes with equal, arithmetic intervals produced a relatively continuous flow frequency distribution and a smooth sediment-load histogram with a well defined peak, indicating an effective discharge which corresponded exactly with bankfull flow. A smaller interval, and correspondingly larger number of classes, produced anomalous results. Experience has shown that in some cases, 25 classes produce unsatisfactory results and that up to 250 classes may be required. Particular care has to be exercised on rivers where there is a high incidence of very low flows. Under these circumstances, the effective discharge may be biased towards the lowest discharge class.

Time base

Mean daily discharges are conventionally used to construct the flow-duration curve. Although this is convenient, given the ready availability of mean daily discharge data from the United States Geological Survey (USGS), it can, in some cases, introduce error into the calculations. This arises because mean daily values can underrepresent the occurrence of short-duration, high magnitude flow events that occur within the averaging period, while overrepresenting effects of low flows.

On large rivers, such as the Mississippi, the use of the mean daily values is acceptable because the difference between the mean and peak daily discharges is negligible. However, on smaller streams, flood events may last only a few hours and the peak daily discharge can be much greater than the corresponding mean daily discharge. Excluding the flood peaks and the associated high sediment loads can result in underestimation of the effective discharge. Rivers with a high flashiness index, defined as the ratio of the instantaneous peak flow to the associated daily mean flow, are most likely to be affected. To avoid this problem it may be necessary to reduce the time base for discharge averaging from 24 hr (mean daily) to 1 hr, or even 15 min on flashy streams. For example, an investigation of discharge data for 11 USGS gaging stations in the Yazoo River Basin, MS, revealed that the annual yields of bed material calculated using mean daily discharge data were approximately 50 percent less than the yields calculated using 15-min data (Watson, Dubler, and Abt 1997). These are relatively small basins (drainage areas less than 1,000 km^2) with high rainfall intensities and runoff characteristics that have been severely affected by land-use change and channel incision. Consequently, hydrographs are characterized by steep rising and falling limbs, with events peaking and returning to base flow in much less than 24 hr.

In practice, mean daily discharge data may be all that are available for the majority of gaging stations and these data may be perfectly adequate. However, caution must be exercised when using mean daily data for watersheds with flashy runoff regimes and short-duration hydrographs. The use of 15-min data to improve the temporal resolution of the calculations should be considered whenever the available flow records allow it.

In the absence of 1-hr or 15-min data, recorded hydrographs from USGS gaging records can be used to refine the high discharge portion of the flow-duration curve. Actual instantaneously recorded hydrographs can be used to determine durations of the highest discharges in the historical record.

Period of record

The period of record must be sufficiently long to include a wide range of morphologically significant flows, but not so long that changes in the climate, land-use or runoff characteristics of the watershed produce significant changes in the data. If the period of record is too short, there is a significant risk that the effective discharge will be inaccurate due to the occurrence of unrepresentative flow events. Conversely, if the period is too long, there is a risk that the flow and sediment regimes of the stream at the beginning of the record may be significantly different to current conditions.

A reasonable minimum period of record for an effective discharge calculation is about 10 years, with 20 years of record providing more certainty that the range of morphologically significant flows is fully represented in the data. Records longer than 30 years should be examined carefully for evidence of temporal changes in flow and/or sediment regimes. If the period of record at a gaging station is inadequate, consideration should be given to developing an effective discharge based on regional estimates of the flow duration.

Ungaged Sites

At locations where gaging records are either unavailable or are found to be unrepresentative of the flow regime, it will be necessary to synthesize a flow-duration curve. There are two possible methods of doing this. The first method is by using records from nearby gaging stations within the same drainage basin. The second is developing a regionalized flow-duration curve.

It must be recognized that these methods simply provide an approximation of the flow-duration characteristics and that there can be considerable uncertainty in the results. The reliability of these methods is a function of the quality of the existing gage data, and the morphologic similarity between the gaged and ungaged locations. Caution is advised whenever the existing gage data are limited, or the site in question has a significantly different morphologic character than the gaged site.

Drainage area - flow duration curve method

This method relies on the availability of gaging station data at a number of sites on the same river as the ungaged location. First, flow duration curves for each gaging station are derived for the longest possible common period of record. This guarantees comparability between the data, as all the gaging stations have experienced the same flow conditions, and ensures that the curves represent the longer period. Provided there is a regular downstream decrease in the discharge per unit watershed area, then a graph of discharge for a given exceedance duration against upstream drainage area will produce a power function with virtually no scatter about the best fit regression line. Figure A3 shows this relationship for the River Wye, UK (Hey 1975). The equations generated by this method enable the flow-duration curve at an ungaged site on that river to be determined as a function of its upstream watershed area.

For sites on streams where there is only one gaging station, flow-duration curves can be estimated at ungaged locations provided the streams are tributaries to rivers where the relation between discharge and drainage area conforms to a known power function. Estimates of the contributing flow to the main stem can be obtained from the difference between discharges on the main stem above and below the tributary junction. Discharge - drainage area relations can then be derived for the tributary given the flow-duration curve at the gaging station and the predicted curve at its confluence with the main stem. However, this technique should not be used if there are distinct and abrupt downstream changes in the discharge per unit area for the watershed. This could occur if portions of the drainage area consisted of different hydrological regions. In this case it would be preferable to use the regionalized duration curve method described in the next paragraph.

Regionalized duration curve method

An alternative to the use of watershed area to generate a flow-duration curve for an ungaged site is to use a regional-scaling method based on data from watersheds with similar characteristics. For example, Emmett (1975) and Leopold (1994) suggest using the ratio of discharge to bankfull discharge (Q/Q_b) as a nondimensional index to transfer flow-duration relationships between basins with similar characteristics. However, bankfull discharge does not necessarily have either a consistent duration or return period (Williams 1978).

To avoid this problem, a nondimensional discharge index was proposed by Watson, Dubler, and Abt (1997) using the regionalized 2-year discharge to normalize discharges (Q/Q_2). For ungaged sites the 2-year discharge may be estimated from regionalized discharge frequency relationships developed by the USGS (1993) on the basis of regression relationships between the drainage area, channel slope, and slope length. These relationships are available for most states.

The dimensionless discharge index (Q/Q_2) can be used to transfer a flow-duration relationship to an ungaged site from a nearby gaged site. The gaged site may be within the same basin, or an adjacent watershed.

To transfer a flow duration relationship within a watershed use the following steps:

a. Develop the regionalized flow-duration curve. Using a flow-duration curve from a gaged site in a physiographically similar watershed, divide the discharges in the flow-duration relationship by the Q_2 for the gaged site. This creates a dimensionless flow-duration curve. If more than one gage site is available an average dimensionless flow-duration curve for all the sites can be developed.

b. Compute the Q_2 for the ungaged site.

c. Calculate the flow-duration curve for the ungaged site. Multiply the dimensionless ratios from the regionalized flow-duration curve by the ungaged Q_2.

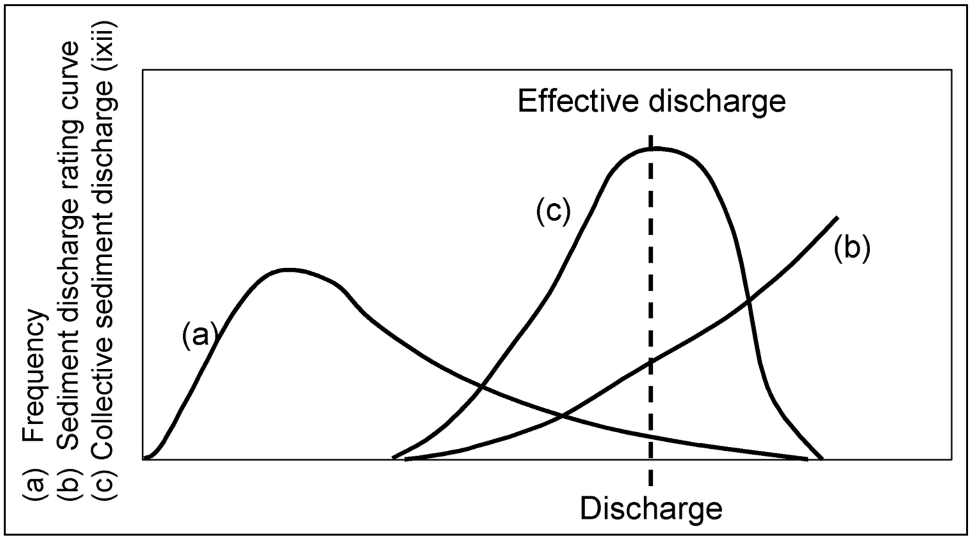

Figure A1. Derivation of total sediment load-discharge histogram (c) from flow frequency (a) and sediment load rating curves (b)

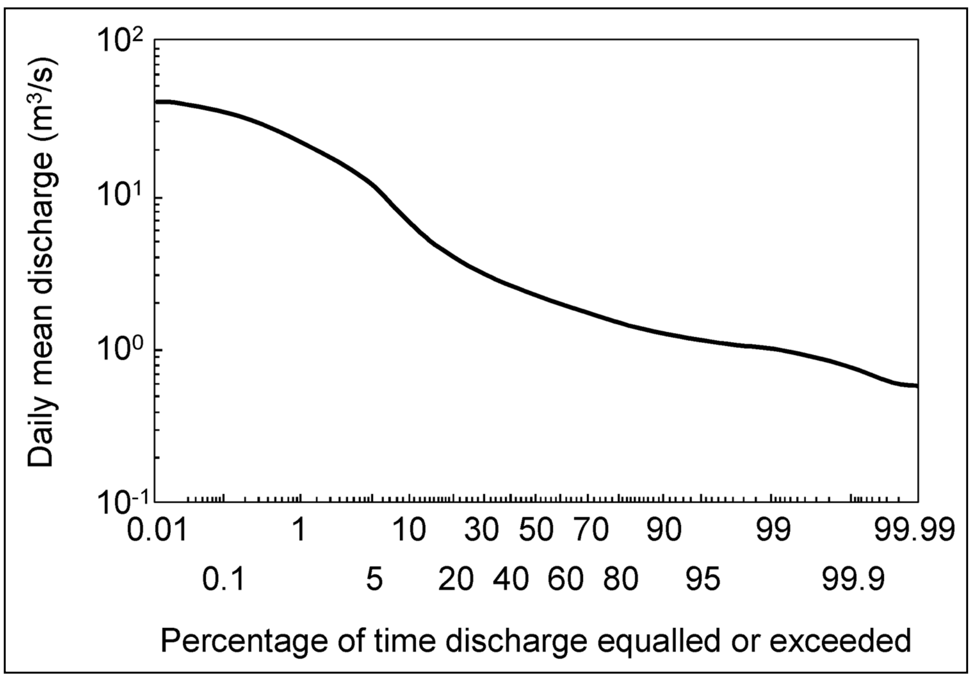

Figure A2. Daily mean flow-duration curve: Sevier River, Hatch, UT (from Hey 1997)

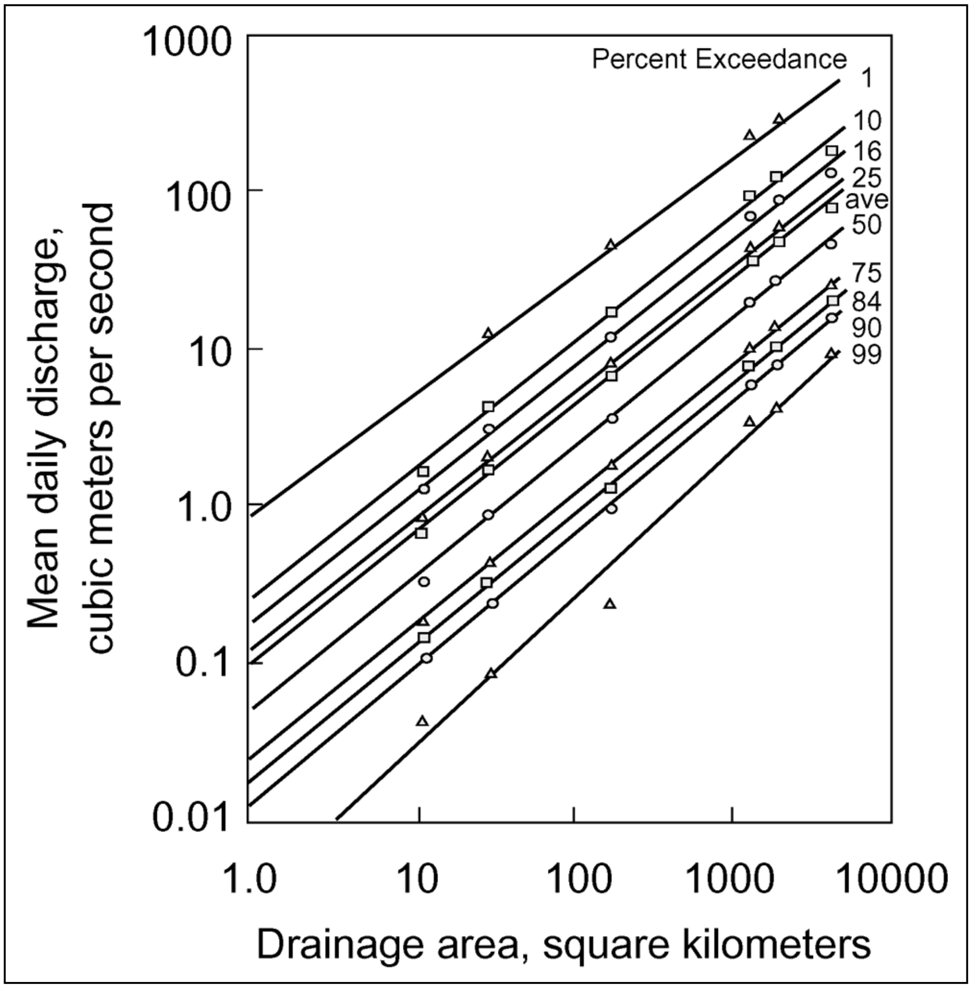

Figure A3. Downstream daily flow-duration curves: River Wye, UK 1937-1962 (Hey 1975)

Appendix B
Stream Reconnaissance Sheets, Thorne (1993)

STREAM RECONNAISSANCE RECORD SHEET

Developed by Colin R. Thorne
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SECTION 1 - SCOPE AND PURPOSE

Brief Problem Statement:-

Purpose of Stream Reconnaissance:-

Logistics of Reconnaissance Trip:-

RIVER LOCATION DATE

From To

PROJECT STUDY REACH

SHEET COMPLETED BY

RIVER STAGE TIME: START TIME: FINISH

General Notes and Comments on Reconnaissance Trip:-

SECTION 2 - REGION AND VALLEY DESCRIPTION

PART 1: AREA AROUND RIVER VALLEY

Terrain	Drainage Pattern	Surface Geology	Rock Type	Land Use	Vegetation
		Bed rock	Sedimentary	Natural	Tropical forest
Mountains	Dendritic	Weathered Soils	Metamorphic	Managed	Temperate forest
Uplands	Parallel	Glacial Moraine	Igneous	Cultivated	Boreal forest
Hills	Trellis	Glacio/Fluvial	None	Urban	Woodland
Plains	Rectangular	Fluvial		Suburban	Savanna
Lowlands	Radial	Lake Deposits			Temperate grassland
	Annular	Wind blown (loess)	**Specific Rock Types (If known)**		Desert scrub
	Multi-Basin				Extreme Desert
	Contorted				Tundra or Alpine
					Agricultural land

Notes and Comments:-

PART 2: RIVER VALLEY AND VALLEY SIDES

Location of River	Height	Side Slope Angle	Valley Side Failures	*Interpretative Observations* *Material Type*	*Severity of Problems*
In Valley	< 5 m			*Bedrock*	
On Alluvial Fan	5 - 10 m	< 5degrees	None	*Soils*	*Insignificant*
On Alluvial Plain	10 - 30 m	5-10 degrees	Occasional	*Loose debris*	*Mild*
In a Delta	30 - 60 m	10-20 degrees	Frequent	***Failure Type***	*Significant*
In Old Lake Bed	60 - 100 m	20-50 degrees	**Failure Locations**	*(see Sketches in Manual)*	*Serious*
Valley Shape	> 100 m	>50 degrees	None		*Catastrophic*
Symmetrical			Away from river		
Asymmetrical			Along river (Undercut)	Level of Confidence in answers (Circle one)	
				0 10 20 30 40 50 60 70 80 90 100 %	

Notes and Comments:-

PART 3: FLOOD PLAIN (VALLEY FLOOR)

Valley Floor Type	Valley Floor Data	Surface Geology	Land Use	Vegetation	Riparian Buffer Strip
		Bed rock	Natural	None	None
None	None	Glacial Moraine	Managed	Unimproved Grass	Indefinite
Indefinite	< 1 river width	Glacio/Fluvial	Cultivated	Improved Pasture	Fragmentary
Fragmentary	1 - 5 river widths	Fluvial: Alluvium	Urban	Orchards	Continuous
Continuous	5-10 river widths	Fluvial: Backswamp	Suburban	Arable Crops	**Strip Width**
	>10 river widths	Lake Deposits	Industrial	Shrubs	None
	Flow Resistance*	Wind Blown (Loess)		Deciduous Forest	< 1 river width
Left Overbank Manning n value				Coniferous Forest	1 - 5 river widths
Right Overbank Manning n value		(* note: n value for channel is recorded in Part 6)		Mixed Forest	> 5 river widths

Notes and Comments:-

PART 4: VERTICAL RELATION OF CHANNEL TO VALLEY

Terraces	Overbank Deposits	Levees	Levee Data	*Interpretative Observations* *Present Status*	*Problem Severity*
None	None	None	Height (m)	*Adjusted*	*Insignificant*
Indefinate	Silt	Natural	Side Slope (o)	*Incised*	*Moderate*
Fragmentary	Fine sand	Constructed		*Aggraded*	*Serious*
Continuous	Medium sand	**Levee Description**	**Levee Condition**		***Problem Extent***
Number of Terraces	Coarse sand	None	None	***Instability Status***	*None*
Trash Lines	Gravel	Indefinite	Intact	*Stable*	*Local*
Absent	Boulders	Fragmentary	Local Failures	*Degrading*	*General*
Present		Continuous	Frequent failures	*Aggrading*	*Reach scale*
Height above		Left Bank			*System wide*
flood plain (m)		Right Bank			*Regional*
		Both Banks		Level of Confidence in answers (Circle one)	
				0 10 20 30 40 50 60 70 80 90 100 %	

Notes and Comments:-

PART 5: LATERAL RELATION OF CHANNEL TO VALLEY

Interpretative Observations

Planform	Planform Data	Lateral Activity	Floodplain Features	*Present Status*	*Problem Severity*
Straight	Bend Radius	None	None	*Adjusted*	*Insignificant*
Sinuous	Meander belt width	Meander progression	Meander scars	*Over wide*	*Moderate*
Irregular	Wavelength	Increasing amplitude	Scroll bars+sloughs	*Too narrow*	*Serious*
Regular meanders	Meander Sinuosity	Progression+cut-offs	Oxbow lakes		***Problem Extent***
Irregular meanders		Irregular erosion	Irregular terrain	***Instability Status***	*None*
Tortuous meanders	**Location in Valley**	Avulsion	Abandoned channel	*Stable*	*Local*
Braided	Left	Braiding	Braided Deposits	*Widening*	*General*
Anastomosed	Middle			*Narrowing*	*Reach scale*
	Right				*System wide*
					Regional

Level of Confidence in percent (Circle one)

0 10 20 30 40 50 60 70 80 90 100 %

Notes and Comments:-

SECTION 3 - CHANNEL DESCRIPTION

PART 6: CHANNEL DESCRIPTION

Dimensions	Flow Type	Bed Controls	Control Types	Width Controls	Control Types
		None	None	None	None
Av. top bank width (m)	None	Occasional	Solid Bedrock	Occasional	Bedrock
Av. channel depth (m)	Uniform/Tranquil	Frequent	Weathered Bedrock	Frequent	Boulders
Av. water width (m)	Uniform/Rapid	Confined	Boulders	Confined	Gravel armor
Av. water depth (m)	Pool+Riffle	Number of controls	Gravel armor	Number of controls	Revetments
Reach slope	Steep + Tumbling		Cohesive Materials		Cohesive Materials
Mean velocity (m/s)	Steep + Step/pool		Bridge protection		Bridge abutments
Manning's n value	(Note: Flow type on day of observation)		Grade control structures		Dykes or groynes

Notes and Comments:-

PART 7: BED SEDIMENT DESCRIPTION

Bed Material	Bed Armour	Surface Size Data	Bed Forms (Sand)	Bar Types	Bar Surface data
Clay	None	D50 (mm)	Flat bed (None)	None	D50 (mm)
Silt	Static-armour	D84 (mm)	Ripples	Pools and riffles	D84 (mm)
Sand	Mobile-armour	D16 (mm)	Dunes	Alternate bars	D16 (mm)
Sand and gravel			Bed form height (m)	Point bars	
gravel and cobbles	**Sediment Depth**	**:rate Size Data**	**Island or Bars**	Mid-channel bars	**Bar Substrate data**
cobbles + boulders	Depth of loose	D50 (mm)	None	Diagonal bars	D50 (mm)
boulders + bedrock	Sediment (cm)	D84 (mm)	Occasional	Junction bars	D84 (mm)
Bed rock		D16 (mm)	Frequent	Sand waves + dunes	D16 (mm)

Notes and Comments:-

Channel Sketch Map

Map Symbols

Study reach limits	North point	Cut bank	Photo point
Cross-section	flow direction	exposed island/bar	Sediment sampling point
Bank profile	impinging flow	structure	Significant vegetation

Representative Cross-section

SECTION 4 - LEFT BANK SURVEY

PART 8: LEFT BANK CHARACTERISTICS

Type	**Bank Materials**	**Layer Thickness**	**Ave. Bank Height**	**Bank Profile Shape**	**Tension Cracks**
Noncohesive	Silt/clay	Material 1 (m)	Average height (m)	(see sketches in manual)	None
Cohesive	Sand/silt/clay	Material 2 (m)			Occasional
Composite	Sand/silt	Material 3 (m)	**Ave. Bank Slope**		Frequent
Layered	Sand	Material 4 (m)	angle (degrees)		**Crack Depth**
Even Layers	Sand/gravel				Proportion of
Thick+thin layers	Gravel				bank height
Number of layers	Gravel/cobbles	**Distribution and Description of Bank Materials in Bank Profile**			
	Cobbles	**Material Type 1**	**Material Type 2**	**Material Type 3**	**Material Type 4**
Protection Status	Cobbles/boulders	Toe	Toe	Toe	Toe
Unprotected	Boulders/bedrock	Mid-Bank	Mid-Bank	Mid-Bank	Mid-Bank
Hard points		Upper Bank	Upper Bank	Upper Bank	Upper Bank
Toe protection		Whole Bank	Whole Bank	Whole Bank	Whole Bank
Revetments		D50 (mm)	D50 (mm)	D50 (mm)	D50 (mm)
Dyke Fields		sorting coefficient	sorting coefficient	sorting coefficient	sorting coef.

Notes and Comments:-

PART 9: LEFT BANK-FACE VEGETATION

Vegetation	**Tree Types**	**Density + Spacing**	**Location**	**Health**	**Height**
None/fallow	None	None	Whole bank	Healthy	Short
Artificially cleared	Deciduous	Sparse/clumps	Upper bank	Fair	Medium
Grass and flora	Coniferous	dense/clumps	Mid-bank	Poor	Tall
Reeds and sedges	Mixed	Sparce/continuous	Lower bank	Dead	Height (m)
Shrubs	**Tree species**	Dense/continuous			
Saplings	(if known)	**Roots**	**Diversity**	**Age**	**Lateral Extent**
Trees		Normal	Mono-stand	Imature	Wide belt
Orientation		Exposed	Mixed stand	Mature	Narrow belt
Angle of leaning (o)		Adventitious	Climax-vegetation	Old	Single row

Notes and Comments:-

Bank Profile Sketches

Profile Symbols

Bank Top Edge	Failed debris	Engineered Structure
Bank Toe	Attached bar	Significant vegetation
Water's Edge	Undercutting	Vegetation Limit

PART 10: LEFT BANK EROSION

Interpretative Observations

Erosion Location		Present Status		*Severity of Erosion*		*Processes*		*Distribution of Each Process on Bank*			
								Process 1		***Process 2***	
General		Intact		*Insignificant*		*Parallel flow*		*Toe (undercut)*		*Toe (undercut)*	
Outside Meander		Eroding:dormant		*Mild*		*Impinging flow*		*Lower bank*		*Lower bank*	
Inside Meander		Eroding:active		*Significant*		*Piping*		*Upper bank*		*Upper bank*	
Opposite a bar		Advancing:dormant		*Serious*		*Freeze/thaw*		*Whole bank*		*Whole bank*	
Behind a bar		Advancing:active		*Catastrophic*		*Sheet erosion*		***Process 3***		***Process 4***	
Opposite a structure						*Rilling + gullying*		*Toe (undercut)*		*Toe (undercut)*	
Adjacent to structure		**Rate of Retreat**		***Extent of Erosion***		*Wind waves*		*Lower bank*		*Lower bank*	
Dstream of structure		m/yr (if applicable		*None*		*Vessel Forces*		*Upper bank*		*Upper bank*	
Ustream of structure		and known)		*Local*		*Ice rafting*		*Whole bank*		*Whole bank*	
Other (write in)		**Rate of Advance**		*General*		*Other (write in)*					
		m/yr (if applicable		*Reach Scale*							
		and known)		*System Wide*							

Level of Confidence in answers (Circle one)

0 10 20 30 40 50 60 70 80 90 100 %

Notes and Comments:-

PART 11: LEFT BANK GEOTECH FAILURES

Interpretative Observations

Failure Location		Present Status		*Instability:Severity*		*Failure Mode*		*Distribution of Each Mode on Bank*			
								Mode 1		***Mode 2***	
General		Stable		*Insignificant*		*Soil/rock fall*		*Toe*		*Toe*	
Outside Meander		Unreliable		*Mild*		*Shallow slide*		*Lower bank*		*Lower bank*	
Inside Meander		Unstable:dormant		*Significant*		*Rotational slip*		*Upper bank*		*Upper bank*	
Opposite a bar		Unstable:active		*Serious*		*Slab-type block*		*Whole bank*		*Whole bank*	
Behind a bar				*Catastrophic*		*Cantilever failure*		***Mode 3***		***Mode 4***	
Opposite a structure		**Failure Scars+Blocks**				*Pop-out failure*		*Toe*		*Toe*	
Adjacent to structure		None		***Instability: Extent***		*Piping failure*		*Lower bank*		*Lower bank*	
Dstream of structure		Old		*None*		*Dry granular flow*		*Upper bank*		*Upper bank*	
Ustream of structure		Recent		*Local*		*Wet earth flow*		*Whole bank*		*Whole bank*	
Other (write in)		Fresh		*General*		*Other (write in)*					
		Contemporary		*Reach Scale*							
				System Wide							

Level of Confidence in answers (Circle one)

0 10 20 30 40 50 60 70 80 90 100 %

Notes and Copmments:-

PART 12: LEFT BANK TOE SEDIMENT ACCUMULATION

Interpretative Observations

Stored Bank Debris		Vegetation		Age		Health		*Toe Bank Profile*		*Sediment Balance*	
None		None/fallow		Immature		Healthy		*Planar*		*Accumulating*	
Individual grains		Artificially cleared		Mature		Unhealthy		*Concave upward*		*Steady State*	
Aggregates+crumbs		Grass and flora		Old		Dead		*Convex upward*		*Undercutting*	
Root-bound clumps		Reeds and sedges		Age in Years				***Present Debris Storage***		*Unknown*	
Small soil blocks		Shrubs				**Roots**		*No bank debris*			
Medium soil blocks		Saplings		**Tree species**		Normal		*Little bank debris*			
Large soil blocks		Trees		(if known)		Adventitious		*Some bank debris*			
Cobbles/boulders						Exposed		*Lots of bank debris*			
Boulders											

Level of Confidence in answers (Circle one)

0 10 20 30 40 50 60 70 80 90 100 %

Notes and Comments:-

SECTION 5 - RIGHT BANK SURVEY

PART 13: RIGHT BANK CHARACTERISTICS

Type
- Noncohesive
- Cohesive
- Composite
- Layered
- Even Layers
- Thick+thin layers
- Number of layers

Bank Materials
- Silt/clay
- Sand/silt/clay
- Sand/silt
- Sand
- Sand/gravel
- Gravel
- Gravel/cobbles
- Cobbles
- Cobbles/boulders
- Boulders/bedrock

Layer Thickness
- Material 1 (m)
- Material 2 (m)
- Material 3 (m)
- Material 4 (m)

Ave. Bank Height
- Average height (m)

Ave. Bank Slope
- Average angle (o)

Bank Profile Shape
(see sketches in manual)

Tension Cracks
- None
- Occasional
- Frequent

Crack Depth
Proportion of bank height

Protection Status
- Unprotected
- Hard points
- Toe protection
- Revetments
- Dyke Fields

Distribution and Description of Bank Materials in Bank Profile

	Material Type 1	Material Type 2	Material Type 3	Material Type 4
Toe				
Mid-Bank				
Upper Bank				
Whole Bank				
D50 (mm)				
sorting coefficient				

Notes and Comments:-

PART 14: RIGHT BANK-FACE VEGETATION

Vegetation
- None/fallow
- Artificially cleared
- Grass and flora
- Reeds and sedges
- Shrubs
- Saplings
- Trees

Orientation
Angle of leaning (o)

Tree Types
- None
- Deciduous
- Coniferous
- Mixed

Tree species
(if known)

Density + Spacing
- None
- Sparse/clumps
- dense/clumps
- Sparce/continuous
- Dense/continuous

Roots
- Normal
- Exposed
- Adventitious

Location
- Whole bank
- Upper bank
- Mid-bank
- Lower bank

Diversity
- Mono-stand
- Mixed stand
- Climax-vegetation

Health
- Healthy
- Fair
- Poor
- Dead

Age
- Imature
- Mature
- Old

Height
- Short
- Medium
- Tall
- Height (m)

Lateral Extent
- Wide belt
- Narrow belt
- Single row

Notes and Comments:-

Bank Profile Sketches

Profile Symbols

Bank Top Edge	Failed debris	Engineered Structure
Bank Toe	Attached bar	Significant vegetation
Water's Edge	Undercutting	Vegetation Limit

PART 15: RIGHT BANK EROSION

Interpretative Observations

Erosion Location	Present Status	*Severity of Erosion*	*Processes*
General	Intact	*Insignificant*	*Parallel flow*
Outside Meander	Eroding:dormant	*Mild*	*Impinging flow*
Inside Meander	Eroding:active	*Significant*	*Piping*
Opposite a bar	Advancing:dormant	*Serious*	*Freeze/thaw*
Behind a bar	Advancing:active	*Catastrophic*	*Sheet erosion*
Opposite a structure			*Rilling + gullying*
Adjacent to structure	**Rate of Retreat**	***Extent of Erosion***	*Wind waves*
Dstream of structure	m/yr (if applicable	*None*	*Vessel Forces*
Ustream of structure	and known)	*Local*	*Ice rafting*
Other (write in)	**Rate of Advance**	*General*	*Other (write in)*
	m/yr (if applicable	*Reach Scale*	
	and known)	*System Wide*	

Distribution of Each Process on Bank

Process 1	*Process 2*
Toe (undercut)	*Toe (undercut)*
Lower bank	*Lower bank*
Upper bank	*Upper bank*
Whole bank	*Whole bank*
Process 3	***Process 4***
Toe (undercut)	*Toe (undercut)*
Lower bank	*Lower bank*
Upper bank	*Upper bank*
Whole bank	*Whole bank*

Level of Confidence in answers (Circle one)

0 10 20 30 40 50 60 70 80 90 100 %

Notes and Comments:-

PART 16: RIGHT BANK GEOTECH FAILURES

Interpretative Observations

Failure Location	Present Status	*Instability:Severity*	*Failure Mode*
General	Stable	*Insignificant*	*Soil/rock fall*
Outside Meander	Unreliable	*Mild*	*Shallow slide*
Inside Meander	Unstable:dormant	*Significant*	*Rotational slip*
Opposite a bar	Unstable:active	*Serious*	*Slab-type block*
Behind a bar		*Catastrophic*	*Cantilever failure*
Opposite a structure	**Failure Scars+Blocks**		*Pop-out failure*
Adjacent to structure	None	***Instability: Extent***	*Piping failure*
Dstream of structure	Old	*None*	*Dry granular flow*
Ustream of structure	Recent	*Local*	*Wet earth flow*
Other (write in)	Fresh	*General*	*Other (write in)*
	Contemporary	*Reach Scale*	
		System Wide	

Distribution of Each Mode on Bank

Mode 1	*Mode 2*
Toe	*Toe*
Lower bank	*Lower bank*
Upper bank	*Upper bank*
Whole bank	*Whole bank*
Mode 3	***Mode 4***
Toe	*Toe*
Lower bank	*Lower bank*
Upper bank	*Upper bank*
Whole bank	*Whole bank*

Level of Confidence in answers (Circle one)

0 10 20 30 40 50 60 70 80 90 100 %

Notes and Copmments:-

PART 17: RIGHT BANK TOE SEDIMENT ACCUMULATION

Interpretative Observations

Stored Bank Debris	Vegetation	Age	Health	*Toe Bank Profile*	*Sediment Balance*
None	None/fallow	Immature	Healthy	*Planar*	*Accumulating*
Individual grains	Artificially cleared	Mature	Unhealthy	*Concave upward*	*Steady State*
Aggregates+crumbs	Grass and flora	Old	Dead	*Convex upward*	*Undercutting*
Root-bound clumps	Reeds and sedges	Age in Years		***Present Debris Storage***	*Unknown*
Small soil blocks	Shrubs		**Roots**	*No bank debris*	
Medium soil blocks	Saplings	**Tree species**	Normal	*Little bank debris*	
Large soil blocks	Trees	(if known)	Adventitious	*Some bank debris*	
Cobbles/boulders			Exposed	*Lots of bank debris*	
Boulders					

Level of Confidence in answers (Circle one)

0 10 20 30 40 50 60 70 80 90 100 %

Notes and Comments:-

Appendix C
Stream Reconnaissance Data Sheets by Baltimore District

Hydraulic Field Data / Project Identification

Stream:	Map:	Reach ID:
Date:	Coordinates of Reach:	
Team:	Drainage Area:	
Landmarks:		

REACH DESCRIPTION / RESTORATION NEED:

Watershed: a) FOREST b) FARM c) MINE d) RURAL RES. e) SUBURBAN f) URBAN

RESTORATION OPTIONS		N/P/H	C. Area	C Access	Description (size)
Project Type:	Bank Protection				
sketch:	Grade Protection				
	Realignment				
	Instream Habitat				
	Low Flow Channel				
	Riparian Buffer				
	Wetland*				
	SWP New/Retrofit*				
	Fish Blockage*				
	Flood Protection*				
	*fill out supplemental sheets				

VISUAL OBSERVATIONS

Infrastructure Damaged: a)YES b)NO DESCRIPTION:

Infrastructure Threatened: a)YES b)NO DESCRIPTION:

Condition of Reach (1-10(best)) **Suitability as a Reference:**

Current Conditions of Channel Bed : a)STABLE b)AGGRADED NOW STABLE c)DEGRADED NOW STABLE d)DEGRADED NOW AGGRADING* e)AGGRADING* f)DEGRADING*

*Extent of Bed Change: a)WIDESPREAD IN REACH b)LOCAL

*Rate of Change in Channel Bed: a)LOW b)MODERATE c)HIGH

Bedload: a)NONE-LOW b)MEDIUM c)HIGH Description:

Plan Form Control Present: a)YES b)NO DESCRIPTION:

Grade Control Present: a)YES b)NO DESCRIPTION:

Debris Jam Present: a)YES b)NO DESCRIPTION:

Gravel Deposition Present: a)YES b)NO DESCRIPTION:

Incision: a)Sig. overbank flow b)Possible/limited overbank flow c)No access EVIDENCE:

Current Conditions of Channel Bank: a)STABLE b)ERODED NOW STABLE c)CURRENTLY ERODING*

sketch section on back:

*Location: a) LEFT BANK b)RIGHT BANK c)LOCAL LEFT d)LOCAL RIGHT

*Rate of Change in Channel Bank: a)LOW b)MODERATE c)HIGH

*Erosion Cause: a)HYDRAULIC b)GEOTECH. c)OTHER:

Left Bank Cover (mark >1/4): a)EARTH b)ROCK c)VEGETATED d)CONC. e)RIPRAP d)OTHER:

Right Bank Cover (mark>1/4): a)EARTH b)ROCK c)VEGETATED d)CONC. e)RIPRAP d)OTHER:

Left Bank Erodability: a)LOW b)MEDIUM c)HIGH d)low-med e)low-high f)med-high

Right Bank Erodability: a)LOW b)MEDIUM c)HIGH d)low-med e)low-high f)med-high

Left Bank Riparian Buffer: a)NONE b)V. Low(1-10') c)Low(10-50') d)Med.(50-100') e)H(>100')

Right Bank Riparian Buffer: a)NONE b)V. Low(1-10') c)Low(10-50') d)Med.(50-100') e)H(>100')

Typical Riffle/Run Cross Section	**CHANNEL MATERIAL**				
Top Bank Width:	mark range of material	Riffles	Pools	Banks	Bar
Top Bank Height: L R	Bedrock				
Wet Width:	Boulder (10-160 in)				
Water Depth:	Cobble (2.5-10 in)				
Variability in Reach a)Sig. b)Some	Gravel (.08-2.5 in)				
Bed Slope:	Sand (.0625-2 mm)				
No. Pools/riffles	Silt (0.004-0.0625 mm)				
	Clay				

Photo Roll No. Photo No.	SKETCHES ON BACK (as appropriate)

Hydraulic Field Data - Flood Damage Sites

Stream:	Map:	Location ID:
Date:	Coordinates of Area:	
Team:	Drainage Area:	
Landmarks:		

OBSERVATIONS	
Description of Site and Proposed Project Description:	
Estimate Frequency of Flooding:	a) Annual b) Frequent (1-10 yr) c) Infrequent (>10 yr) d) Catastrophic
Describe Last Flooding Incident and Source of Information:	
Types and Number of Structures Threatened:	
	Homes:
	Businesses:
	Public Buildings:
	Other:
Flood Damage Because:	a) Construction in Floodplain b) Backwater from Road Crossing
	c) Backwater from Channel Constriction Type:
	d) Inadequate Storm Drainage e) New Construction f) Ice Jam g) Deposition in Channel
	h) Other:
Estimate Height of Lowest Damage Location from Channel Invert:	
Estimate Height of Highest Damage Location from Channel Invert:	
Describe Existing Flood Protection (as applicable):	
Proposed Fix:	a) SWM Pond (fill out supplemental sheet) b) Storm Drainage c) Levee d) Floodwall
	e) Ringwall f) Flood Proofing g) Flood Warning h) Increase Channel Conveyance
	i) Buyout j) Other:
Construction Access:	a) Limited b) Moderate c) Accessible Description:
Presence of Utilities:	a) Present b) No Evidence Description:
Existing Wetlands Adjacent to Area:	a) Yes b) No Description:
***Fill out as appropriate:**	
Estimate Levee/Floodwall Length:	
Estimate Levee/Floodwall Height:	
Describe Channel Conveyance Excavation:	
Describe Past Dredging:	
Describe Road Abutments:	
Road Crossing Type:	a) Bridge b) Bottomless Arch c) Pipe Arch d) Box Culvert e) Pipe f) Ford g) Other:
Describe Opening (no. cells, size etc.):	
Describe Modification to Road Crossing:	

Sketch Plan of Existing/Proposed Site	**Sketch Section of Existing/Proposed Site**
Photo Roll No. Photo No.	SKETCHES ON BACK (as appropriate)

Hydraulic Field Data - Storm Water Ponds/Wetlands

Stream:	Map:	Location ID:
Date:	Coordinates of Area:	
Team:	Drainage Area:	
Landmarks:		

OBSERVATIONS	
Description of Site and Proposed Project Description:	
Project Type:	a)POND RETROFIT* b)OFF-LINE WETLAND** c)ONLINE WETLAND** d)NEW POND**
Construction Access:	a) LIMITED b)MODERATE c)ACCESSIBLE DESCRIPTION:
Presence of Utilities:	a)PRESENT b)NO EVIDENCE DESCRIPTION:
Present Land Use:	a) FOREST b) FARM-CROPS c) PASTURE d) FARM-ABANDONED e) MINED-ABANDONED
	f) URBAN g) RESIDENTIAL h) OTHER:
Adjacent Property:	
Existing Wetlands Adjacent to Area:	a) YES b) NO DESCRIPTION:
Side Slope of Site:	a)VERY STEEP b)STEEP c)MODERATE ESTIMATE:
Input Sources:	
Receiving Stream Description:	
***Type of Existing Pond:**	a) SWM b)RECREATION c)WILDLIFE d)IRRIGATION e)LIVESTOCK
	f) MINE g)UNKNOWN h)OTHER
***Condition of Existing Pond:**	a) GOOD b)MODERATE c)POOR Evidence:
***Maintenance Required:**	
*** Freeboard:**	
***Sign of Entophication (green water):**	a) YES b)NO
***Expand Site Horizontally:**	a)POSSIBLE b)NO DESCRIPTION:
***Excavate Bottom of Site:**	a)POSSIBLE b)NO DESCRIPTION:
***Raise Embankment**	a)POSSIBLE b)NO DESCRIPTION:
***Replace Emergency Spillway:**	a)POSSIBLE b)NO DESCRIPTION:
***Riser Description:**	
***Existing Control:**	
***Existing Discharge Barrels:**	
***Existing Outfall:**	
***Existing Embankment Height:**	
****Excavation Required:**	a) HORIZONTAL b)INVERT DESCRIPTION:
****Existing Slope in Site:**	
****Approx. Required Embankment Size:**	
****Estimate Height of Site Above Adjacent Stream:**	
****Evidence of High Groundwater:**	a)YES b)POSSIBLE c)NO DESCRIPTION:
****Proposed Control:**	
****Approx. Area (Ac.):**	****Approx. Volume (Ac-Ft):**

Sketch Plan of Existing/Proposed Site	**Sketch Section of Existing/Proposed Site**
Photo Roll No. Photo No.	SKETCHES ON BACK (as appropriate)

Hydraulic Field Data - Fish Blockage

Stream:	Map:	Location ID:
Date:	Coordinates of Barrier:	
Team:	Drainage Area:	
Landmarks:		

OBSERVATIONS	
Description of Site and Proposed Project Description:	
Type of Barrier:	a) Dam* b) Road Crossing* c) Pipe Crossing* d) Beaver Dam e) Natural Falls/Rapids f) Other:
Proposed Fix:	a) Removal b) Fish Latter c) Modification Describe:
Permanence:	a) Temporary b) Permanent c) Unknown
Condition:	a) Total Blockage b) Partial Blockage
Blockage Because:	a) Water Too Fast b) Water Too Shallow c) Water Drop d) Other
	Water Vel. Estimate:
	Water Depth (at blockage):
	Water Drop:
Fish Passage Device Exists?	a) Yes b) No
	If Yes, why is it not functioning:
Construction Access:	a) LIMITED b)MODERATE c)ACCESSIBLE DESCRIPTION:
Presence of Utilities:	a)PRESENT b)NO EVIDENCE DESCRIPTION:
Adjacent Property:	
***Condition of Structure:**	a) GOOD b)MODERATE c)POOR Evidence:
***Maintenance Required:**	
***Estimate Age of Structure:**	Evidence:
***Ownership:**	a) Federal b) State c) Local d) Private e) Unknown Evidence:
*** Material**	a) Stone b) Earth c) Concrete d) Wood e) Metal f) Plastic g) Other:
***Fill out as appropriate:**	
Purpose of Existing Dam:	a) SWM b)RECREATION c)WILDLIFE d)IRRIGATION e)LIVESTOCK
	f) MINE g)UNKNOWN h)OTHER:
Existing Control:	
Freeboard:	
Existing Outfall:	
Existing Embankment Height:	
Road Crossing Type:	a) Bridge b) Bottomless Arch c) Pipe Arch d) Box Culvert e) Pipe f) Ford g) Other:
Describe Opening (no. cells, size etc.):	
Purpose of Pipe:	a) Sanitary Sewer b) Storm Sewer c) Water Supply d) Unknown e) Other:
Evidence of Pipe Leakage:	a) Yes b) No Evidence:
Pipe Undermined	a) Yes b) No Description:

Sketch Plan of Existing/Proposed Site	**Sketch Section of Existing/Proposed Site**
Photo Roll No. Photo No.	SKETCHES ON BACK (as appropriate)

Appendix D Guidelines for Sampling Bed Material

Purpose of Bed Material Sampling

Knowledge of streambed characteristics is necessary for a variety of engineering and environmental purposes related to stream restoration projects. Bed material sampling programs must be carefully designed to meet the particular needs of a specific study. Stream restoration studies may include objectives related to: the source, transport and fate of pollutants; fish habitat; resource management; morphological trends and/or river engineering works. Contaminates typically attach to cohesive sediment and therefore are distributed over a wide area, especially in areas where flow velocity is low. Sampling for a contaminate study should concentrate on depositional zones in the stream and overbank. Fish habitat studies may be concerned with the suitability of the streambed for spawning. Sampling for this type of study should be relatively extensive, identifying lateral, longitudinal, and temporal variations in the surface layer over a wide area of the stream. Resource management studies are frequently concerned with the need or feasibility of sand and gravel mining. Core or substrate sampling that identifies vertical variation of the streambed is essential for this type study. Morphologic and engineering studies are typically concerned with changes in the character of the river over time. These studies frequently require knowledge of the grain size distribution of both the bed surface material and subsurface material for sediment transport calculations, critical shear stress determinations, determining potential for particle sorting and armoring, and for determining hydraulic roughness.

Bed material sampling is frequently conducted in order to make sediment transport calculations. For this purpose the sampling program should identify a "representative" bed material gradation, but it is also necessary to identify any lateral, longitudinal, vertical, and/or temporal variation in bed material composition. Lateral variations in bed material gradation can be much more significant than longitudinal variations. In sand bed streams the sample is typically taken from the upper five centimeters of the bed surface. In gravel bed streams with coarse surface layers, samples of both the surface and subsurface layers are required. Ideally, bed material samples should be taken at different times during the year to account for seasonal variations.

Table D-1 provides guidance relative to where a bed-material sample might be taken as a function of the type of geomorphologic or engineering analysis to be conducted.

Table D1
Bed Material Sampling Sites

Purpose of analysis	Sample location
To estimate the maximum permissible velocity in a threshold stream	Riffle
To estimate the minimum permissible velocity in a threshold stream	Areas of local deposition
To estimate sediment yield for an alluvial stream	Crossing or middle bar
To quantify general physical habitat substrate condition	Bars, riffles, and pools

Bed Material Characteristics

Deposited sediment is sampled to provide information on the individual sediment particles, the sediment mixture, and the bulk sediment deposit. Particle characteristics include grain size, shape, specific gravity, lithology, and mineralogy. The quantity and type of contaminates attached to particles are frequently of interest. Data that describes the distribution of the various particle sizes and of specific contaminates are frequently required. Characteristics of the sediment deposit itself include: stratigraphy, density, and compaction. For some of these purposes a sample can be disturbed, others require undisturbed sampling. Different samplers and sampling procedures are appropriate for different environments. Water depth and velocity and bed material size are the most important factors used to identify appropriate samplers and sampling procedures.

When the sediment particles are noncohesive, mechanical forces dominate the behavior of the sediment in water. The three most important properties that govern the hydrodynamics of noncohesive sediments are particle size, shape, and specific gravity. A discussion of these properties is found in “Sedimentation Investigations in Rivers and Reservoirs,” EM 1110-2-4000.

The boundary between cohesive and noncohesive sediments is not clearly defined. It can be stated, however, that cohesion increases with decreasing particle size for the same type of material. Clays are much more cohesive than silts. Electrochemical forces dominate cohesive sediment behavior. The three most common minerals that have electrochemical forces causing individual particles to stick together are illite, kaolinate, and montmorillonite. The dispersed particle fall velocity, flocculated fall velocity of the suspension, clay and nonclay mineralogy, organic content, and cation exchange capacity characterize cohesive sediment. The fluid is characterized by the concentration of important cations, anions, salt, pH, and temperature. More detailed information is presented in “Tidal Hydraulics,” EM 1110-2-1607.

Sampling Procedures

Several factors influence both sampling site selection and sampling procedure. The most significant factor is the data necessary to meet the objectives of the study at hand. The objective of a bed material sampling program may be to determine a representative bed gradation for a particular reach of a stream, or it may be to determine the variability and diversity of the sediment bed. Data needs should be clearly defined before the sampling program is planned. The second factor to consider is field conditions. Will the bed of the stream be wet or dry? Is the site accessible by road, boat, trail, or only by helicopter? Field conditions will determine both the practicality and type of sampling equipment to be used in the sampling program. Another factor that influences the type of sampling equipment and the appropriate sampling procedure is the character of the streambed itself. Sand bed streams typically have a more uniform bed gradation and therefore require a smaller volume sample than gravel bed streams. Typically, equipment appropriate for sampling sand bed streams is inappropriate for gravel bed streams. Thus, it is necessary to know the general streambed characteristics before the sampling program is established. Finally, available resources must be considered as a limiting factor when establishing a bed-sampling program. Equipment, manpower, and funds are frequently limited and therefore priorities must be established.

It is helpful if the bed material sampling location is near a stream gaging station in order to relate sediment data to measured hydrologic and hydraulic data.

Site Selection for Representative Sampling

There is no simple rule for locating representative sampling sites or reaches. The general rule is as follows: *Carefully select sampling locations and avoid anomalies that would bias either the calculated sediment discharge or the calculated bed stability.* The location must be representative of the hydraulic and sedimentation processes that occur in that reach of the river. The site should be morphologically stable (constant slope and width upstream and downstream). To ensure data reflects reach-averaged river conditions there should be no tributary inflow in the proximity of the site as it may interfere with the homogeneity of the section by supplying sediment for deposition. The site should not be located adjacent to a zone of active bank erosion as the material deposited in the channel near the area may not be representative of the reach. Although bridges provide good access, bridge crossings are typically not appropriate sampling sites because they are located at natural river constrictions or their abutments and piers create constrictions and local scour. Dead water areas behind sand bars or other obstructions should be avoided, as these are not representative of average flow conditions.

Sand Bed Streams

Sand bed streams are characterized by a relatively homogeneous bed material gradation. Vertical and temporal variability is typically insignificant in stable streams. Longitudinal variability typically occurs over distances of many kilometers. However, lateral variability, especially in bends, can be significant. In sand bed rivers, sampling of material is most frequently carried out in the low flow channel. The equipment and methodology depends on the river depth and velocity. The task can be accomplished in flowing streams either by wading or from a boat, or in ephemeral and intermittent streams in the dry. Vertical variations in the bed material are usually insignificant in flowing water and samples are collected from the surface. However, in standing water or on dry beds, a layer of fine material deposited on the recession of a flood hydrograph is sometimes found on the surface. It is standard practice to remove this fine surface layer before collecting a sample from this kind of area.

Einstein (1950) recommended using only the coarsest 90 percent of the sampled bed gradation for computations of bed material load. He reasoned that the finest 10 percent of sediment on the bed was either material trapped in the interstices of the deposit or a lag deposit from the recession of the hydrograph and should not be included in bed material load computations.

Representative bed material sampling in sand bed streams may be accomplished by one of two methods. Employing the cross-section approach requires selecting a site and time for sampling where and when the bed characteristics are typical. This method requires considerable experience, and unanimity of opinion about where and when the typical condition occurs cannot be expected even among experienced river scientists. Frequently, judgement is influenced by the type of streams the sampler has experienced and by the intended use of the data. Employing the reach approach where samples from several systematically selected cross sections are averaged to obtain a representative sample may eliminate some uncertainty associated with the cross-section approach.

Cross-section approach. This approach requires the selection of a representative cross section. In streams with relatively uniform depths, between five and three samples should be taken across the section to account for lateral variations. In streams with variable depths more samples are required. Twenty verticals are commonly taken in braided streams. Taking bed material samples at crossings where flow distribution is typically more uniform reduces the lateral variation in the samples. However, at low flow, crossings may develop a surface layer gradation that reflects sediment transport conditions at the lower discharge, which may be coarser or finer then the bed gradation at bankfull discharge. Also, crossings are typically submerged and more elaborate sampling equipment is required than at exposed bars where a shovel is frequently sufficient. However, samples collected on a point or alternate bar may exhibit considerable variation. Figure D-1 illustrates typical bed material gradation patterns on a point bar. Note that although the typical grain sizes found on the bar surface form a pattern from coarse to fine, there is no one location which always captures the precise distribution that will represent the entire range of sedimentation processes.

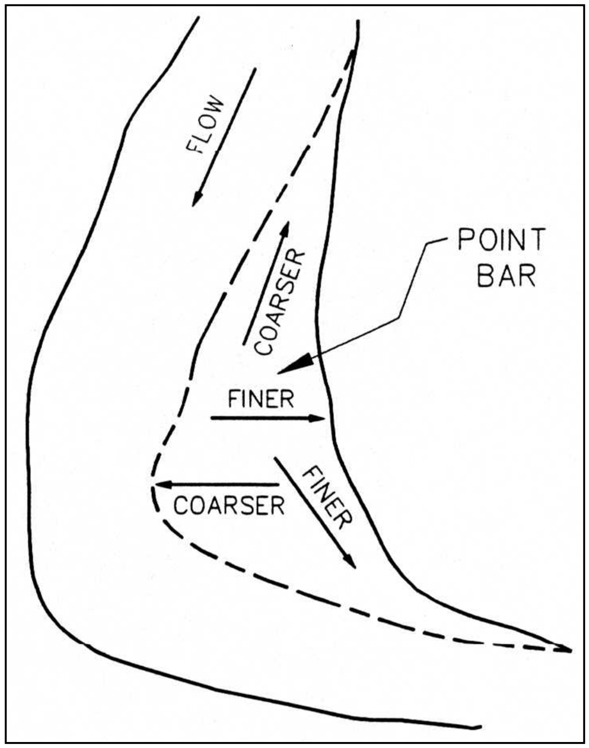

Figure D1. Gradation pattern on a point bar

Reach approach. An alternative to the cross-section approach is the reach approach.[1] A reach is defined as a portion of the stream with similar morphology (identified by its homogeneity). Generally, five cross sections are laid out in the homogeneous reach. If there is a gage in the reach, locating the center cross section near the gage is preferred. If the stream reach is straight, the spacing should be approximately two to five stream widths, and if the reach is meandering, the spacing should occur within one meander length as shown in Figure D-2. The same criteria used in the cross-section approach to determine the number of verticals are applied here. The reach approach is especially applicable to rivers with meanders of different wavelengths and amplitudes.

[1] Zrymiak, P. (1997). "Field procedures for sediment date collection: Vol. 2, bed material," (draft) Environment Canada, Ottawa, Ontario.

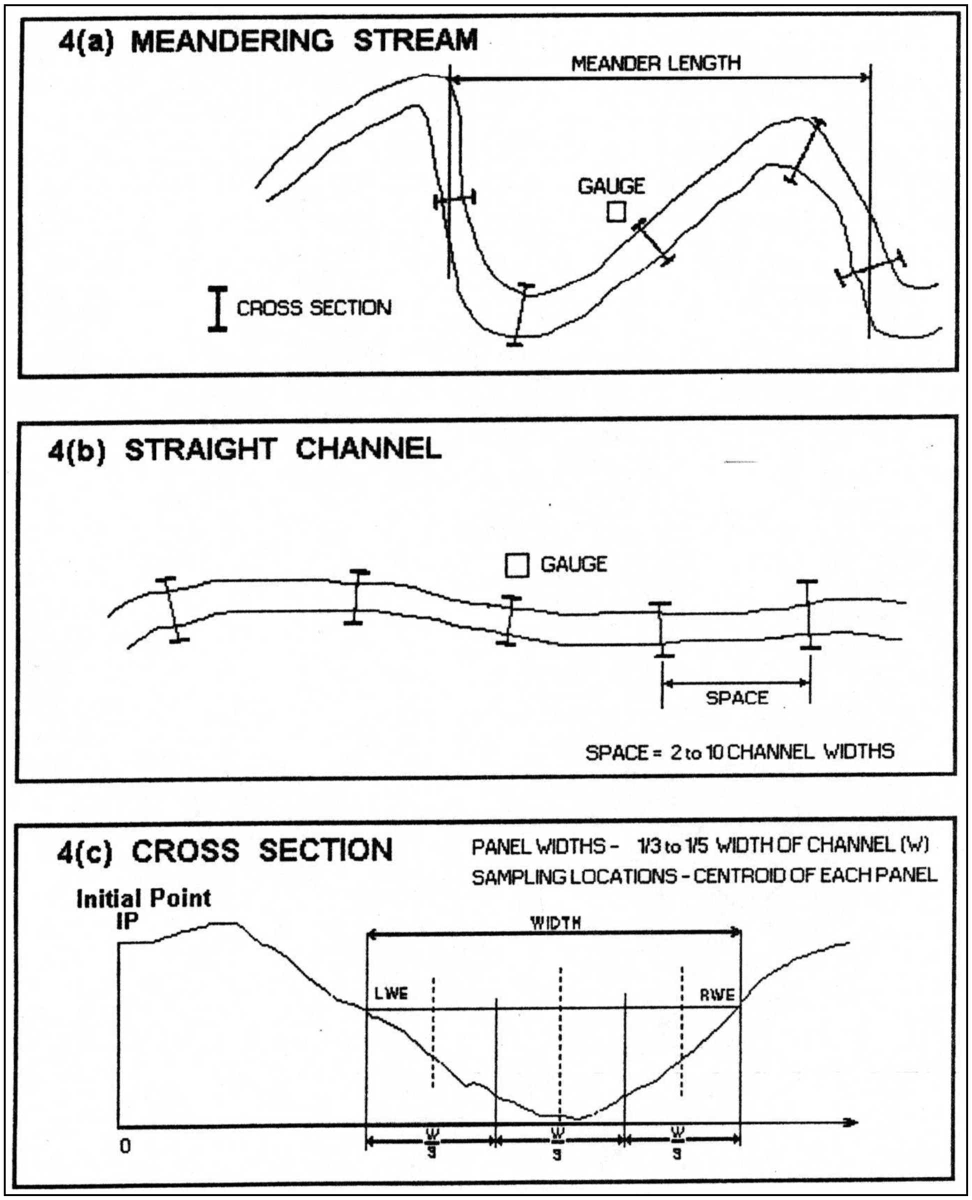

Figure D2. Bed sampling locations for sand bed streams (Zrymiak 1997, op. cit.)

Gravel Bed Streams

Coarse beds (gravel, cobble, and boulder) are characterized by significant vertical, spatial, and temporal bed material variability. The most distinctive characteristic is a coarse surface layer that may form in both the low flow channel and on bars. Frequently the low flow channels of coarse bed streams are armored with large cobbles and boulders while bars consist primarily of sand and gravel.

Since the spatial variability in most coarse bed streams is high, it is very difficult to perform representative sampling. River bars are frequently chosen as sampling sites and specific bar types have been determined to be more representative than others. A bar type hierarchy established to aid site selection (Bray 1972; Yuzyk 1986) is shown in Figure D-3. Midchannel and diagonal bars are selected as most ideal sites because they are exposed to the highest velocities, which transport the largest materials. Point bars are not as ideal because velocities are highly variable, decreasing toward the inside bank. Channel side or lateral bars are least desirable because they exist in zones of low velocities due to boundary and bank effects. In small streams with no bars and a pool-riffle sequence the riffles may be sampled to characterize bed-material size. However, the riffle can be expected to be much coarser at low flow when sediment transport is typically negligible than at bankfull flow when sediment transport is active.

Based on the assumption that the coarsest materials in the bed exert the predominant effect on channel behavior and flow resistance, some recommend that samples be collected at the upstream end of bars (Bray 1972; Church and Kellerhals 1978; Yuzyk 1986). Sediments at this location are indicative of the sediments in the main channel, readily identifiable and generally exposed. The upstream end of bars usually consists of the coarsest material in the channel and not the average size in the reach because the upstream end of the bars is the location most frequently exposed to the highest stream velocities.

In coarse bed streams it is necessary to determine the characteristics of both the surface and subsurface bed layers. Bulk sampling is employed to characterize the subsurface layer. Both bulk and areal sampling are employed to characterize the surface layer. Bulk surface sampling is preferred if it is possible to identify and collect only the surface layer material. This is difficult when the surface layer has a wide range of size classes. Bulk surface sampling provides information about the finer grain sizes trapped in the interstices of the surface layer, which is useful for permeability studies for fish habitat and for sediment transport studies. Areal surface sampling is used to characterize the coarse surface layer and is used to determine hydraulic roughness, critical shear stresses, armoring, and sediment transport.

A common methodology for areal sampling is a pebble count (Wolman 1954) where individual particles are collected at random by hand and the intermediate axis is measured. This method requires that the stream be wadeable, although divers may be employed. At least 100 particles should be included in the sample. However, to be very precise or to accurately measure small percentiles, the number of sampled particles should be increased. One method for choosing the particles is a random walk laterally across the stream or longitudinally along a point bar; another is to set up a longitudinal or square grid and measure particles at the intersection of grid points. The gradation curve developed from these data is based on the number of particles in each size class not their weights or projected surface areas. Studies have shown that particles smaller than 8 mm are typically missed with areal sampling, especially if the bed surface is submerged, and thus the pebble count may be biased toward the larger sizes. This problem can be overcome by truncating pebble-count samples at 8 mm and using a bulk surface sample to define the percentage and distribution of material finer than 8 mm.

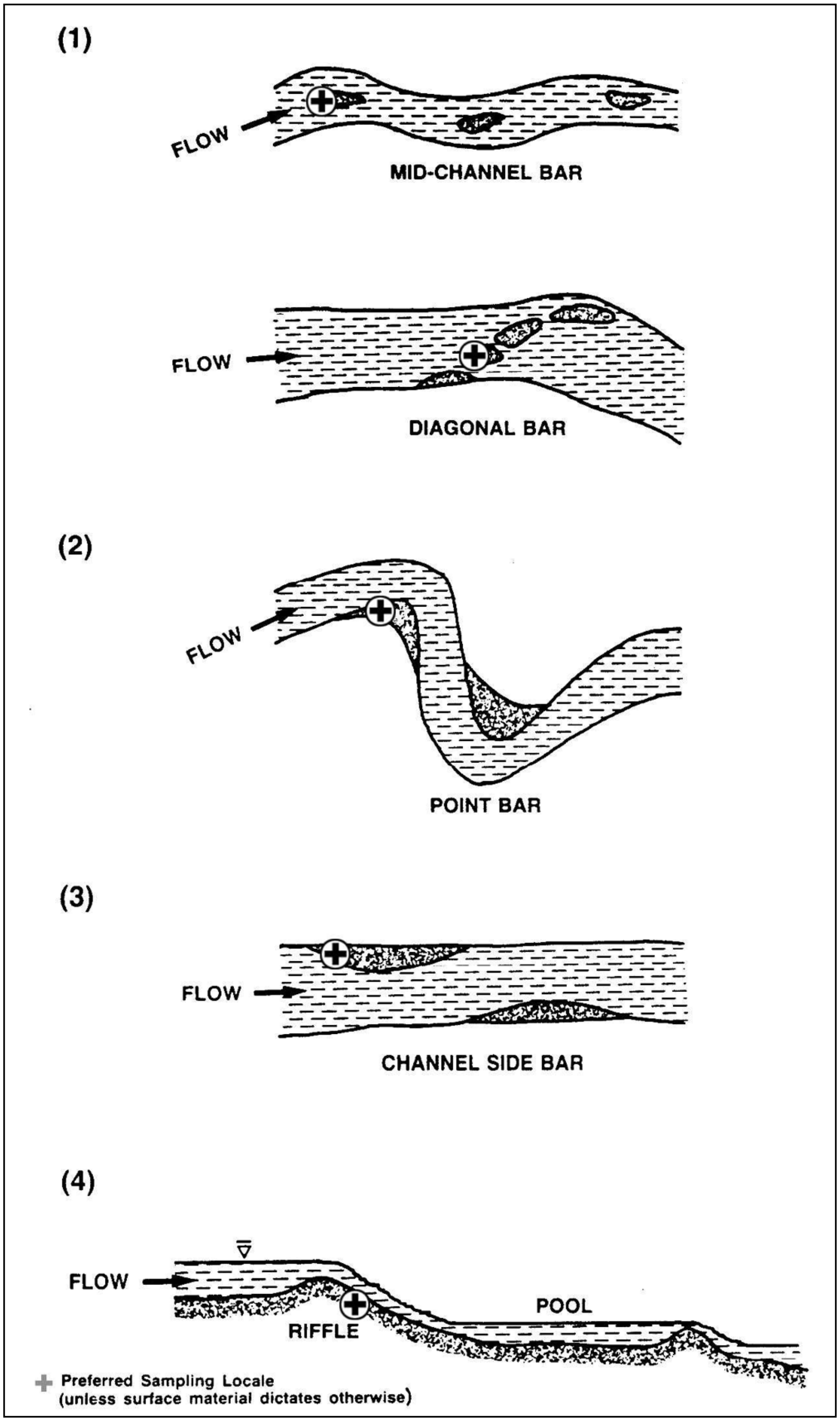

Figure D3. Coarse bed stream sampling hierarchy (Bray 1972)

In addition to determining bed-material gradations, it is often important to determine the characteristics of the stream bank. The bank material can help define the stability of the channel section and may be responsible for a significant percentage of the total sediment load. It is advisable to assess the soil type and gradation of each of the bank strata. Therefore several bulk samples should be taken at a given location. When sampling the bank, it is also advisable to assess such features as layering and lensing and to look for evidence of piping and seepage and related features.

Step-by-Step Field Sampling Procedures

Maintain detailed records of all data collected

Step 1. Select and mark out the required cross sections and the sampling locations. Use as many of the site selection criteria previously outlined as possible. The fixed permanent initial point should be on the left bank (looking downstream). Establish the control (horizontal and vertical) and reference all points.

Step 2 Sketch the site on data forms and reference the control points. If the streambed contains a mixture of sand and gravel deposits then map areas and record deposits of different size material. Develop a sampling strategy that will sample each zone.

Step 3 Collect a photographic record of the reach, controls, cross sections, sample locations (if possible), bed material (use a scale for reference) and bank conditions.

Step 4 Select appropriate sampler for the task (based on depth, velocity, and sample requirements). Verify that the sampler is operational.

Step 5 Surface bulk sample - sand bed. Move to a sampling location. In shallow streams use a tape to measure from the permanently fixed initial point (IP), and wade to a sampling vertical on the section. Approach the sampling verticals from the downstream side to prevent disturbing the bed at the sampling section. In deep streams using a boat and some type of positioning system (tag line in narrow streams, electronic distance measurement (EDM) in wide streams), hold the boat steady over the sampling location. Obtain a sample of about 250 g at each chosen location using the selected sampler.

Surface areal sample - coarse bed. To obtain a surface areal sample in a coarse bed stream, several techniques are employed. These include random walks, setting up square or linear grids, and removing all the surface particles within a specified area. The spacing of the sampling points must be at least two times the diameter of the largest particle in the sampling area. This reduces the influence of nearby particles. Use 100 sample points (Wolman 1954; Hey and Thorne 1983; and Yuzyk 1986) for ease of data reduction. The random walk method devised by Wolman (1954) can easily be employed on a dry bed or in wadeable flow, and with more difficultly by divers. To obtain a sample, a team member paces along a selected path, collecting a pebble with each step. With closed or averted eyes the first pebble touched is selected. This method generally produces a sample biased toward coarse size classes. Other forms of grid sampling include laying out a linear tape and selecting the pebble at a designated interval and laying out a preconstructed rectangular grid and selecting the pebble at grid point intersections. Collecting the entire surface layer within a specified area generally requires a specialized sampler. The process may be aided by spray painting the surface if the bed is dry, although this technique is rather tedious. Regardless of the approach chosen, the measuring process may be streamlined in the field by using a gravelometer to measure the sieve diameter of each particle immediately after the particle is selected.

Surface bulk sample - coarse bed. To obtain a surface bulk sample, carefully remove and collect all sediment in the surface layer to a thickness of the intermediate axis of the largest particle in the area. Care should be taken to insure that fine sediment is not washed out of the sample. The required sample mass is a function of the largest particle on the surface and can be determined from Figure D-4.

Subsurface bulk sample - coarse bed. If the surface layer has not already been removed then scrape away the surface layer of coarse material to the thickness of the intermediate axis of the largest particle in the area. The required sample mass is a function of the largest particle in the subsurface and can be determined from Figure D-4.

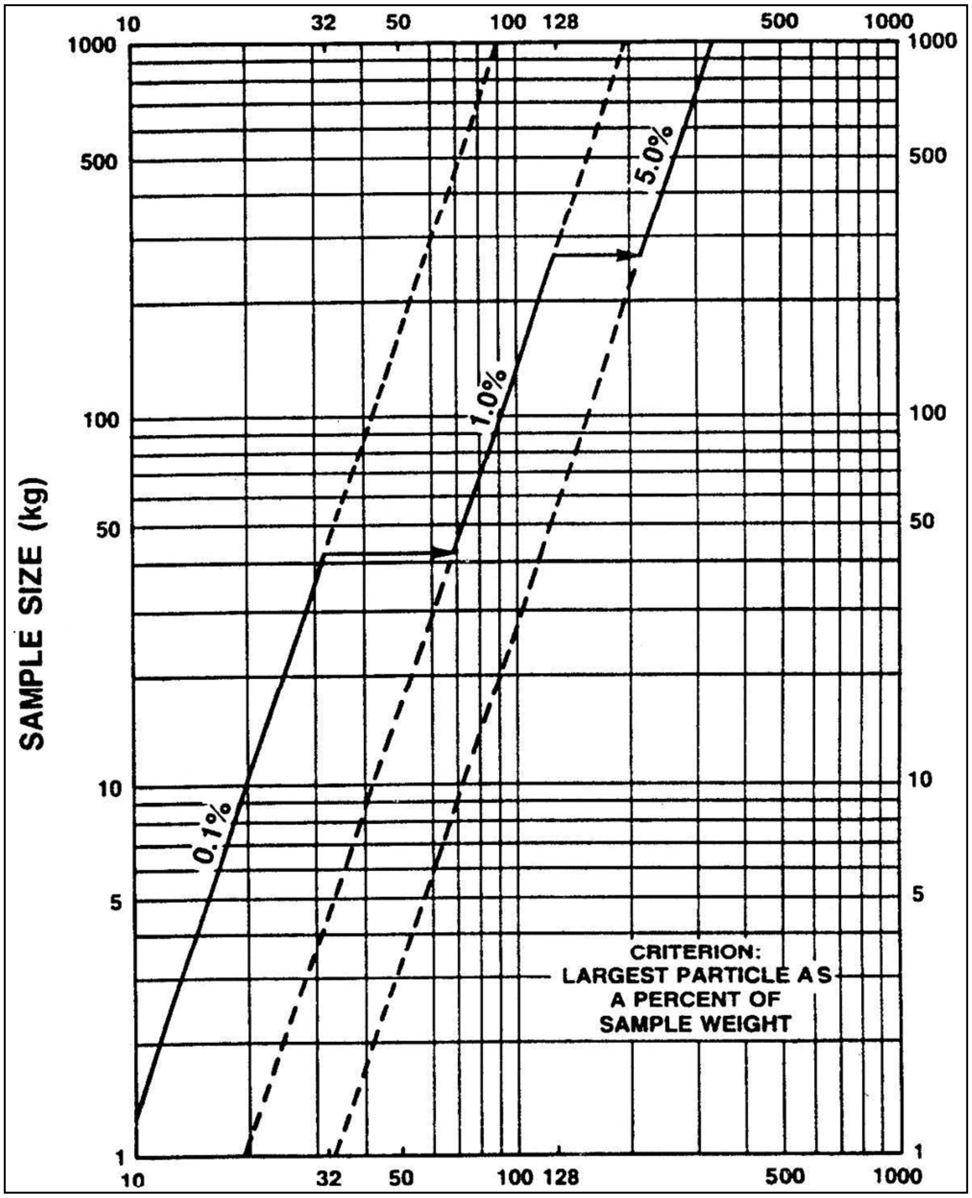

Figure D4. Bulk sampling standards for gravel and cobble streams (Yuzyk 1986; Church, McLean, and Wolcott 1987)

Step 6 Field sieving. This step is an alternative to transporting large bulk samples to a laboratory. Set up a weighing station. This may consist of a tripod with a scale suspended for weighing pails of material. Assemble field sieve sets and insert correct sieves. Collect pails, spades, template, labels, field note forms, sturdy plastic bags, and tarpaulins. Spread out two tarpaulins. Obtain tare weights for the pails. Shovel subsurface material into pails, weigh and record. Pour material into top of the field sieves (8-, 16-, 32-, 64-, 128-mm sieves). Rock and shake the sieve set until material has moved to its retained size sieve. Weigh material retained on each sieve and on the pan. Record in field notes.

Save the material passing the finest sieve size for laboratory analysis. Save the 10 largest particles. Repeat the process until the required mass has been sieved. Measure the three perpendicular axes of the 10 largest particles. Retain up to 10 kg of the combined material from the pan and discard the rest of the sample.

Step 7 Transfer the sample to a clean heavy-gage plastic bag.

Step 8 Complete and attach a label and sediment field note form for each sample. Specify the stream, station, cross section, vertical location, date, time, bed form and flow conditions, personnel on crew, type of sampler, sample number, and sample depth.

Appendix E
Computation of Average Annual Sediment Yield Using Weighted Events

Average annual sediment yield can be approximated using calculated flood hydrographs and an adjustment factor. This calculation requires the following data:

a. discharge hydrographs for the project reach for a range of flood events

b. sediment-rating curve for the project reach

c. flow-duration data for a gage on the same stream or on a similar stream

d. discharge hydrographs at the gage location

e. sediment rating curve at the gage location.

The theory behind this methodology is that the annual sediment load can be computed by adding up the contribution from hydrograph events, each weighted by their annual probability of occurrence. However, this summation gives only the contribution of the annual peak events, and not the smaller events that occur during the year. A correction factor must be computed to account for these events. It is computed using both flow-duration data and event hydrograph data at a gaged site. The gage should be as close to the project reach as possible: ideally on the same stream, but certainly in the same hydrophysiographic region.

Step 1. Compute event sediment yield at the project reach for a range of flood events, using the flood discharge hydrographs. In this example, the 2-, 5-, 10-, 25-, 50-, and 100-year flood events are used. The SAM program can be used to perform this computation.

Step 2. Plot the sediment yield for each event against its annual frequency of occurrence to give a single event sediment yield vs. annual recurrence interval curve.

Step 3. Determine the area under the curve to obtain the average annual sediment yield for the annual series of storm events (this is not the entire average annual sediment yield). This is mathematically given as:

$$Y_s = \int_0^1 Y_i dP$$

where Y_s is the sediment yield calculated from the sum of the frequency events, Y_i is the sediment yield associated with a given frequency of occurrence and P is the probability. This equation may be approximated numerically as:

$$Y_s = 0.015Y_{100} + 0.015Y_{50} + 0.04Y_{25} + 0.08Y_{10} + 0.20Y_5 + 0.40Y_2 \qquad \text{(E1)}$$

Note that there are other possible numerical approximations for the sediment yield. The preceding equation is based on trapezoidal segments. The curve should be examined to make sure the expressions for the first and last segments are reasonable. At the high end of the curve (above Y_{100}), an additional computation for Y_{200} may improve the estimate. At the lower end of the curve (below Y_2), the recurrence interval where the yield goes to zero should be estimated to give the proper coefficient for that segment of the area. A computation for an event below the 2-year might improve the estimate.

Step 4. Compute the correction factor, *J*. The calculated area under the curve represents the contribution of the series of annual storm events to the average annual sediment yield, but it does not include the contribution of lesser storm events. The area under the curve must be multiplied by a correction factor *J* to account for the difference between the annual peak series and the partial duration series and to account for other errors that may be associated with the numerical integration and with the difference between synthesized event hydrographs and a natural series of hydrographs. The correction factor is the ratio of the sediment yield computed using measured flow data to the sediment yield computed using the numerical approximation. The correction factor, *J*, is computed at a gage site, which should be as similar as possible to the project site in sediment and hydrologic characteristics.

So the basic equation is:

$$Y_a = J(0.015Y_{100} + 0.015Y_{50} + 0.04Y_{25} + 0.08Y_{10} + 0.20Y_5 + 0.40Y_2) \qquad \text{(E2)}$$

where Y_a is the average annual sediment yield.

Step 4a. Compute sediment yield at the gage site using weighted event hydrographs. This is a repeat of Steps 1, 2, and 3, except performed at the gage site.

Step 4b. Compute sediment yield at the gage site using mean daily flows and a site-specific sediment-rating curve. The SAM program can be used. If the stream is flashy, 15-min or 30-min data should be used if available.

Step 4c. Calculate the correction factor J, which is the ratio of the result of Step 4b to the result of Step 4a:

$$J = \frac{\text{sediment yield at the gage site computed using measured flows}}{\text{sediment yield at the gage site computed using weighted event hydrographs}}$$

Once the correction factor is computed, it may be reasonable to use it throughout a watershed. This could be particularly useful on studies where a hydrologic model of the watershed has been set up (so that discharge hydrographs are available). Average annual sediment yields could be computed fairly easily for a number of sites without gage data.

Step 4d. Calculate Y_a.

Appendix F
Example Scope of Work for Stability Assessment

Scope of Work

Preliminary Stream Restoration Assessment of Upper Studebaker River Watershed and Project Reach

General

Project

Upper Studebaker River Section 206 Aquatic Ecosystem Restoration.

Location

Upper Studebaker River, South Lake, CA.

Project Description

The U.S. Army Corps of Engineers (the Corps), and the City of South Lake are undertaking an aquatic ecosystem restoration project on the Upper Studebaker River (USR) as authorized by Section 206 of the Water Resources Development Act of 1996 (WRDA 96). Anthropogenic activities within the USR watershed such as logging, grazing, and commercial and residential development have impaired the natural functioning of its ecosystem. Additionally, these activities have also contributed to the ecosystem degradation of the USR's terminus, South Lake. The Upper Studebaker River Section 206 Aquatic Ecosystem Restoration (USR 206) Project seeks to remedy anthropogenic impacts to the USR watershed and South Lake aquatic ecosystems by implementing measures to restore the conditions of, and relationships between, its channel, riparian, and wetland habitats.

Study area, project reach, and subreaches

The study area for the USR 206 geomorphic assessment is the entire USR watershed. The watershed of the Upper Studebaker River covers more than 50 square miles. The upper end of the drainage basin begins in the mountains at about elevation 3,048 m (10,000 ft). From its headwaters, the USR flows westerly about 1.60934 km (1 mile), and then northerly about 8.04672 km (5 miles) through a steep, narrow canyon. Upon leaving the canyon the stream flows through a gently sloping valley that ends in South Lake. The upper basin is characterized by steeply rising, heavily timbered mountain slopes that terminate in large granitic outcrops at their crests. The upper basin is relatively pristine and has experienced minimal anthropogenic impacts. The lower basin has been impacted by logging, agriculture and urban development. Figure F1 displays the USR 206 study area location.

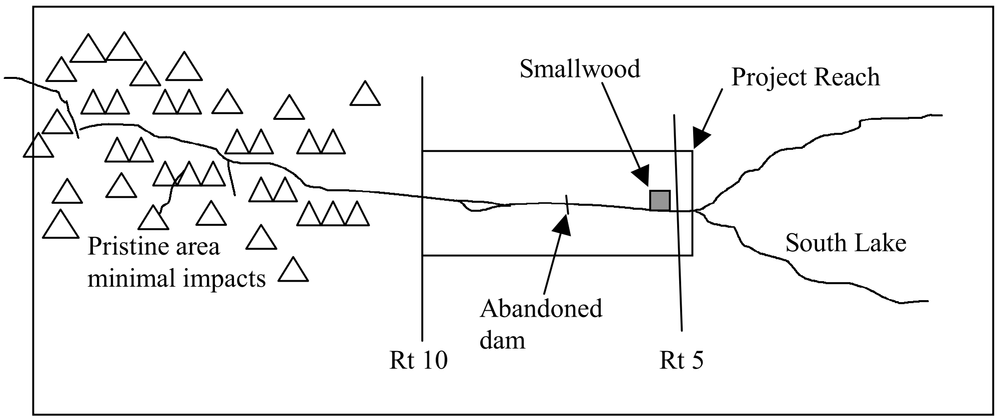

Figure F1. Study area and project reach

The project reach for the USR 206 project is located in the lower basin. It is defined as the USR channel as well as the current and historic floodplain between South Lake and the Highway 10 bridge crossing upstream and includes the community of Smallwood. The lower portion of the project reach has been channelized.

The USR 206 project reach has been subdivided into hydraulically relevant subreaches. Table F1 defines the hydraulic subreaches.

Table F1
USR 206 Project Hydraulic Subreaches

Hydraulic Subreach	Start Station	End Station	Boundary Descriptions (D/S – U/S)
1	0+00	6+00	South Lake Blvd. to Rt 5 bridge crossing
2	6+00	25+00	Rt 5 bridge crossing to upstream town limits of Smallwood. Channelized reach.
3	25+00	90+00	Upstream town limits of Smallwood to abandoned dam. Agricultural overbank.
4	90+00	160+00	Abandoned dam to downstream main channel/cutoff channel confluence. Agricultural overbank.
5	160+00	200+00L	Downstream main channel/cutoff channel confluence to upstream main channel/cutoff channel confluence. Left descending channel. Gravel mining in reach.
6	160+00	200+00R	Downstream main channel/cutoff channel confluence to upstream main channel/cutoff channel confluence. Right descending channel. Gravel mining in reach.
7	200+00	250+00	Upstream main channel/cutoff channel confluence to Rt 10 Bridge. Logging in overbank area.

Description of Services Required

General

The geomorphic processes of sediment generation and fluvial transport are fundamental and determining factors in the condition of the USR watershed's aquatic habitats. Therefore, a geomorphic assessment (GA) is to be performed in support of the USR 206 project in order to characterize how geomorphic conditions within the USR watershed influence its ecosystem. Habitats of primary interest to the restoration effort are aquatic channel, riparian, and wetland habitats. Parameters of primary importance to the quality of aquatic habitat are diversity and stability.[1] Therefore, particular emphasis shall be placed on characterizing how geomorphic conditions within the USR watershed impact the ecology and relative stability of morphometric channel and floodplain conditions within the project reach.

The geomorphic processes of sediment generation and fluvial transport within the USR watershed are also relevant to the issue of South Lake's diminishing clarity, its most visible and noteworthy symptom of ecosystem degradation. Therefore, another goal of the GA is to characterize how geomorphic conditions within the USR watershed influence South Lake's diminishing clarity.

The GA shall consist of a reconnaissance-level analysis of the entire USR watershed and a more detailed level analysis of the project reach. Watershed analysis shall focus on characterizing sediment sources and contributing land-use practices, particularly where sediment generation rates appear to be inordinately high. Project reach analysis shall focus on assessing the channel's ecology, geomorphic stability, and behavior.

Review of available material

Familiarization of the study area shall be achieved through discussions with pertinent individuals and agencies and review of prior reports of the study area, including Federal Emergency Management Agency (FEMA) studies, hydrologic models, and aquatic surveys.

Field investigations

Generally, field investigations shall be performed as necessary to characterize geomorphic conditions in the USR watershed and support a geomorphic stability assessment of the project reach. It is expected that the field efforts will be conducted with an engineer who is familiar with hydraulics, sedimentation, and geomorphology as well as a biologist familiar with the ecological area. Specific field investigations required are described in the following sections.

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[1] Stability in this context is relative to channel conditions of dynamic equilibrium, i.e., the balancing of sediment inflow and sediment transport capacity by relatively modest adjustments in channel dimensions.

Watershed assessment

Available material shall be researched, compiled, and reviewed. Field investigations shall be conducted as necessary to perform the following tasks. It is expected that field examinations and verifications will be necessary for completion of these tasks.

a. Define the relevant geologic characteristics of the USR watershed and the USR 206 project reach. Acquire and review available topographic data including U.S. Geological Survey (USGS) topo maps, surveyed topography, aerial photos, etc. Identify general subsurface, soil types, cover conditions, and relevant properties within the project reach.

b. Prepare a summary and a time line of the history of land-use activities and associated geomorphic conditions in the USR watershed. Include all identifiable events of geomorphic significance.

c. Identify USR sediment sources and contributing land-use practices. Note where sediment generation rates appear to be inordinately high. Provide a general characterization of sediment sources based on their relative contribution to the project reach's bed-load, suspended-load and wash-load supply.

d. Identify dominant geomorphic processes within the USR watershed. Assess whether each process is natural or anthropogenic.

e. Identify any apparent geologic and/or anthropogenic structural controls on the geomorphic conditions of the watershed.

f. Identify aquatic species that have been impacted in the study reach.

g. Assess point and nonpoint source water quality impacts on the watershed with particular emphasis on the limitations that it may place on target aquatic species.

h. Contact the state fish and wildlife agencies with regard to their stocking program and locate stocking sites on the maps (frequency, species, etc). Determine if the state has historic records of declining fisheries on any particular streams and/or reaches and locate these on a map. Assess records and studies to determine if specific blockages to fish passage have adversely impacted fisheries.

Project reach assessment

Available material shall be researched, compiled, and reviewed. Field investigations shall be conducted as necessary to perform the following tasks. It is expected that an experienced field crew will need to walk the entire reach.

a. Acquire and review available topographic data including USGS topo maps, surveyed topography, aerial photos, etc. Identify current and historic channel types, locations, planform characteristics (to include sinuosity, wavelength, and meander belt width). Prepare a figure displaying historic channel locations.

b. Compare current and historic topographic (planform and vertical) data. Identify indications of historic channel behavior and current channel condition. When feasible, identify the effects of relevant anthropogenic activities on channel morphology. Estimate historic lateral migration rates. Prepare a figure displaying locations.

c. Identify current geomorphic subreaches. Prepare a table displaying the beginning and ending stations of each subreach. Table F1 may be used as a reference. Subsequent tasks shall reference geomorphic subreaches identified, where appropriate.

d. Assess point and nonpoint source water quality impacts to the reach. Collect water quality samples from high flow and low flow conditions from each of the subreaches. Prepare a figure displaying locations.

e. Characterize the longitudinal location of the project reach relative to the watershed in terms of its sediment transport/geneneration behavior (i.e., zone of erosion, transportation, or deposition). Prepare a figure displaying locations.

f. From field measurements, identify average channel morphometry for each subreach to include bankfull channel dimensions (defined as minimum width to depth ratio), overbank characteristics, and base flow channel dimensions.

g. Identify active and remnant floodplain surfaces (terraces). Prepare a figure displaying locations.

h. Characterize the bed material and bed forms. For each subreach, sample a representative reach for bed-material load calculations and one representative area for low flow habitat conditions. Prepare a figure or table identifying sample locations and characteristic hydraulic conditions relative to project stationing. Perform standard laboratory size distribution (sieve) analysis. Prepare a standard plot(s) of bed sample gradation curves and a table(s) of bed sample grain size data. Provide a general characterization of the sources of project reach bed material.

i. Provide a general characterization the ecology of each subreach. The inventory of aquatic habitat will utilize existing data on benthos and finfish sampling as well as a rapid bioassessment of physical instream habitat to indicate current habitat conditions.

j. Characterize the bank material and stratigraphy for each of the geomorphic subreaches. For each subreach, collect representative bank material samples. Perform standard laboratory-size distribution (sieve) or hydrometer analysis, as appropriate. Identify bank material soil types and properties. Provide a general characterization of the relative cohesiveness and erodability of the bank materials. Identify bank erosion and failure mechanisms. Characterize the existing bank vegetation.

k. Identify significant sediment sources and sinks within the study reach. Assess sediment impacts from sources in the upper basin above the study reach.

l. Observe tributary, distributary, and relict channels in the project reach, and identify indications of channel behavior and geomorphic conditions.

m. Observe anthropogenic features including bridge abutments and piers, grade control structures, low flow crossings, and bank protection. Identify impacts of features and indications of channel behavior and geomorphic conditions. Identify significant geomorphic controls in the project reach.

n. Acquire and review USGS gaging station records, including surveyed cross sections and rating tables. Perform specific gage analysis and identify indications of channel behavior and geomorphic conditions.

o. Characterize the current geomorphic stability of the subreaches in project reach channel, whether incising or aggrading. Identify the severity and extent of any existing vertical or horizontal instabilities via a qualitative index.

p. Characterize the grade conditions of the channel, whether incised, aggraded, or at-grade. Apply appropriate channel evolution models to identify current channel stage, subsequent stages of evolution, and the evolved stable channel form. Qualitatively estimate the time scale of channel recovery. Characterize the impacts of anthropogenic features on channel morphology and stability.

q. Identify potential problem areas in the project reach. Characterize the potential for significant increase and/or decrease in the project reach sediment supply. Characterize the sensitivity of channel form to such variations, including expected channel form response, and the magnitude and time scale of expected adjustments. Characterize the impacts of anthropogenic features on expected channel behavior.

r. Characterize the relative uncertainty of the assessment performed. Identify any additional analyses required to develop a reasonably certain geomorphic assessment.

Restoration recommendations

Based on the geomorphic assessment performed, recommend restoration measures and appropriate analysis and design methodologies. Specific items to be addressed are described in the following sections.

a. Recommend measures to restore geomorphic stability and ecological health to the project reach. Include an assessment of the appropriateness of bank protection, grade protection, instream habitat enhancement, channel realignment, wetland creation, and fish blockage removal.

b. Recommend measures to reduce impacts to water quality from the project reach area. Include an assessment of the appropriateness of bank protection, grade protection, channel realignment, and riparian modifications.

c. Document each proposed measure. The information will include an estimate of the potential project size, the general project type, a sketch of the site, the impact to and proximity of utilities, and an assessment of construction area and access. Provide a general estimate of costs.

d. Address specifically the appropriateness and feasibility of possible projects. Particularly address impacts to existing floodplains. Address social and biologic controls that may limit possible projects.

Meeting attendance

There will be four required meetings to review study progress. These shall be at the initiation of work, at the completion of field investigation activities, at the midpoint of the study process and finally at the end of the study to present findings.

Quality Control and Quality Assurance

Quality control (QC) of the technical products produced under this scope of work shall consist of development and execution of a Quality Control Plan (QCP), independent technical review (ITR), and Quality Control Certification (QCC). The experience and background of personnel selected for the field assessments will be reviewed to ensure that the study is conducted by qualified personnel. Products shall be reviewed for compliance with standard engineering and professional practices, adequacy of the scope of the associated document, appropriateness of data used, consistency, accuracy, comprehensiveness, and reasonableness of results.

Appendix G
Example Sediment Impact Assessment and Stable Channel Design

A sediment impact assessment was conducted as part of the reconnaissance level planning study for a flood damage reduction project for the City of Carlsbad, NM (Copeland 1995).[1] The purpose of the sediment impact assessment was to identify the magnitude of possible sediment problems that might be associated with the proposed project. One potential source of flooding was Dark Canyon Draw, a tributary of the Pecos River (Figure G1). One of the flood damage reduction alternatives being considered was a bypass channel that would divert Dark Canyon Draw around the City of Carlsbad. The proposed diversion would begin near the city airport and flow northeasterly to the Pecos River to a location about 8.04 km (5 miles) downstream from the city.

Depending on the diversion channel design, several sedimentation and channel stability problems could occur. If a threshold channel is constructed (a channel designed with little or no sediment transport potential), then bed material delivered from upstream would deposit at the diversion entrance. Sediment deposits would have to be removed periodically. If a channel is designed to carry the incoming sediment load, there will be a period of adjustment for the channel, as the bed and banks become established. Bed armoring may progress quickly or slowly, with extensive degradation, depending on the consistency of the material through which the diversion channel is cut and the sequence of annual runoff that occurs. And finally, if the diversion channel is too efficient in terms of sediment transport capacity, it could degrade and induce additional channel degradation upstream from the diversion location. The sediment impact assessment was conducted to determine the magnitude of possible sediment degradation or aggradation problems and to obtain relatively stable dimensions for the diversion channel.

[1] All references cited in this appendix are listed in the References section following the main text.

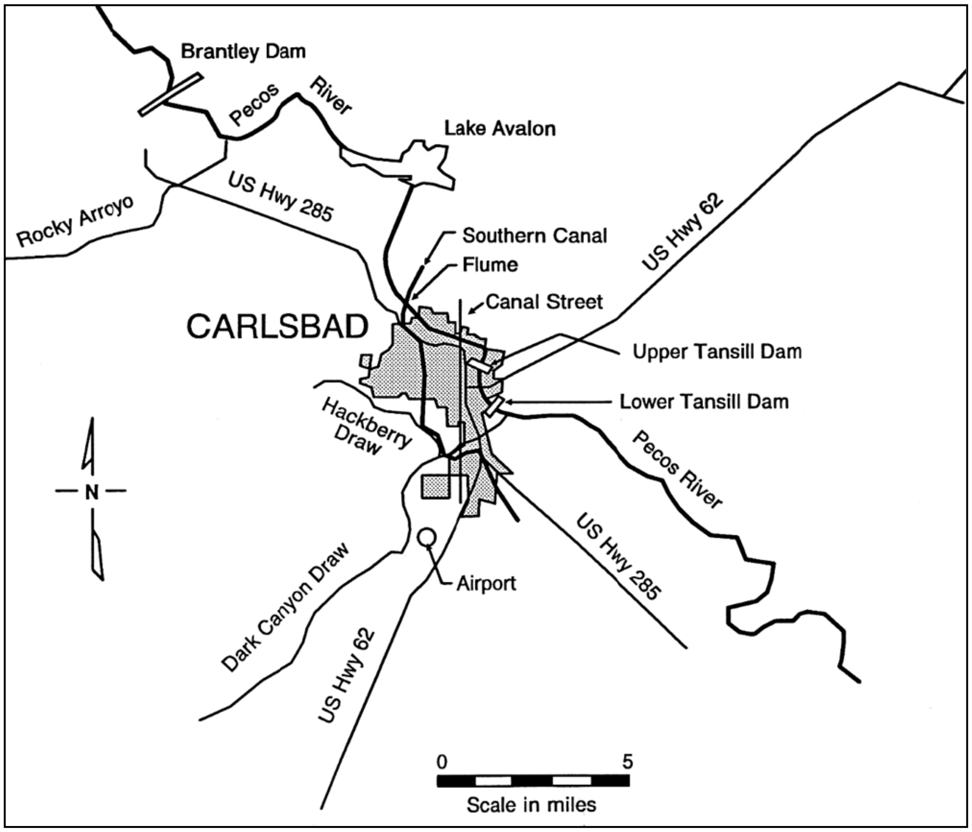

Figure G1. Carlsbad and surrounding areas (To convert miles to kilometers, multiply by 1.609347)

Field Reconnaissance

A preliminary assessment of channel stability and potential sediment impacts were determined during a two-day field reconnaissance. This brief reconnaissance provided insight for general observations related to channel stability.

Dark Canyon Draw transitions from a wide shallow alluvial channel, characteristic of Southwestern United States alluvial fans, at its canyon mouth to an incised arroyo at its confluence with the Pecos River. Gravel mining is currently active in the lower reaches of Dark Canyon Draw between the Pecos River and the city airport and appeared to have been occurring for many years. Due to the gravel mining, the channel had been both widened and deepened. The channel also showed signs of incision/degradation upstream from the airport. The bed and banks of the incised channel were capable of supplying significant quantities of sediment to the stream. The bed surface of Dark Canyon Draw consisted primarily of coarse gravel and cobbles. Banks were generally composed of loose alluvial material ranging in size from clays and silts to boulders. The channel tended to migrate laterally, eroding banks and creating remnant gravel bars in former channels. Armoring was generally observed in the existing low flow channel. However, at high flows the channel would migrate, mobilizing significant sediment from the gravel bars and from bank erosion.

Bed material samples were collected during the field reconnaissance. Sample size class distributions were determined using the Wolman (1954) pebble count method. Due to the limited scope of the sediment impact assessment, samples were collected at only two sites. Both surface and subsurface samples were collected at the mouth of the canyon, which is several kilometers (miles) upstream from the proposed diversion channel. There was no coarse surface layer at the second site, which was located on a gravel bar, about 1.5 km (1 mile) downstream from the canyon mouth. The thoroughly mixed bed form was an indication that active-layer mixing had occurred during the last flow event at this site. Median grain size ranged between 22 and 55 mm for all the samples. The gradation determined at the downstream site was selected as the representative gradation for the sediment impact assessment because it was characteristic of a fully mobile bed. Bed material gradations determined from these samples are shown in Figure G2.

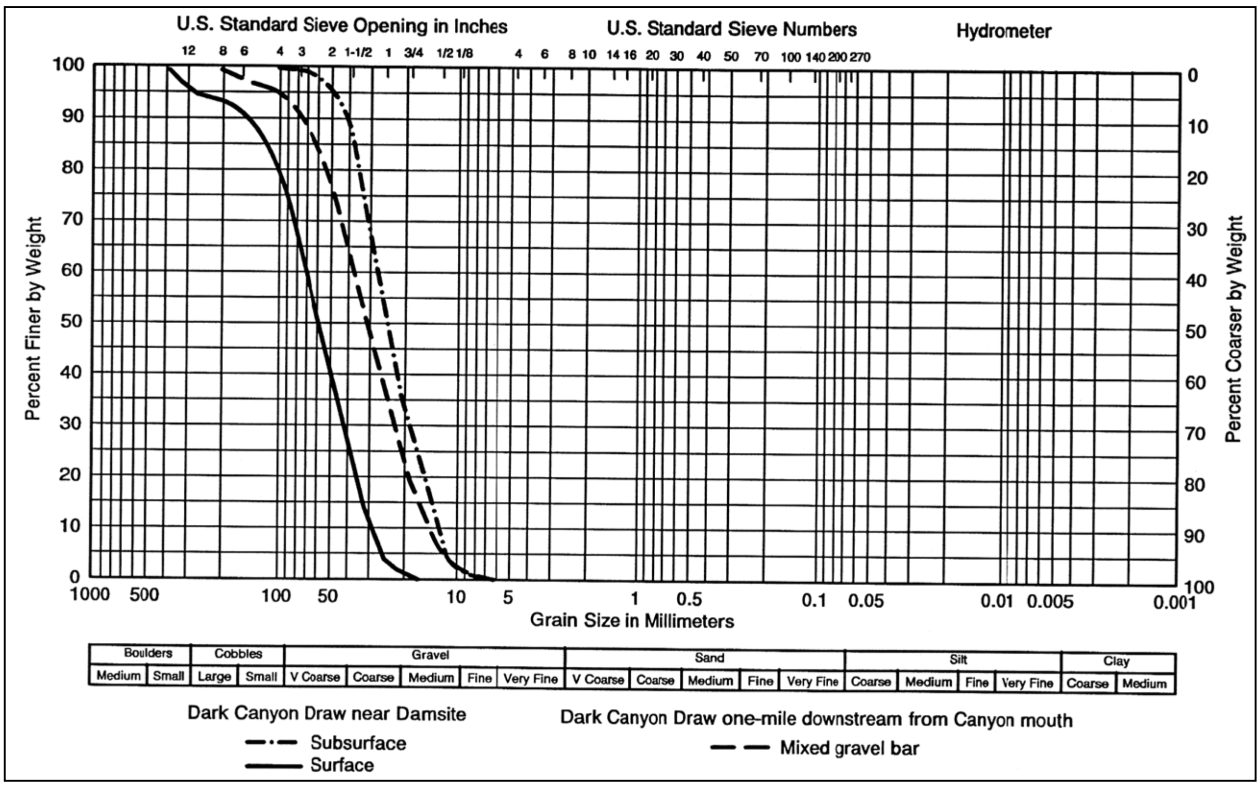

Figure G2. Bed material gradations, Dark Canyon Draw

Hydrology

Hydrographs used in the sediment impact assessment were developed using the HEC-1 hydrograph package (USACE, HEC 1981). These were used to calculate sediment yield for flood events. The peak discharge for the 1-percent exceedance flood was 2,000 m^3/s (75,000 ft^3/s). The 10-percent chance exceedance hydrograph was assumed to have the same shape as the 1-percent chance exceedance flood; discharges on the hydrograph were calculated by multiplying the 1-percent chance exceedance hydrograph by the ratio of the peaks. The 10-percent chance exceedance peak discharge was 570 m^3/s (20,000 ft^3/s). The 1-percent chance exceedance hydrograph for Dark Canyon Draw is shown in Figure G3.

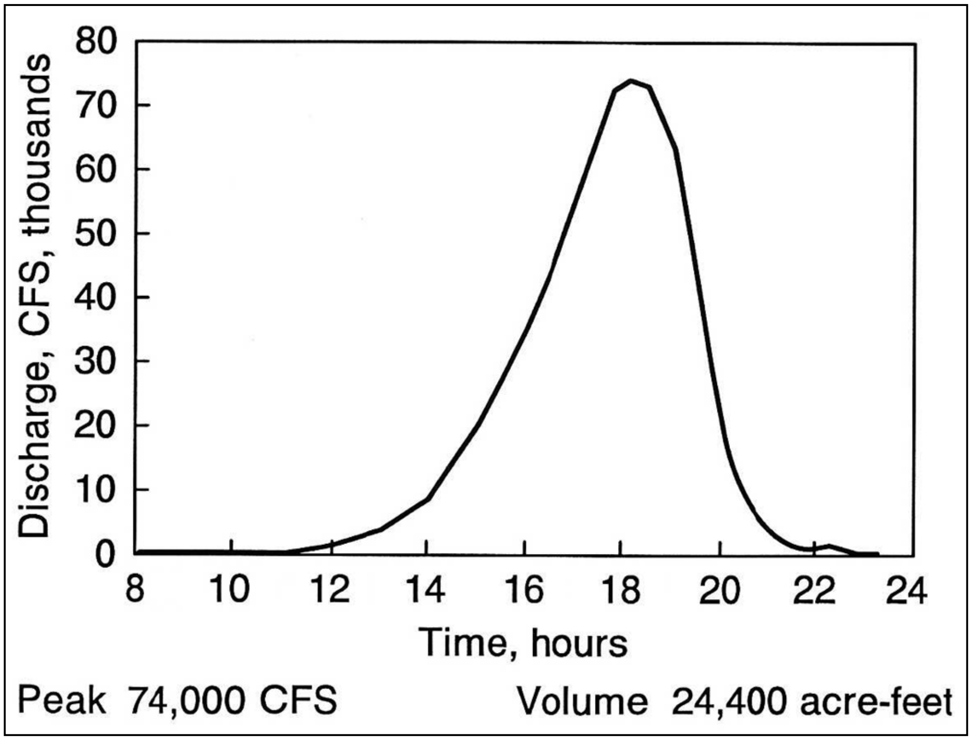

Figure G3. One-percent chance exceedance hydrograph, Dark Canyon Draw (To convert cubic feet per second to cubic meters per second, multiply by 0.02831. To convert acre-feet to cubic meters, multiply by 1,233.489)

A flow-duration curve was developed from 18 years of USGS mean daily flow data from the Dark Canyon at Carlsbad gage. Durations of published peak flows greater than the maximum mean daily flow were added to the flow-duration data by assuming the historical flood hydrographs had shapes similar to the 1-percent change exceedance hydrograph. The flow-duration curve is shown in Figure G4.

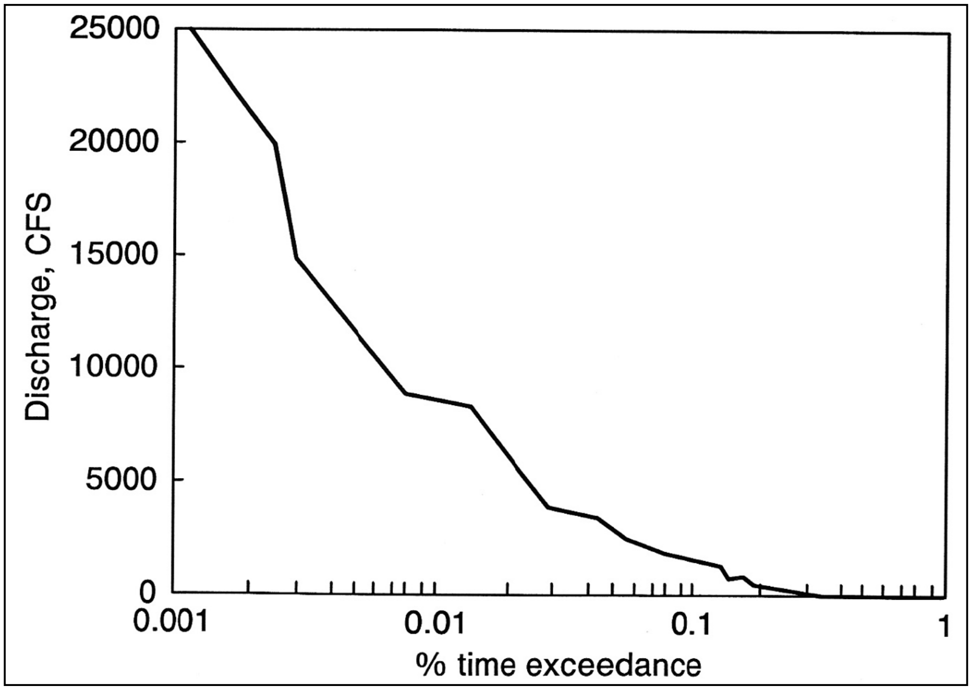

Figure G4. Dark Canyon Draw at Carlsbad, NM (To convert cubic feet per second to cubic meters per second, multiply by 0.02831)

Average Hydraulic Parameters

A typical reach in the existing Dark Canyon Draw channel was selected from a HEC-2 (USACE, HEC 1990) backwater model. The typical reach chosen for this analysis was about 3.21 km (2 miles) long and was located adjacent to the Carlsbad Airport. The reach was considered to be in a state of nonequilibrium due to its proximity to gravel mining operations. A reach further upstream, less influenced by gravel mining operations, would have been preferred for determining long-term sediment yield. However, the existing backwater model did not extend any further upstream. It was recommended that additional cross-section surveys upstream be obtained for more detailed sediment studies.

Water-surface elevations and hydraulic variables were calculated using the HEC-2 model for a range of discharges. Average values for hydraulic variables were then determined using the reach-length weighted averaging procedure in SAM (Thomas et al. 2000).

Sediment Transport Rating Curve

The bed material sediment yield of Dark Canyon Draw is important when considering sediment transport and channel stability questions. The bed material sediment load consists of the sediment sizes that exchange with the streambed as they are transported downstream. The bed-material yield is most likely to be relatively small compared to the total sediment yield because the bed of Dark Canyon Draw consists primarily of gravels and cobbles. The wash load component of the total sediment yield will be transported through the system to the Pecos River, unless it is trapped by a reservoir or introduced into a ponded area.

Sediment transport was calculated using several sediment-transport equations available in the SAM program. The equations chosen were equations that included at least some data from gravel bed rivers in their development. As can be seen from the sediment-discharge rating curves, shown in Figure G5, there is a wide range in predicted sediment transport rates. There are no available data on Dark Canyon Draw to aid in the selection of a transport equation. However, the guidance program in SAM identified the North Saskatchewan and Elbow Rivers in Saskatchewan, Canada, as having similar median bed grain sizes, depths, velocities, and slopes as Dark Canyon Draw at high flow. The guidance program determined from the available set of equations in SAM that the Schoklitsch equation (Shulits 1935) best reproduced measured data on the North Saskatchewan and Elbow Rivers. Based on the comparison of calculated sediment transport rating curves using different sediment transport functions shown in Figure G5, the Schoklitsch equation will produce a relatively low sediment yield. In order to cover the uncertainty range in the calculated bed material sediment yield, two additional sediment transport equations were chosen to calculate yield. The Parker equation (Parker 1990) was used to represent a high sediment transport load, and the Einstein (1950) equation was chosen to represent an intermediate sediment transport load.

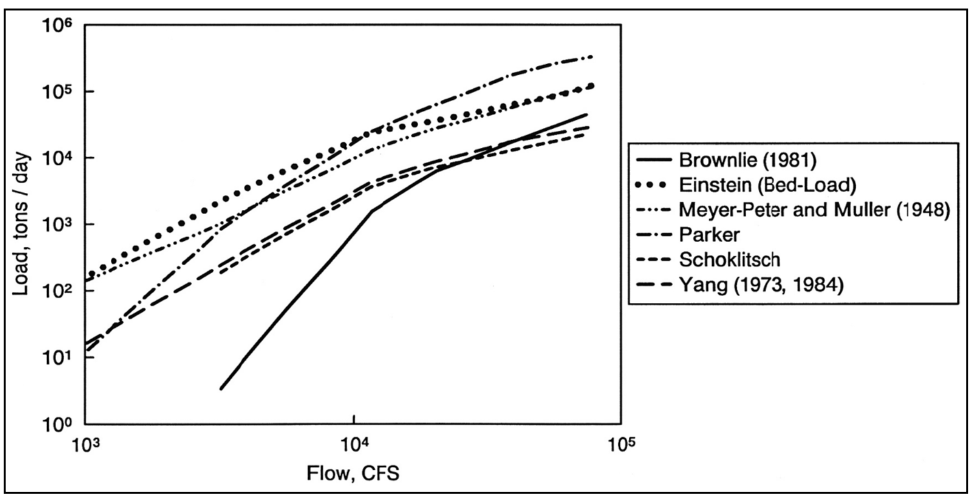

Figure G5. Bed material sediment transport rating curves, Dark Canyon Draw (To convert cubic feet per second to cubic meters per second, multiply by 0.02831. To convert tons to newtons, multiply by 8,896.443)

Diversion Channel Design

The stable-channel analytical design method in SAM was used to size the low-flow channel. This method provides channel dimensions that will transport the incoming bed-material sediment load for a specified discharge. The method uses the Brownlie (1981) equation to calculate sediment transport and roughness on the channel bed. This equation was not developed for gravel bed streams, and predicts lower sediment transport rates at lower discharges than other tested equations (Figure G5). This apparent deficiency in the sediment-transport equation is accounted for later by testing the resultant cross-section geometry using other transport equations.

The criteria chosen for the diversion channel design were: (a) a composite channel geometry with a low-flow channel designed to carry the effective discharge, and (b) the overbank designed using threshold criteria for the one-percent chance exceedance flood. Assigned side slopes were 1V:3H, with a side slope Manning's roughness coefficient of 0.05.

The effective discharge is the discharge that transports the largest percentage of the bed material sediment load. This was determined by integrating the flow-duration curve for Dark Canyon Draw and a sediment-transport rating curve developed using the Einstein formula. A plot of percentage of bed-material load versus discharge increment is shown in Figure G6; an effective discharge of 240 m^3/s (8,500 ft^3/s) was indicated.

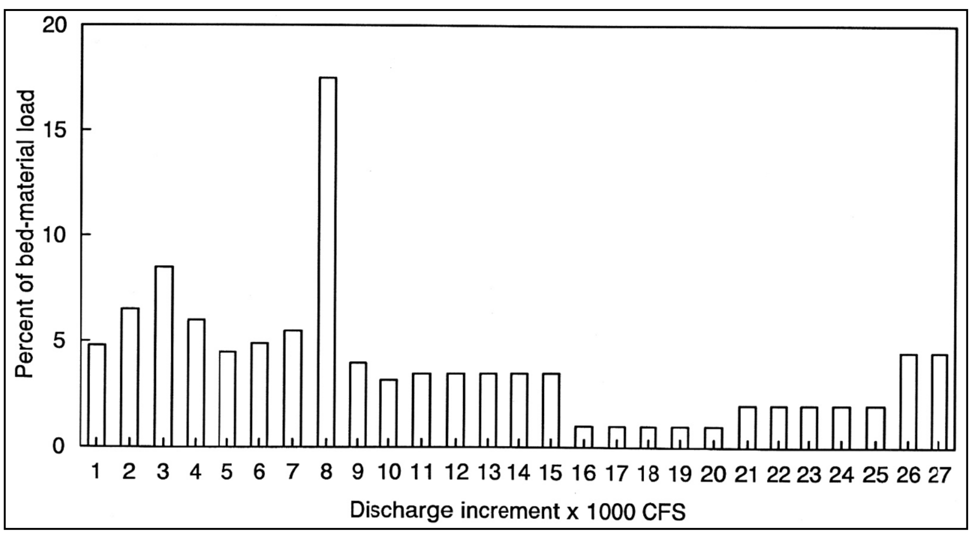

Figure G6. Effective discharge, Dark Canyon Draw (To convert cubic feet per second to cubic meters per second, multiply by 0.02831)

The inflowing sediment concentration was determined for the effective discharge from the sediment-transport rating curve developed for the typical reach of Dark Canyon Draw. The Brownlie sediment-transport equation was used for typical reach to be compatible with the calculations in the design reach. The bed material gradation in the diversion channel was assumed to be the same as in Dark Canyon Draw. This is a reasonable assumption for the long-term condition in the diversion channel, but not for initial conditions. The transition from initial to final conditions could be determined in future, more detailed studies using a numerical model such as HEC-6 (USACE, HEC 1993).

Using the natural slope between the proposed Dark Canyon Draw diversion and the Pecos River, a unique solution for width and depth was determined for the effective discharge channel. The average slope between Dark Canyon Draw at the airport and the Pecos River is 0.0047. The ground slope is steeper at the airport and becomes very mild as it crosses the Pecos River floodplain. A more detailed analysis should include different channel geometries due to variation in slope.

The stable channel curve for 240 m^3/s (8,500 ft^3/s) is shown in Figure G7. This curve defines combinations of width and slope that would provide for movement of the inflowing sediment load through the diversion channel. The average slope for the diversion channel, if no drop structures were employed, would be 0.0047. With this slope, the stable channel method suggests that a base width of about 122 m (400 ft) would be stable. The calculated depth was 1.1 m (3.5 ft).

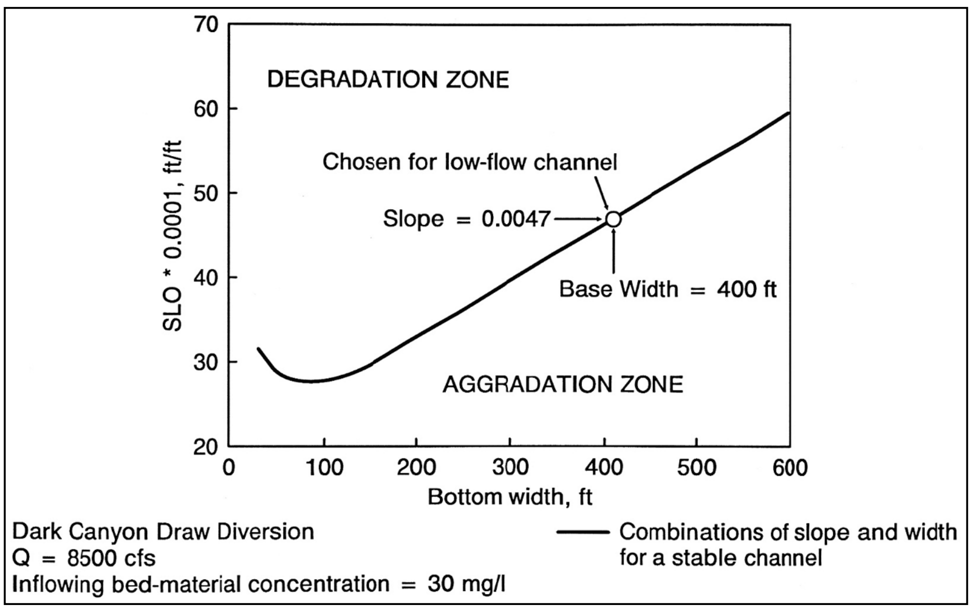

Figure G7. Preliminary diversion channel design (To convert feet to meters, multiply by 0.3048)

The width of the overbank portion of the channel was determined by trial and error using the threshold velocity criteria from EM 1110-2-1418. With a median bed material size of about 30 mm and a water depth of 1.5 m (5 ft), a threshold velocity up to 1.8 m/s (6 ft/s) would be appropriate for channel stability considerations. Roughness on the overbank was calculated using the Brownlie roughness predictor. The total width of the overbank and channel was determined to be 850 m (2,800 ft). The details of the final geometry is shown in Figure G8. If the threshold velocity is exceeded, degradation can be expected.

The extent of degradation can be estimated in a more detailed study using the HEC-6 numerical sedimentation model.

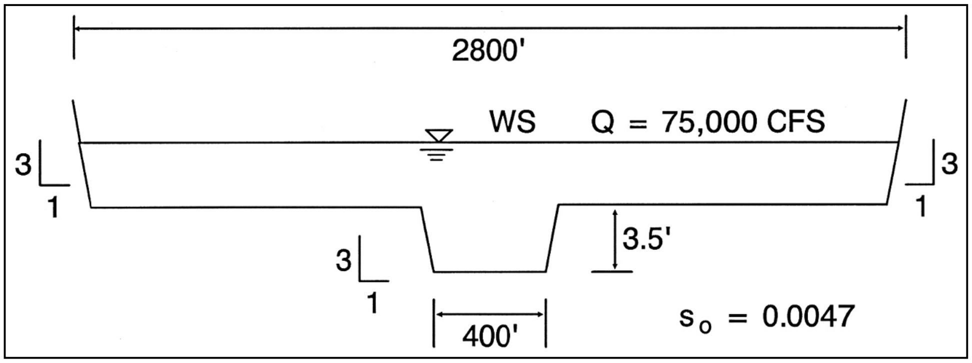

Figure G8. Cross section Dark Canyon Draw Diversion Channel (To convert cubic feet per second to cubic meters per second, multiply by 0.02831. To convert feet to meters, multiply by 0.3048)

Sediment Budget

The magnitude of potential aggradation or deposition problems in the Dark Canyon channel can be determined by calculating bed material sediment yield through a typical reach of the existing channel and comparing it to calculated sediment yield in the project reach.

Bed material sediment yield was calculated for the existing channel using the flow-duration sediment transport curve method and SAM. Sediment yields were calculated for the 1-percent and 10-percent chance exceedance floods using synthetic hydrographs, and for average annual conditions, using the flow-duration curve. Bed-material sediment yields calculated using three different sediment transport equations are listed in the following tabulation.

Calculated Bed-Material Sediment Yield[1]
Dark Canyon Draw

	1-percent exceedance flood		**10-percent exceedance flood**		**Average Annual**	
	m^3	**yds^3**	**m^3**	**yd^3**	**m^3**	**yd^3**
Schoklitsch	2,400	3,100	530	690	180	230
Einstein	11,300	14,800	3,300	4,300	1,300	1,700
Parker	27,700	36,200	4,100	5,400	1,100	1,500

[1] Sediment yield volume calculated assuming specific weight of deposit of 1,500 kg/m^3 (93 lbs/ft^3)

Sediment yield was determined in the diversion channel using the same procedure that was used to calculate sediment yield in the typical reach of the existing channel. Trap efficiency was then determined for flood hydrographs and for average annual conditions.

The potential for aggradation/degradation in the diversion channel for a 10- and 1-percent chance exceedance floods and for average annual conditions was determined using the sediment budget approach. Bed material sediment yield was calculated using three sediment transport equations and compared to the calculated bed material sediment yield in the existing Dark Canyon Draw. Bed material sediment transport was assumed to occur only in the low flow channel in the diversion. Calculated bed material sediment yield and its percentage of the total bed material yield calculated for Dark Canyon Draw is shown in the following tabulation. This tabulation indicates that deposition will occur in the diversion channel for all cases tested. For the 1-percent chance exceedance flood, between 34 and 38 percent of the inflowing bed material sediment load will deposit in the diversion channel. For the 10-percent chance exceedance flood, between 12 and 17 percent of the inflowing bed material load will deposit. For average annual conditions, between 6 and 18 percent of the inflowing sediment load will deposit. A range of the quantities of deposition can be determined from these calculations. Recall that the Schoklitsch equation produced sediment transport quantities closest to the measured data from a river with similar characteristics.

Calculated Bed-Material Sediment Yield [1]
Diversion Channel

Sediment Transport Function	1-Percent Exceedance Flood			10-Percent Exceedance Flood			Average Annual		
	m^3	yd^3	Percent of Inflow	m^3	yd^3	Percent of Inflow	m^3	yd^3	Percent of Inflow
Schoklitsch	1,600	2,050	66	450	590	86	150	190	82
Einstein	7,500	9,800	66	2,900	3,800	88	1,200	1,600	94
Parker	17,100	22,400	62	3,400	4,500	83	1,000	1,300	87

[1] Sediment yield volume calculated assuming specific weight of deposit of 1,500 kg/m^3 (93 lbs/ft^3)

At the next level planning, it would be necessary to evaluate the temporal development of the diversion channel using the HEC-6 numerical sedimentation model. In this sediment impact assessment, the bed material gradation was assumed to be already developed. A more detailed study would require knowledge of the existing soil profile through which the channel will be cut. The armoring process would have to be simulated with a numerical model. In addition, the slope of the diversion channel will vary between the diversion point and the Pecos River. This requires a more detailed analysis of spacial variability in the sedimentation processes.

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14. ABSTRACT

This document provides a systematic hydraulic design methodology to aid hydraulic engineers in the design of stream restoration projects. The objective is to achieve a channel design that fits into the natural system within the physical constraints imposed by other project objectives. In the Corps of Engineers, stream restoration projects are frequently associated with or part of a flood-control project. Thus, projects have more than one objective and compromises may be required to meet essential portions of each objective. The hydraulic design of a stream restoration project should provide for a channel that is in dynamic equilibrium with its sediment load. A sound stream restoration design incorporates techniques from both fluvial geomorphology and physics. The study area to which these techniques are applied must extend beyond the limits of the project site to the extent that both the project's effect on the stream system and the stream system's effect on the project reach can be determined. The iterative systematic approach presented includes defining project objectives and constraints; determining appropriate hydrologic data; conducting a stability assessment of the existing stream system channel to establish baseline geomorphological conditions and to evaluate the effectiveness and geomorphological impacts of project alternatives; and a methodology for hydraulic design of project features and for assessing hydraulic and sediment transport impacts of alternatives. Appendices provide useful tools and examples for use in this methodology.

15. SUBJECT TERMS

Alluvial channels	Bed material sampling	Effective discharge	Hydraulic design
Bankfull discharge	Channel-forming discharge	Flood damage relief techniques	Hydraulic geometry assessment
	Channel stability	Geomorphic assessment	SAM hydraulic design package (cont.)

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