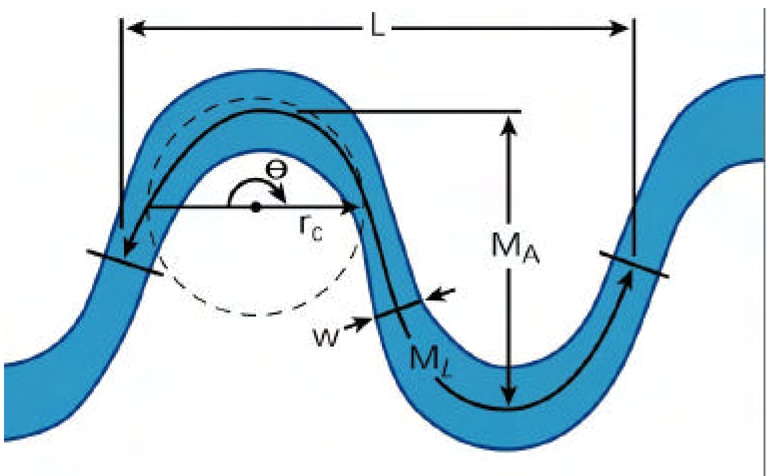
Volume 1

II. Stream Restoration



L meander wavelength
M_L meander arc length
w average width at bankfull discharge
M_A meander amplitude
r_c radius of curvature
Θ arc angle

II. Stream Restoration

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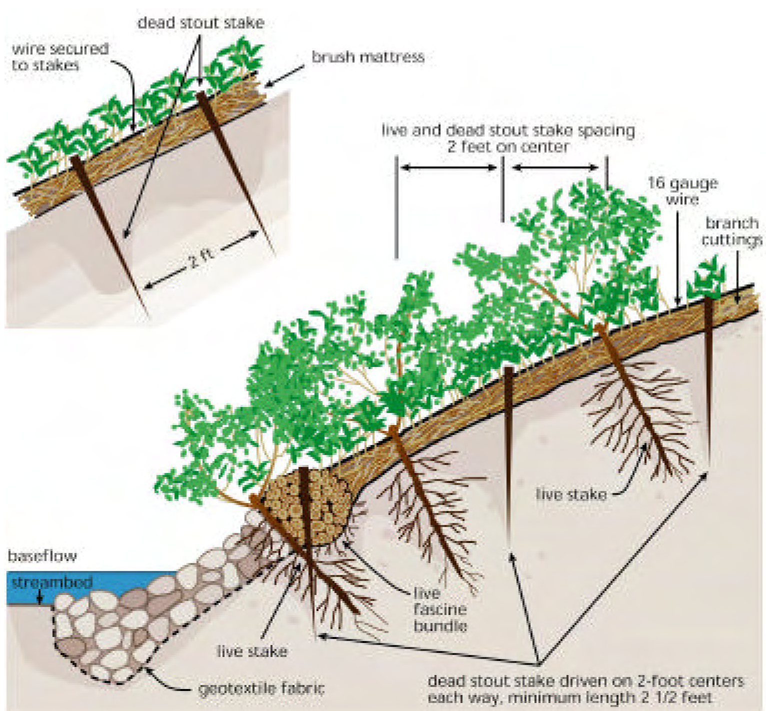
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2 ft
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16 gauge wire
branch cuttings
live stake
baseflow
streambed
live fascine bundle
live stake
geotextile fabric
dead stout stake driven on 2-foot centers each way, minimum length 2 1/2 feet

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AN OVERVIEW OF THE USACE STREAM RESTORATION GUIDELINES

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Abstract: This paper presents an overview of U.S. Army Corps of Engineers guidelines for hydraulic design of channel restoration projects. The methodology uses sound physical principles based on well established engineering formulae. These guidelines bring together and summarize many of the stream restoration hydraulic design techniques used by the Corps. The guidelines describe: 1) achieving stakeholder consensus on project objectives and constraints, 2) conducting hydrology studies, 3) conducting a stability analysis of the existing stream and watershed, and 4) a hydraulic design methodology.

INTRODUCTION

As a result of increased public appreciation of the environment, many federal, state and local governments as well as grass roots organizations are actively engaged in stream restoration. Engineers are being asked for assessments, characterizations, analyses and designs, which focus on restoring, establishing or maintaining natural physical stream environments. The purpose of this paper is to provide a summary of a systematic hydraulic design methodology for hydraulic engineers involved in stream restoration projects. The USACE guidelines are not a 'cook-book' or set of steps that must be exactly duplicated in all situations. The objective of the methodology is to design a stream restoration project that fits into the natural system within the physical constraints imposed by the environment and project objectives. This paper focuses primarily on small stream restoration projects. However, the principles presented also have application to a wider range of projects.

Channel stability is often essential to the maintenance of favorable environmental conditions. Stream restoration projects addressed herein include the alteration of a stream channel to a stable geometric configuration that is in balance with imposed flow and sediment regimes and with the character of the watershed. A stable channel is defined as a channel where the planform, cross-section, and longitudinal profile are maintainable over time. Under natural conditions, the channel may migrate laterally and longitudinally and still be considered stable. However, allowing a restored channel to migrate may not be feasible due to project constraints.

A wide variety of analysis techniques can be applied to the hydraulic design of stream restoration projects. A sound stream restoration design incorporates techniques from both fluvial geomorphology and physics. 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 provides generalized guidance related to appropriate dimensions for cross-section geometry and channel planform. It is important to recognize that fluvial geomorphology is primarily observationally based. As a result, predicted trends and changes tend to represent average conditions. Assessment and design for a specific project area also require the use of physically based calculations. These calculations should address areas of hydrology, hydraulics and sediment transport.

The approach for the hydraulic design of stream restoration projects presented herein includes: 1) defining project objectives and constraints; 2) determining hydrologic information; 3) assessing the existing stability of the watershed and stream; and 4) the hydraulic design of the restored stream and project features.

OBJECTIVES AND CONSTRAINTS

General: The perceived success or failure of many stream restoration projects can be as much a function of the success criteria as the design. Therefore, the importance of establishing achievable project objectives is critical. Once established, objectives will define the data collection effort, methodologies for assessments, and finally the design itself. Since few people possess all the skills necessary to conduct a successful stream restoration study and design, an interdisciplinary team is required. While the exact makeup of the team can vary, it should include engineering, geomorphological and ecological expertise.

Objectives: The first step in a stream restoration project, as with any engineering project, is to clearly define project objectives in cooperation with stakeholders. Generalities in objectives, such as 'fixing' the stream, can lead to problems. Narrowing the objectives reduces ambiguity for the study team members. Objectives should be specific, as well as realistic and achievable.

Restoring a stream to a 'natural' condition is not a clear objective. The natural condition is often defined by aesthetic guidelines, which are subjective. It is important to realize that a stream that is behaving 'naturally' may be detrimental to riparian land use and may not possess optimal habitat for aquatic and riparian wildlife. For example, natural braided streams are typically characterized by shallow flow, avulsions, lack of pools and frequent out-of–bank flooding. Although these characteristics are natural, they may not provide optimum habitat benefits for a targeted species. The geometry of a braided stream reach is sometimes altered by forcing the stream into a single thread. Since this change alters the natural condition, it is important to recognize that considerably more engineering effort would be required to hold the altered channel in place over that required if the target conditions were more in keeping with the existing morphology of the stream.

Restoring streams to some historical condition may be an objective. If this is the approach, care must be taken to assure that physical or biological changes in the watershed have not prohibited a return to an historic condition. For example: the objective for an incised and widening stream in an urban watershed could be to restore it to support a sensitive fish species that was present before development. Changes in water quality and runoff patterns could make this objective impossible to achieve. As a result, it should be recognized that many "restoration" projects are actually environmental enhancement projects, since it may not be feasible to return a system to an historical condition.

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

1- Existing conditions of the stream and watershed.
2- The scale and severity of the resource loss or degradation.
3- Factors that have resulted in the current stream condition through an assessment of causal factors and controls - are channel conditions being formed by current flow regimes or are they a product of past conditions?
4- The condition the channel is likely to evolve to without a project. This often involves a strong reliance on engineering judgment.
5- A physical limitation on possible design variables due to water quality, construction area, and budget constraints.
6- Solutions that are possible and acceptable to the stakeholders.

Constraints: Determining project constraints is just as important as establishing objectives. An ideal stream restoration design might include a natural channel that has a vegetated bank and the capacity to carry the mean-annual flood. The channel might be free to migrate laterally and longitudinally down the valley. Such a design would preclude any 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 polluted sediment in the streambed or in the banks. In order to make sure these polluted sediments stay in place it may be necessary to stabilize the banks, preventing the natural channel migration process.

HYDROLOGY

Design Flows: 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. It is useful to identify several design flows in the restored channel. The ordinary low water or baseflow level is important for aquatic life. Between ordinary low water and ordinary high water vegetation can be supported on the channel banks. The bankfull channel elevation is often assumed to be geomorphically significant and is used to determine channel dimensions. Flood discharges are important in the design of channel structures and in establishing riparian vegetation on the floodplain.

The cross-sectional flow area at low discharges often defines the limiting biologic condition for aquatic organisms. Minimum depths are often given as a design goal. In gravel-bed streams it is often desirable to design the channel section at a minimum flow level so that fine sediment does not deposit. Analysis and design at minimum flow conditions may involve incipient motion and threshold techniques. Habitat features are often designed to function at baseflow conditions or at ordinary low water. However, these features must be designed to withstand a much larger flow so that they are not destroyed during high flow periods. Consideration should also be given to sediment inflow so that they are not buried by sediment and rendered ineffective.

Woody vegetation typically occurs 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 between the ordinary low water and ordinary high water elevations, especially for alluvial channels.

Bankfull stage is typically defined at a point where the width to depth ratio is at a minimum. A bankfull flow is often considered to be synonymous with channel-forming discharge and is used in initial determination of main channel dimensions. There are many fluvial geomorphological regime equations that relate drainage area or discharge, to 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. In this situation it may be appropriate to use the bankfull hydraulic conditions to design bank protection and to determine stability of habitat structures. However, when the floodplain is narrow or obstructed, channel velocities may continue to increase with rising stage. As a result, it may be appropriate to use a discharge greater than bankfull discharge to design channel features such as bank protection and habitat structures.

The floodplain or upland zone is typically found above the bankfull stage. This area is favorable to plants and animals that live on land that is rarely submerged. Incipient motion analysis and threshold methods may be appropriate to design features in this zone. If a project involves riparian plantings, it is advisable to assess the expected depths and velocities in this area in light of the tolerance of the proposed species. For all stream restoration projects, some consideration must be given to extreme flood flows such as the regulatory floodplain and the one-percent chance exceedance flood. Some analysis should be done to assess the impact of the stream restoration project on flood elevations in the floodplain. The stability requirements for the features used in the lower parts of the channel are typically defined around an extreme flood event. In addition, sediment continuity continues to be an issue during extreme events.

Channel-Forming Discharge Concept: A representative or channel-forming discharge may be appropriate for determining initial or preliminary design dimensions for a channel restoration project. The channel-forming discharge concept is based on the idea that 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.

Care must be exercised in applying the channel-forming discharge concept, 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. The existing channel therefore may represent a condition that accurately depicts present flow and sediment-transport conditions. Assigning a single value to this theoretical channel-forming discharge is problematic. 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 one and three years. The third is effective discharge. Bankfull discharge is the maximum discharge that the channel can convey without overflowing onto the floodplain. 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. 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. 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.

Range of Natural Discharge: Ultimately, channel stability will be determined not only by peak discharges but by the sequence and duration of the natural hydrograph. Thus, development of a flow duration curve or natural runoff hydrograph may also be required in the hydrologic study.

WATERSHED CHARACTERIZATION AND ASSESSMENTS

General: A systematic investigation of existing conditions in the watershed and the stream by personnel experienced in river hydraulics and geomorphology is a necessary part of stream restoration. Watershed assessments can be used to characterize existing stream conditions, assess trends, refine project goals, and, if necessary, to locate potential project reaches. Fundamentally, the objective of this investigation should be to formulate a sufficient understanding of watershed processes and to aid in the selection and design of project alternatives. Basic watershed characterization should include a landscape analysis followed by a field assessment. The effort required for each is dependent on project scale and purpose.

Landscape Analysis: The landscape analysis is a first-cut attempt to identify the causal factors, direct and indirect, and controls likely to be present in the study watershed and at a study site. Much of the landscape analysis can be accomplished in the office by reviewing old reports, maps, and aerial photos. Direct causes include not only obvious practices that alter planform, cross-section and grade such as levees, floodwalls, dam construction, and channelization; but also include items such as roads, bridges, pipe crossings, and vegetative removal in the riparian zone. Indirect causes include landuse changes that may result in increased runoff for a given storm event and in changes in sediment load.

In assessing an existing stream condition, it is also important to identify historical developments that may have affected or are still affecting channel morphology and stability. These effects can be anthropogenic and very recent, such as watershed development that has resulted in altered stream flows and sediment yields. Effects could have occurred within a longer time frame such as farming practices that may have resulted in significant sediment deposition. Streams in these watersheds can be still adjusting naturally to an aggraded condition by slowly downcutting. Causal effects could have occurred on a geological time frame such as in glaciated regions. The examination and review of geological information, local historical accounts, historical thalweg and cross section information, gage data, FEMA maps, biological monitoring, hydrologic modeling, watershed development and landuse patterns, and aerial photographs can be useful in this assessment. Recent gage data should be reviewed to determine if current conditions might be the result of a recent extreme event rather than long term and systematic instabilities.

Field Assessments: Once the dominant processes in the watershed are characterized, a site assessment should be conducted. Field reconnaissance is undertaken to gather data and make observations necessary to formulate an understanding of the processes and condition of the stream. It is critical that experienced personnel conduct this effort. The use of a study specific data collection technique can be helpful. It is often appropriate to create a field sheet that addresses issues specific to the project that may be present in the watershed. A variety of field assessment techniques are presented in the literature. The type of conditions that they can be used to assess and the level of effort they require vary greatly. 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 typically best made during low water conditions when there is minimal vegetation so the banks can be more readily examined. For safety and logistical reasons, this fieldwork is best accomplished by teams of at least two people. It is recommended that this minimal team include a biologist who is familiar with characteristics of the objective habitat and an engineer who is experienced in hydraulics, hydrology, geomorphology, and sediment transport. Inspections at bridge crossings should be treated with caution since the locations of bridges may not be characteristic of the stream as a whole. Bridges are frequently placed at constrictions and/or at bedrock outcrops. 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.

Generally, the following basic information should be collected:

1. Descriptions of the watershed land use, floodplain characteristics, channel planform and stream gradient.
2. Assessment of historical conditions. This can be obtained via interviews with knowledgeable landowners.
3. Measurements of low flow and bankfull channel dimensions and channel slope in critical reaches. Identification of terraces and active floodplains.
4. 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.
5. Descriptions of river bank profiles, bank materials and evidence of bank instability.
6. Descriptions and locations of point bars, pools, riffles, bed instability and evidence of sedimentation processes.
7. Observations of channel alterations and consequences to channel shape and size, potential watershed influences and evidence of stream recovery from previous impacts.
8. Descriptions of channel debris and bed and bank vegetation.
9. Preliminary stream restoration alternatives. Information should include size and extent as well as the identification of constraints such as access, utilities, and staging areas.

It is important to recognize the limitations of field assessments, including observer dependence, temporal limitations, and spatial limitations. Issues related to observer bias can be partially overcome with the consistent use of trained personnel. This will minimize relative differences between observations. Temporal bias can be minimized with an examination of historical records but these may be incomplete. Spatial bias can be partially controlled by having the field team walk a continuous reach of stream.

It is recommended that restrictions in construction area and access that can greatly influence project cost be identified. In addition, situations where the damage caused by construction access may exceed the on-site benefits of the project should also be identified.

Channel Typing: It is often useful to summarize the condition of the stream for use in communication and compiling observations. This can include the use of general typing or classification techniques that describe the existing condition of individual reaches. There are many techniques available that range in complexity and required effort. Channel "type" is a channel description based primarily on observation. The channel description may include parameters such as channel and floodplain geometry, bed and bank material, planform, vegetation, bedforms, evidence of aggradation or degradation, grade control, etc. Channel typing is an elementary level of stream classification, which uses generic terms. For instance, a stream may be typed as a meandering sand bed channel. Geomorphic channel classification involves the selection of a classification system and the categorization of a channel into a specific class based on factors and measurements such as dominant mode of sediment transport, entrenchment ratio, sinuosity, etc. Some of the most widely used classification systems are described in EM 1110-2-1418 (USACE 1994) and in the Federal Interagency Stream Corridor Restoration Manual (1998). Streams can also be classified by their biota, habitat conditions, base flow levels, and direct measures of water quality.

HYDRAULIC AND SEDIMENTATION DESIGN

General: In alluvial streams, the independent variables that drive the hydraulic design of the channel are discharge, sediment inflow and bank and bed-material composition. The dependent or design variables are width, depth, slope, and planform. The hydraulic design methodology presented herein is intended for cases where a historically stable channel is to be or has been realigned creating instability. It is also applicable where hydrologic and/or sediment inflow conditions have changed to a degree that the channel is currently unstable. If the existing channel is stable in the project reach, an attempt should be made to maintain the same channel geometry in the restored channel. In this case, the selected geometry should be analyzed to ensure that it can transport the incoming water and sediment load for the selected channel design discharge. 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.

This design procedure generates a preliminary channel geometry that can transport the incoming water and sediment load for the selected channel design discharge. The design philosophy is to use appropriate geomorphic principles combined with analytical equations for flow resistance and sediment transport to solve for the dependent design variables of width, depth, slope, and planform. Geomorphic principals that can be used with the analytical equations include analogy methods, hydraulic geometry, and the extremal hypothesis. Project constraints often narrow the range of feasible solutions. The long-term stability of the preliminary channel design is evaluated using a full range of discharges. Design adjustments may then be made to the channel design based on issues related to stability, flood effects and sedimentation.

Analogy methods: Analogy methods are based on the premise that conditions in one reach can be copied to another if the site and watershed conditions are the same. In the procedure outlined here, width, depth and slope are determined first. One variable (width or slope) is copied from a selected reference reach, and the remaining two variables are calculated using hydraulic resistance and sediment transport equations. The reference reach must be stable and alluvial and have the same channel-forming discharge as the project reach. The reference reach may be upstream or downstream from the project reach, or in a 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 in the reaches.

An alternative to the reference reach approach is to reconstruct the channel to a stable pre-disturbance width and planform. This is feasible if historical width and planform can be determined from mapping, aerial photos, and/or soil borings. This technique is generally not applicable if the watershed water and sediment runoff characteristics have changed over time.

Hydraulic geometry methods: Hydraulic geometry relations can be used to select a value for one of the dependent variables. 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 dependant 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 geomorphological trends. Data scatter is expected about the developed curve even in the same river reach. 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 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.

Calculation of the remaining unknown design variable: Once one of the dependent design variables (width,depth or slope) is determined, the other two variables should be calculated using one of several resistance and sediment transport equations available in the literature. The stable-channel analytical method in the U.S. Army Corps of Engineer hydraulic design package SAM may be used to determine the unknown dependent design variables. This method is based on a representative 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 method uses the Brownlie sediment transport and roughness equations for sand-bed streams and the Meyer-Peter and Muller sediment transport equation with the Limerinos bed resistance equation for gravel-bed streams. This procedure assumes a fully mobile bed. Details are available in the SAM users manual (Thomas et al 2000).

The stable channel analytical method in SAM produces a family of solutions for slope and depth for given widths for a specific discharge. 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 right-of-way limits, a maximum depth constraint might be imposed by flood control considerations. The valley slope would impose a maximum slope constraint. 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 stability curve. However, extensive field experience demonstrates that channels can be stable with widths, depths, and slopes different from extremal conditions.

Sediment Impact Assessment: Stream restoration projects are often designed using a single flow event and the sediment load transported by this event. This approach does not account for potential instability driven by the range of natural flow events. 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 and sediment supply. Mean annual sediment load from each reach is calculated by numerically integrating the annual flow-duration curve with a bed-material sediment rating curve. 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). The sediment impact assessment is a closure loop at the end of the design procedure to: 1) validate the efficacy of the restored channel geometry; 2) identify flows which may cause aggradation or degradation over the short term (these changes are inevitable and acceptable in a dynamic channel); and 3) recommend minor adjustments to the channel design to ensure dynamic stability 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.

Sediment Rating Curve Analogy Analysis: The sediment rating curve analogy analysis is a technique that can be used to give a qualitative answer to the project's performance for the full range of discharges when a flow-duration curve cannot be developed. 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. The sediment rating curve analogy analysis is suitable for a stream where the sediment supply is not limited, that is the stream is alluvial. It is not suitable for threshold streams. This qualitative technique does not require stream gage data or sediment gage data. The information that is required includes an estimate of the supply reach grain size distribution, an estimated range of peak flows, and the hydraulic characteristics of both the study and supply reaches. Hydraulic parameters can be determined from normal depth calculations, hydraulic modeling, or by using existing FEMA floodplain mapping. Peak flows can be estimated using regional regression curves or hydrologic modeling. Bed-material sediment transport capacities are calculated for flows 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. The more closely the sediment rating curve for the project reach matches the sediment rating curve for the supply reach, the less aggradation or degradation is expected.

Natural variability in cross-section shape: Variability in channel width and depth can either be allowed to develop naturally or can be part of the project design. Sand-bed streams have the ability to create natural variability in channel form rather quickly because they are characterized by significant bed-material sediment transport. Gravel-bed streams typically adjust much more slowly. Streams with very little bed-material movement may not adjust at all. If variability is to be included in the project design dimensions for cross-sections in riffles and pools can be obtained from stable reaches of the existing stream or from reference reaches. Thorne (1988) has provided morphologic relationships for channel width for a meandering sand-bed river. Other researchers have correlated variability to riparian and bank conditions. Analogy methods have also been used in the design of variability.

Planform: Planform sinuosity is determined from the calculated channel slope and valley slope. Remaining planform design parameters include the meander wavelength, an appropriate channel length for one meander wavelength, and the trace of the channel. Existing methods often rely on 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 non-availability of a reference reach, subtle but important fluvial, sedimentary or morphological differences between it and the study reach, or restrictions on the right-of-way, which may preclude the import of meanders with the amplitudes observed in the reference reach. Alternatively, meander wavelength can be determined using hydraulic geometry techniques. The most reliable hydraulic geometry relationship is typically wavelength versus 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. The channel trace may also be determined analytically using the sine-generated curve as suggested by Langbein and Leopold (1966). Finally, a string, cut to the appropriate length, can be laid on a map and fit to existing constraints and to the proper wavelength to form a meandering planform.

<u>In-stream structures</u>: Successful stream restoration often includes the use of bank protection, grade control, and habitat features. To restore a stream with physical habitat features resembling a natural stream, 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. Systems composed of living plant materials are often used in association with inert materials, such as wood or rock, and manufactured products. A significant flood event (normally no smaller than the 10-year frequency discharge) is used to size structures and compute scour depths. In addition, the quantity of water and its related hydroperiod largely determines what type of vegetation will grow in an area. The flexibility of these features depends on the project goals, tolerance for project change, and consequences of failure. Consideration is given to the effects that proposed features could have on flooding. For example, vegetation often increases boundary roughness, decreasing velocities, and increasing flood profiles. Additional design considerations include the level of risk that is acceptable, natural system dynamics, anthropogenic activities in the watershed, the construction time frame, existing infrastructure, desired speed of improvement, cost, and maintenance. The Corps has a wide variety of reports, technical notes, technical papers, and manuals that address different aspects of appurtenance design and usage.

SUMMARY

The stream restoration design procedure presented herein attempts to bring together geomorphic principles and traditional engineering methods. The combination of hydraulic geometry theory and analytical techniques can result in a reliable and cost-effective design, which is adapted to site-specific goals and conditions.

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A PRACTICAL METHOD OF COMPUTING STREAMBANK EROSION RATE

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Abstract: Accelerated streambank erosion is a major cause of non-point source pollution associated with increased sediment supply. A quantitative prediction of streambank erosion rate provides a tool to apportion sediment contribution of streambank sediment source to the total load transported by a river. A method for developing quantitative prediction of streambank erosion rates and examples of its implementation are presented. The prediction model presented utilizes a rational estimation, process-integration approach. A streambank erodibility index and calculated near-bank stresses are utilized in the prediction model. Streambank characteristics involving measurements of bank heights, angles, materials, presence of layers, rooting depth, rooting density and per cent of bank protection, are used to develop the streambank erodibility index. Measured data are converted to a normalization index for application for a wide range of channel sizes and types. Near-bank stress requires calculation of vertical velocity profiles and shear stress for subsequent distribution of energy calculations in the near-bank region.

The measured field values, converted to prediction indices, were tested against measured annual streambank erosion rates. The results of an analysis of variance performed on two independent data sets from two varied hydro-physiographic regions indicated a highly significant relation. Application in regions other than those used to develop the relations are also presented.

Applications in river and riparian management, stream channel stability analysis, streambank stabilization programs, river restoration, and sediment studies are presented. This model was also used to compare geologic erosion with anthropogenic sources and the consequence of riparian vegetation changes on streambank erosion rates. The model has particular advantages when used for stream channel stability departure analysis and sedimentTMDL's.

INTRODUCTION

The significance of streambank erosion processes that contribute sediment to the total annual sediment transport has often been overlooked or misunderstood. Most studies on sediment supply have been directed to surface erosion processes, which in many disturbed landscapes are the major sediment sources. Streambank erosion contributions were shown to be the majority of total sediment supply in the West Fork Madison River, Montana (Rosgen, 1973, 1976). Restoration work and subsequent bedload and suspended sediment measurements conducted by the author on the East Fork River, Colorado has shown that three miles of unstable, braided channel was contributing 49% of the total sediment yield of a 140 km^2 watershed. This study involved the comparison of total sediment yield measurements upstream versus downstream due to streambank erosion acceleration from willow removal. More recent studies in the loess area of the Midwest United States, indicated that streambank material contributed as much as 80% of the total sediment load eroded from incised channels (Simon et al, 1996). Streambank erosion varies from 1.5 m/yr on the Obion/Forked Deer drainages in West Tennessee (Simon, 1989), to 14 m/yr in the Cimmaron River in Kansas (Schumm and Lichty, 1963), 50 m/yr. In the Gila River, Arizona 100 m/yr on some reaches of the Toutle River, Washington (Simon, 1992). Recent programs by several Federal agencies including the Natural Resources Conservation Service and U.S. Fish and Wildlife Service, have been providing financial assistance to private landowners for riparian management and protection in an effort to; decrease bank erosion rates, reduce downstream impacts associated with increased sediment supply, help aquatic and terrestrial habitats and protect land from erosion.

The adverse consequence of increased streambank erosion results not only in accelerated sediment yields, but also to changes in stream channel instability and associated stream type changes. Stream types can evolve in over a wide range of scenarios from meandering to braided, to incised channels due to various processes (see evolution scenarios Rosgen, 2001 In Press, Interagency Sediment Conf.). These instabilities and consequential shifts in stream type not only produce higher sediment yields, but can degrade the physical and biological function of rivers.

PRINCIPLES

Streambank erosion can be traced to two major factors: stream bank characteristics (erodibility potential) and hydraulic/gravitational forces. The predominant processes of stream bank erosion include: surface erosion, mass failure (planar and rotational), fluvial entrainment (particle detachment by flowing water, generally at the bank toe), freeze-thaw, dry ravel, ice scour, liquifaction/collapse, positive pore water pressure, both saturated and unsaturated failures and soil piping. Hydraulic and gravitational forces occur within the soil mantle as well as within the water column of the stream itself. The velocity, velocity gradients, boundary shear stress, strong down-welling and up-welling currents in the near-bank region, back-eddy circulation and other flow mechanics also affect rates of erosion. Extensive research has been underway for some time dealing with failure types and mechanics and factor of safety calculations. Recent streambank mechanics and streambank stability analysis prediction has been published by Thorne (1982), Simon and Thorne, (1996), Darby and Thorne (1997), Thorne, (1999) and Simon, et al (1999). These process research studies need to be continued for us to better understand the complexities involved. The complexity of the quantitative consequence of each individual physical processes of erosion, however, has precluded reliable streambank erosion rate prediction.

GENERAL METHOD

This empirically derived, process-integrated-streambank erosion prediction model requires field practitioners to integrate rather than isolate individual streambank erosion processes. Streambank characteristics (susceptibility to detachment/collapse) were identified separate from near-bank velocity gradients and shear stress in the model. Erodibility and near-bank stress relations were established between measured field variables that were sensitive to a wide range of erosional processes. Numerical values were converted from the field measurements to a scaling factor of risk ratings. In addition to the streambank erodibility factors, measured vertical velocity profiles were obtained on numerous sites in order to evaluate velocity gradients and shear stress in the near-bank region. To test these relations, direct measurements of annual erosion rates were obtained using bank pins and bank profiles, compared with the field variables used to develop the indices of bank erosion hazard index (BEHI) and near-bank stress (NBS). Two separate hydro-physiographic regions were selected for independent study: the Lamar Basin in Yellowstone National Park, Montana and the Front Range of Colorado on the USDA Forest Service, Arapaho and /Roosevelt and Pike/San Isabel National Forests. These studies were carried out in 1987 and 1988 with the assistance of Park Service and USDA Forest Service personnel. Prior to snowmelt and stormflow runoff, erosion study sites were established for a wide range of BEHI and NBS ratings, then re-surveyed the following year. Relations were empirically derived between BEHI, NBS and measured annual streambank erosion rates. An analysis of variance was performed on each of the two regional, independent data sets to obtain levels of significance and coefficients of determination of predicted versus actual annual bank erosion rate. The model was tested in other regions for validation and subsequent potential applications by field practitioners.

MODEL DEVELOPMENT

Stream Bank Characteristics. Key streambank characteristics were identified that would be sensitive to the various processes of erosion in order to develop the BEHI rating. These streambank variables included: bank height ratio (stream bank height/maximum bankfull depth), ratio of rooting depth/bank height, rooting density, per cent surface area of bank protected, bank angle, number and location of various soil composition layers or lenses in the bank, and bank material composition. An expert system was used to transfer field observations of potential erodibility to relative ratings (Figure 1). Field experience from direct observations of streambank instability was used to document streambank conditions associated with active erosion and various modes of failures. The field measured variables assembled as predictors of erodibility (BEHI) were converted to a risk rating of 1-10 (10 being the highest level of risk). The risk ratings from 1 to 10 indicate corresponding adjective values of risk of very low, low, moderate, high, very high, and extreme potential erodibility (Figure 1). The total points obtained as converted from the measured bank variables to risk ratings are shown in Table 1. These relationships were established based on a catalog of field observations as opposed to a factor of safety analysis as described by Thorne (1999) and Simon, et.al. (1999). Since these factor of safety analyses were not related to measured erosion, the process-integration approach was used as an alternative to provide a linkage for the field practitioner to estimate annual bank erosion rate.

Near-bank velocity gradient and shear stress distribution. At selected measured stream bank erosion study sites, vertical velocity profiles, corresponding velocity isovels and velocity gradients were obtained. Velocity isovels are shown in Leopold et al (1964) and Rosgen (1996). The stream width was divided into thirds to apportion the shear stress in the near-bank (one third width) region compared to bankfull shear stress of the entire channel. Calculations of both velocity gradient and near-bank shear stress (ratio of near-bank shear stress/bankfull shear stress) were obtained. These measured velocity gradients and near bank stress values were then converted to a risk rating system from very low to extreme stress (Table 2).

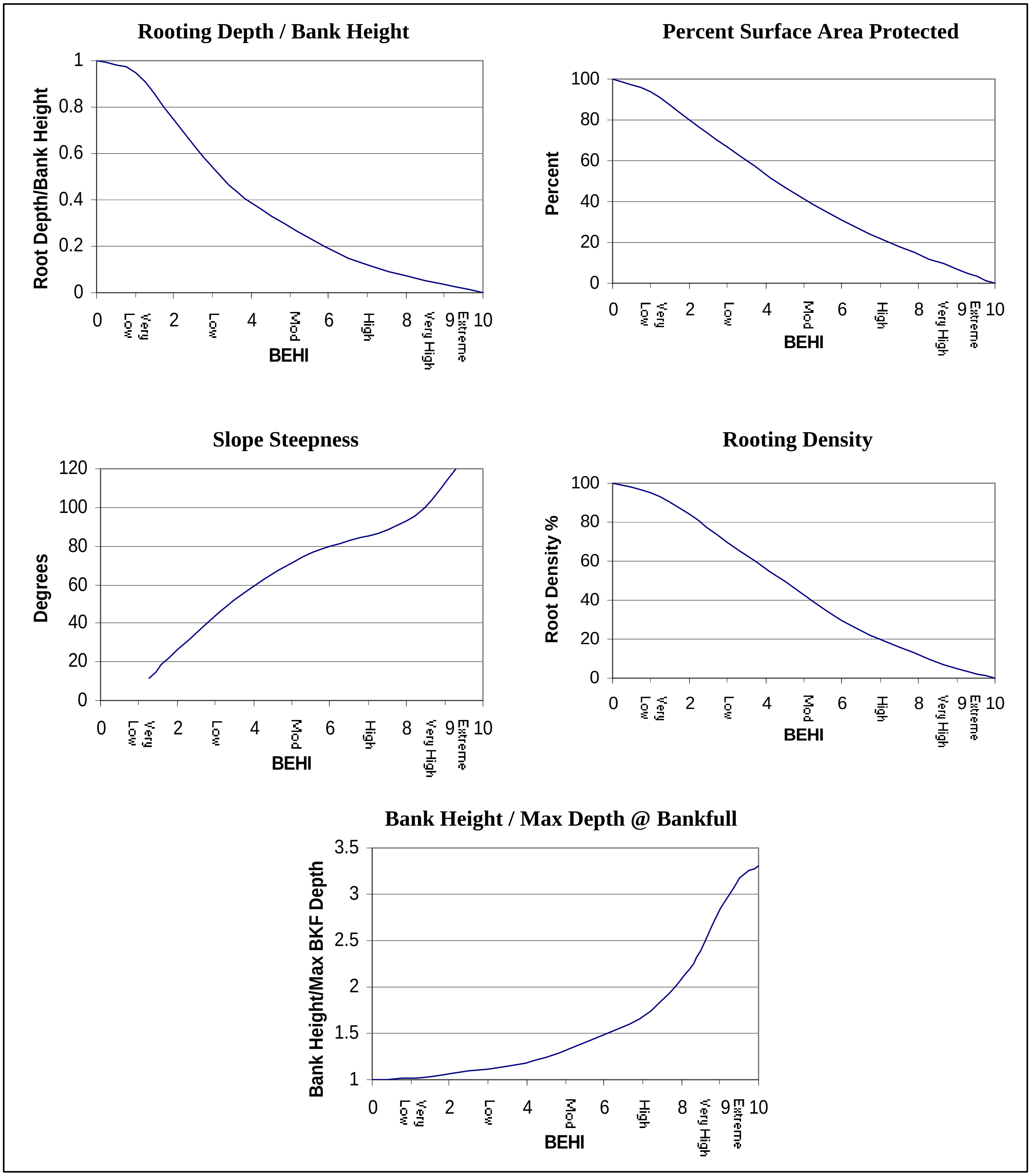

Figure 1. Example of streambank erodibility variables in relation to the Bank Erosion Hazard Index (BEHI)

Table 1. Streambank characteristics used to develop Bank erosion Hazard Index (BEHI)

Adjective Hazard or risk rating categories		Bank Height/	Root Depth/	Root	Bank Angle	Surface	Totals
		Bankfull Ht	Bank Height	Density %	(Degrees)	Protection%	
VERY LOW	Value	1.0-1.1	1.0-0.9	100-80	0-20	100-80	
	Index	1.0-1.9	1.0-1.9	1.0-1.9	1.0-1.9	1.0-1.9	5-9.5
LOW	Value	1.11-1.19	0.89-0.5	79-55	21-60	79-55	
	Index	2.0-3.9	2.0-3.9	2.0-3.9	2.0-3.9	2.0-3.9	10-19.5
MODERATE	Value	1.2-1.5	0.49-0.3	54-30	61-80	54-30	
	Index	4.0-5.9	4.0-5.9	4.0-5.9	4.0-5.9	4.0-5.9	20-29.5
HIGH	Value	1.6-2.0	0.29-0.15	29-15	81-90	29-15	
	Index	6.0-7.9	6.0-7.9	6.0-7.9	6.0-7.9	6.0-7.9	30-39.5
VERY HIGH	Value	2.1-2.8	0.14-0.05	14-5.0	91-119	14-10	
	Index	8.0-9.0	8.0-9.0	8.0-9.0	8.0-9.0	8.0-9.0	40-45
EXTREME	Value	>2.8	<0.05	<5	<119	<10	
	Index	10	10	10	10	10	46-50

For adjustments in points for specific nature of bank materials and stratification, the following is used:
Bank Materials: Bedrock (very low), Boulders (low), cobble (subtract 10 points unless gravel/sand>50%, then no adjustment), gravel (add 5-10 points depending on % sand), sand (add 10 points), silt/clay (no adjustment).
Stratification: Add 5-10 points depending on the number and position of layers.

Table 2. Velocity gradient and near-bank stress indices

Bank Erosion Risk Rating	Velocity gradient	Near-bank stress/shear stress
Very low	Less than 0.5	Less than 0.8
Low	0.5 -1.0	0.8 -1.05
Moderate	1.1 -1.6	1.06 -1.14
High	1.61 - 2.0	1.15 - 1.19
Very High	2.1 -2.4	1.20 -1.60
Extreme	Greater than 2.4	Greater than 1.60

RESULTS

Yellowstone Park, Montana and Front Range Colorado Data. The methods and results presented here to predict annual streambank erosion rate represent an approach different and more quantitative than previous studies. The rate of erosion was measured in distance of bank recession per year. The measured annual, lateral erosion rate for 49 separate sites are plotted for the Front Range Colorado and for 40 sites in the Lamar River Basin Montana, Figure 2 and Figure 3, respectively. An analysis of variance (SAS Users Guide, 1989) was used to assess the relationship between bank erosion hazard index (BEHI) and Near-Bank Stress (NBS) in the prediction of erosion rate. There are significant differences in two or more of the means (p=.0001) in both cases for both parameters, thus both BEHI and NBS are highly significant predictors of bank erosion rate. Mean BEHI values for the highest and lowest NBS indices (X axis) were used to locate and plot the four BEHI models for their corresponding erosion rate as shown in Figures 2 and 3. The models plotted in Figure 2 and 3 represent the means derived from analysis of variance and are used to graphically predict bank erosion rate from field level data compilations. "Site" was a significant parameter in the analysis indicating the Montana and Colorado data sets could not be aggregated. Coefficients of determination, or r^2 values were 0.92 and 0.84 for the Colorado and Yellowstone data, respectively. Since the Colorado and Montana data could not be aggregated, it is necessary to empirically develop these relations unique for a given geology. For example, loess soils of the Mid-Western United States would yield much higher erosion rates for the same BEHI and NBS ratings than the curves presented in Figure 2 and Figure 3. Thus, it would require field practitioners to establish the local curves in a similar fashion as was initially completed in Montana and Colorado.

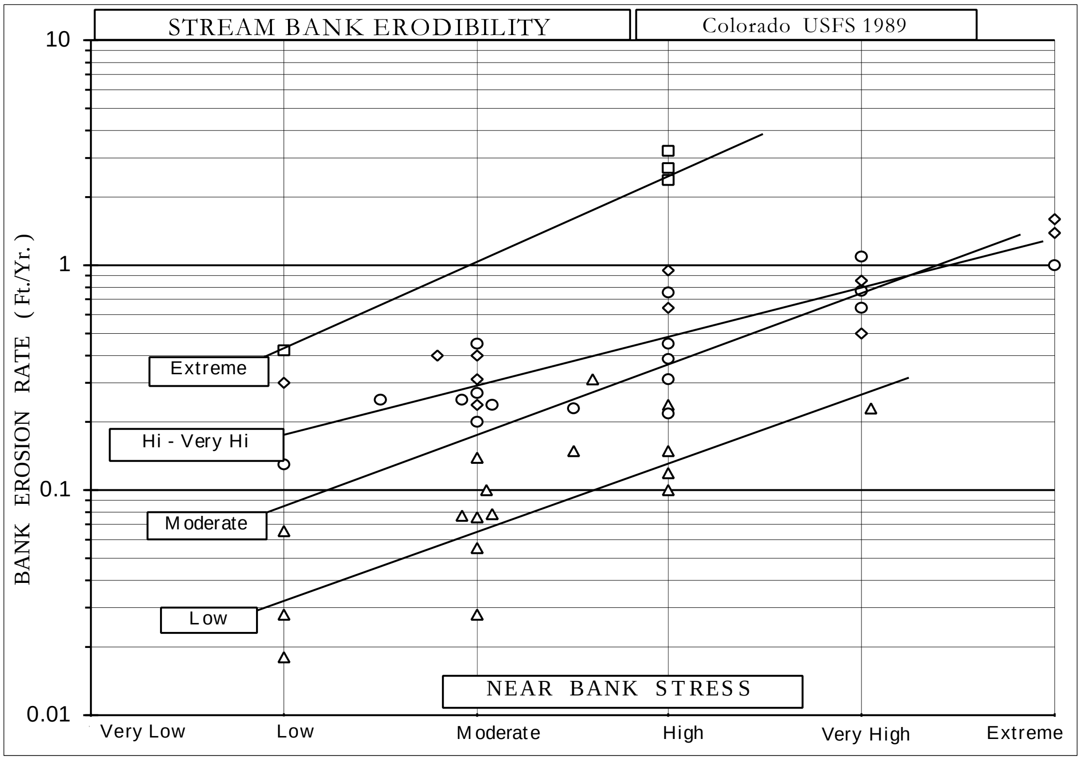

Figure 2. Relation of Streambank Erodibility (BEHI), Near-Bank Stress (NBS) and measured streambank erosion rates for the Front Range of Colorado,USFS data, 1987 to 1988

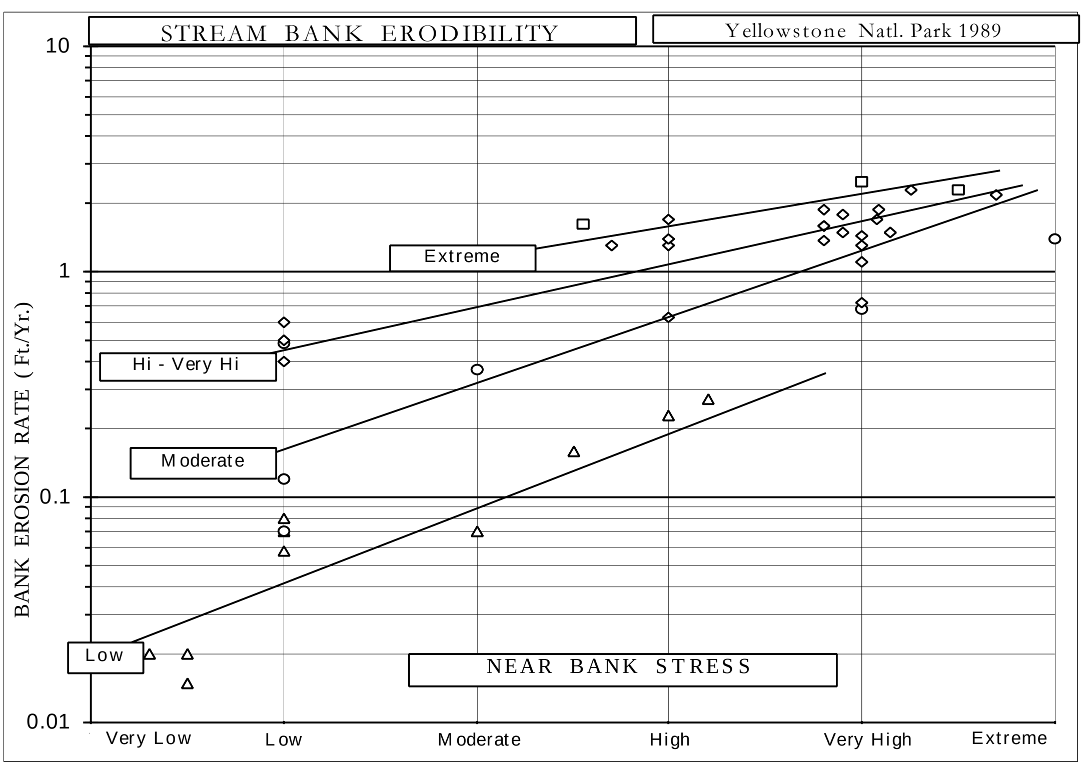

Figure 3. Relation of Streambank Erodibility (BEHI), Near-Bank Stress (NBS) and measured streambank erosion rates for the Lamar River and Tributaries from 1987 to pre-fire, 1988 (Yellowstone N.P.), (from: Rosgen, 1996)

Subsequent Research. The initial results prompted continued research of model prediction to measured annual streambank erosion rates. Research was conducted in North Carolina by the combined efforts of North Carolina State University and personnel of the USDA Natural Resources Conservation Service, (Harmon and Jessup, personal communication, 1999). The results of these studies are shown in Table 3. The data from North Carolina plots quite close to the Colorado data set (Figure 2). This may be due to the similar alluvial composite bank type of their study sites with the Colorado sites.

Table 3. Streambank study results on Mitchell River, North Carolina (Harmon and Jessup, 1999).

Bank Erodibility Hazard (BEHI)	Near-Bank Stress (NBS)	Predicted Streambank Erosion (Colorado curve) m/yr	- ft/yr	Observed Streambank Erosion m/yr	- ft/yr
Moderate	High	0.012	0.38	0.09	0.30
Moderate	Extreme	0.45	1.5	0.21	0.70
High	Extreme	0.76	2.5	0.85	2.8
Extreme	Extreme	4.27	14.0	3.35	11.0

Research on the Illinois River in Oklahoma (Harmel, et al 1999) found that streambank erosion rate increased as the bank erosion hazard increased. The near-bank stress combined with the streambank erosion prediction indices relationship, however showed a poor correlation. In this study, cross-sectional area ratios were used rather than either near-bank shear stress or velocity gradient. Our studies have shown that either velocity gradients or shear stress ratios predict much better than the cross-sectional area ratio, thus users should not apply the latter for near-bank stress. As a result of the effort by Harmel, et al (1999), we may want to partition this application by soil type. Their poor correlation may be also due to fact that the flows generating the measured erosion rate were four times the bankfull stage. The data presented for the Colorado and Montana data sets are associated with flows at or near the bankfull discharge. Complexities of streambank mechanics and hydraulics during suchfloods, may create such differential rates of erosion making predictions very difficult.

Streambank erosion studies were conducted in 1998 and 1999 on a C4 stream type reach on the Weminuche River in Southwestern Colorado that had been subjected to poor grazing practices. Predicted values compared to measured values of streambank erosion for various BEHI and NBS ratings using the relations in Figure 2 are shown in Table 4 and summarized in Figure 4. Horizontal placed bank pins and elevation rod readings were taken from the toe pin to profile the bank before and after runoff. Cross-sections are also obtained to determine vertical and horizontal stability changes concurrent with the streambank erosion study. The C4 stream type is associated with aterraced alluvial valley with streambanks composed of a composite mixture of fine alluvium, sand, gravel and cobble. The riparian type is a willow/grass type, with reaches converted to a grass/forb riparian plant community. The research on the Weminuche shows encouraging results that field data collected at low flow utilizing this process-integration model can provide comparable results to measured values.

Selection of representative curves to be used for erosion rate prediction for corresponding BEHI and NBS is based on the river type and materials characteristic of the empirically derived data. For example, the Weminuche River resembles the meandering alluvial stream types in Colorado, thus, Figure 2 was used. However, the studies on the East Fork San Juan River, a D4 (braided channel) mostly resembles the braided river of the Lamar River and tributaries, thus, the relation in Figure 3 was used to predict and compare erosion rate on this D4 stream type.

Table 4. Predicted values versus measured streambank erosion rates for reaches of the Weminuche River, Southwestern Colorado.

Cross-section location	Bank Erosion Hazard Index (BEHI)	Near-Bank Stress (NBS)	Predicted erosion rate m/yr.- (ft./yr.)	Measured erosion rate m/yr.- (ft./yr.)
25 + 62	Very High	Extreme	0.457 - (1.5)	0.481 - (1.58)
27 + 15	Very High	Very high	0.268 - (0.80)	0.335 - (1.1)
40 + 26.5	Very high	Moderate	0.055 - (0.18)	0.064 - (0.21)
41 + 00	Extreme	Moderate	0.335 - (1.1)	0.427 - (1.4)
44 + 25	Low	Very High	0.79 - (0.26)	0.091 - (0.3)

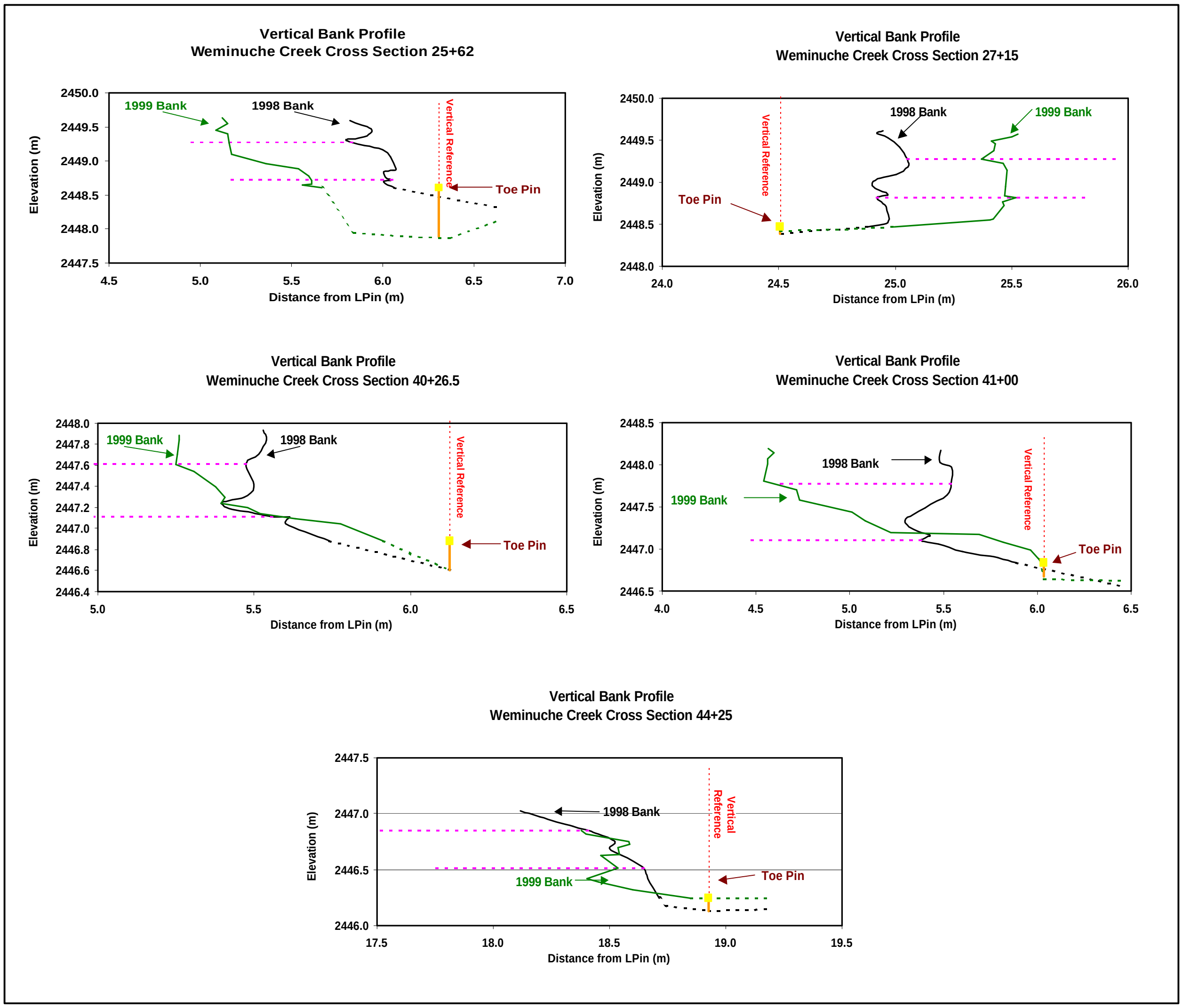

Figure 4. Streambank profiles on the Weminuche River Study – Colorado, showing streambank erosion rate for several locations during one runoff season, (1998-1999). Streamflows included a bankfull event.

A streambank erosion study from 1999-2000 on the braided (D4 stream type), East Fork of the San Juan River in Southwestern Colorado showed close agreement to the relations in Montana (Figure 3) due to the similarity of the braided (D4) stream type and relatively coarse river alluvium. The prediction and subsequent annual measurements were made by advanced level students of the Wildland Hydrology Research Institute and Educational Center for River Studies in Pagosa Springs, Colorado. The results are shown in Table 5.

Table 5. Predicted versus actual measured streambank erosion rates for braided reach of East Fork San Juan River.

Bank Erosion Hazard Index (BEHI)	Near-Bank Stress (NBS)	Predicted Streambank Erosion (Yellowstone)		Measured Streambank Erosion	
		M/year	Ft./year	M/year	Ft./year
Extreme	Extreme	0.85	2.8	0.73	2.40
Extreme	High	0.55	1.8	0.59	1.95
Moderate	High	0.19	0.62	0.22	0.73
Low	High	0.06	0.20	0.06	0.20
High	Low	0.14	0.45	0.12	0.40
High	Low	0.14	0.45	0.14	0.47

APPLICATIONS

A particular need in watershed management is to determine the volume, size and source of sediment. Once a relationship between BEHI and NBS is established with corresponding measured bank erosion rates, inventories of bank conditions along extensive reaches of rivers can be obtained. Potential lateral erosion rates corresponding to BEHI and NBS ratings, multiplied times bank height, times the length of similar conditions can produce volumes/year of sediment introduced to the stream from streambank erosion processes. The size of introduced sediment is also important for predicting channel response. This tool is also useful to provide a rapid inventory to assist in channel stability evaluation, assess priorities for restoration and provide information for riparian habitat management recommendations. Clean sediment TMDL's can also benefit from a quantitative assessment of potential sediment supply from streambank erosion, leading to mitigative measures to reduce accelerated sediment supply from this source.

The potential reduction in streambank erosion can be shown using effectiveness monitoring by designing restoration methods that decrease BEHI and NBS ratings and their corresponding annual erosion rate. Such monitoring as carried out in Southwestern Colorado on Turkey Creek and the Weminuche River respectively involved an upstream/downstream comparison of measured bank retreat rates. Erosion rates showed a reduction from 0.128 m/yr, and 0.55 m/yr. to virtually zero following post-restoration runoff. Natural stable alluvial streams with both BEHI and NBS ratings of very low have negligible rates of erosion. Reductions in tons of sediment/year can provide verification of the effectiveness of reducing sediment supply from restoration efforts in order to satisfy restoration objectives as well as meeting TMDL's established by individual states to comply with the Clean Water Act requirements.

Streambank erosion studies were conducted by the author on Wolf Creek in Southwestern Colorado to determine the results of spraying willows on a C4 stream type (a gravel bed, meandering, low gradient alluvial channel with awell developed floodplain. Accelerated streambank erosion occurred due to a conversion from willow/grass to grass/forb composition and stream channel instability followed, converting a C4 stream type to a D4 stream type (gravel bed, braided channel). The BEHI and NBS ratings on the C4 stream type immediately above the sprayed areas were low/low, respectively. Using Figure 2, the predicted streambank erosion rate of .0091 meters/year (.03 feet/year) was compared to the measured values of .0061 meters/year (.02 ft./year). The sprayed reach immediately downstream that initially was the same C4 streamtype, had BEHI and NBS ratings of very high/extreme, respectively. The predicted rate of erosion was 0.457 meters/year (1.5 feet/year) compared to the measured rate of 0.597 meters/year (1.96 feet/year). The model closely predicted a nearly three orders of magnitude increase in erosion rate as a consequence of spraying willows that converted the riparian type to a grass/forb plant community. During major floods on this reach 18.3 meters (60 feet) of erosion occurred during a three-year period in the sprayed reach compared to 0.012 meters (.04 feet) in the undisturbed C4 stream reach. The excessive land loss that increased sedimentation could have been prevented if the organization responsible for the spraying would have been able to predict the adverse consequence of streambank erosion, associated channel instability and eventual change in stream type from meandering (C4) to braided (D4).

An application that separated natural geologic erosion rates from anthropogenic helped provide quantitative prediction of the consequence of riparian vegetation change. For example, in the winter range of the Lamar valley in Yellowstone National Park, riparian vegetation composition was changed from a willow/alder/grass community to a grass/forb community due to severe browsing utilization in the winter range by elk and buffalo (Kay, 1990). Streambank erosion rates were measured on a reference reach or "control" upstream of the winter range on the same river, on the same stream type, the same bank stratigraphy and for similar streamflows in the same runoff season. The comparison of the upstream reach (good riparian vegetation condition of willows) compared to downstream reach (poor riparian condition of grass/forbs) indicated an erosion rate increase over geologic by three orders of magnitude. The extent of this accelerated streambank erosion affected many miles of stream and associated stream channel instability in the winter range of the Lamar valley (Rosgen, 1993). As shown in other studies, a conversion of riparian plant community from a predominantly cottonwood/willow to grass/forb on C4 stream types results in several orders of magnitude increase in annual streambank erosion rate. Floods particularly do extensive damage as these streams become "set up" for failure. Conversion of stream type due to the accelerated streambank erosion initiated an evolutionary shift from a C4 (meandering) to D4 (braided) stream type that presently exists within the winter range of the Lamar River and many of its tributaries. These same stream type conversions observed on the

Lamar River have been observed on many other heavily grazed riparian communities, including the East Fork San Juan River, Weminuche River, and Wolf Creek, Colorado.

CONCLUSIONS

The use of this process-integration approach to predict annual streambank erosion associated with normal high flow, shows excellent promise for management. Stratification by geologic and soil types should be accomplished to establish a family of curves for various geologic and hydro-physiographic provinces. Once a quantitative relationship is obtained, mapping changes in the BEHI and NBS ratings can be used to estimate consequence of change in locations beyond where the measured bank erosion data is obtained. Since streambank erosion measurements are very time consuming, extrapolation of these relations can extend the application and effectiveness of river assessments.

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A STREAM CHANNEL STABILITY ASSESSMENT METHODOLOGY

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ABSTRACT: Various definitions of stream channel stability are presented including "the natural stable channel", the graded river, dynamic equilibrium, and regime channels, and a quantitative assessment methodology is presented that distinguishes between stability states. The assessment procedure involves a stream channel stability prediction and validation methodology on a hierarchical framework. The stream channel stability method develops field-measured variables to assess: 1) Stream state or channel condition variables, 2) Vertical stability (degradation/aggradation), 3) Lateral stability, 4) Channel patterns, 5) Stream profile and bed features, 6) Channel dimension factor, 7) Channel scour/deposition (with competence calculations of field verified critical dimensionless shear stress and change in bed and bar material size distribution), 8) Stability ratings (modified Pfankuch method) adjusted by stream type, 9) Dimensionless ratio sediment rating curves by stream type and stability ratings, and 10) Selection of position in stream type evolutionary scenario as quantified by morphological variables by stream type to determine state and potential of stream reach.

The stability assessment is conducted on reference reach (stable) reaches and a departure analysis is performed when compared to an unstable reach of the same stream type. The assessment procedure utilizes various hierarchical levels for prediction and subsequent validation. Changes in the variables controlling river channel form, primarily streamflow, sediment regime, riparian vegetation, and direct physical modifications can cause stream channel instability. Separating the difference between anthropogenic versus geologic processes in channel adjustment is a key to prevention/mitigation/restoration of disturbed systems

The adverse consequence of stream channel instability (dis-equilibrium) is associated with increased sediment supply, land productivity change, land loss, fish habitat deterioration, changes in both short and long-term channel evolution and loss of physical and biological function.

INTRODUCTION

<u>Definitions</u>: Within the scientific community, the terms "channel stability", "equilibrium", quasi-equilibrium and "regime channels" evoke a deluge of various interpretations. Imagine the quantitative inconsistency of the field observer in trying to implement a stream channel stability assessment procedure with which there is not common agreement on what is meant by the term? Thus, it is not uncommon for journey-level professionals working with rivers to disagree on a consistent working definition of what constitutes a stable river, even though they often use the term "channel stability". A review of the literature provides insight into previous interpretation of terms, that all appear to be synonymous, or at the least, have a common thread of similarity. Davis (1902), defined a "graded " stream as the condition of "balance between erosion and deposition attained by mature rivers". Mackin (1948), as reported by Leopold et al (1964), defined a graded stream as "one in which, over a period of years, slope is delicately adjusted to provide, with available discharge and with prevailing channel characteristics, just the velocity required for the transport of the load supplied from the drainage basin. The graded stream is a system in equilibrium; its diagnostic characteristic is that any change in any of the controlling factors will cause a displacement of the equilibrium in a direction that will tend to absorb the effect of the change." The controlling factors described by Leopold et al (1964) were width, depth, velocity, slope discharge, size of sediment, concentration of sediment and roughness of the channel. If any one of these variables were changed it sets up a series of concurrent adjustments of the other variables to seek a new equilibrium. The central tendency of rivers to seek a probable state was described by Leopold (1994). Strahler (1957) and Hack (1960), used the term "dynamic equilibrium" referring to an open system in a steady state in which there is a continuous inflow of materials, the form or character of the system remains unchanged. Equations showing river variables as a function of discharge were derived by Leopold and Maddock, (1953), and by Langbein, (1963). These hydraulic geometry relations described adjustable characteristics of open channel systems in terms of independent and dependent variables in quasi-equilibrium (not aggrading nor degrading). Streams described to be "in regime" are synonymous with "stable channels" and equations describing three dimensional geometry of stable, mobile gravel-bed rivers were presented by Hey and Thorne (1986). Additional equations and discussion on stable river morphology were presented in Hey (1997). Regime channels, as discussed by Hey (1997) allows for some erosion and deposition but no net change in dimension, pattern and profile for a period of years. The following definition of stream channel stability was presented by Rosgen (1996): "is the ability of a stream, over time, in the present climate, to transport the sediment and flows produced by its watershed in such a manner that the stream maintains its dimension, pattern and profile without either aggrading nor degrading". Processes of stream channel scour and or deposition have to occur in a natural stable channel, but over time, if this leads to degradation or aggradation, respectively, then the stream would not be stable. This definition summarizes many of the key points previously presented in the literature. This definition is predictable and verifiable, and as such, was used in the development of the stream channel stability assessment methodology.

PRINCIPLES

River instability needs to be evaluated on spatial and temporal scales. It is also critical to recognize natural geologic erosion and transport mechanics versus anthropogenic influences. Following major floods, due to requirements to provide flood damage restoration plans, the author studied alluvial gravel-bed streams on slopes less than 0.02 where the pre and post-flood morphological variables were similar. Other reaches, however, that were in poorer stability condition prior to the flood, received major damage by the same flow. The stable rivers became reference reaches where data were collected on dimension, pattern, profile and channel materials. The1984 Lawn Lake flood in Colorado inundated Fall River, a C4 stream type (for stream type descriptions see Rosgen, 1994, 1996) that was in a stable meandering pattern. The extensive sediment load and corresponding "flood of record" did not create instability. The stream maintained its dimension, pattern and profile and did not aggrade nor degrade. Accumulations of sand occurred in the channel and within a few years the sand was routed through without the net effect of aggradation. This stream is but one of many examples where the author has field evidence where post-flood instability did not occur, even though these streams had potentially erodible material in their bed and banks. Reference reaches such as this become a blueprint of the variables associated with stable natural channels. Field and photographic evidence of channel change over time is an excellent reference procedure. Selection of the reference reach involves collection of such evidence. Descriptions and applications of the reference reach methodology are described in Rosgen (1998).

Stream channels that have been improperly managed and have poor riparian vegetation are subjected to accelerated streambank erosion and corresponding channel adjustments leading to instability. An example of instability that occurred due to willow removal on a C4 stream type on the Weminuche River in Southwestern Colorado is dramatic not only for the magnitude of change, but the consequence of change, as well. The details of the combined effects of willow spraying on stream channel instability and changes in dimension, pattern and profile for this reach are summarized in Rosgen (1996). The consequence of a wide range of stream channel instability can be described and quantified through an evolution of stream types (Figure 1). The evolution sequence that ensued on the Weminuche River due to channel adjustment following disturbance, created a change in morphological stream types that is associated with sequence category #3 in Figure 1. The conversion changed the pre-disturbance C4 stream type (gravel-bed, meandering channel with a floodplain), to D4 (braided), to G4 (incised gully due to avulsion), to an F4 (entrenched, meandering channel) and was widening to eventually re-establish a C4 stream type, but at a lower elevation. Every tributary was rejuvenated due to the change in local base level, which created a tremendous increase in sediment supply and transport and caused the water table to drop in the meadow, decreasing productivity. Thus, the consequence of spraying the willows and induced stream channel instability was associated with major loss of: land, vegetation productivity, fish habitat, visual values and loss of ability to handle future floods. Increased sedimentation, both on-site and downstream occurred. Many other evolutionary scenarios induced by channel instability and associated channel adjustment can occur. The author has observed at least eight separate evolutionary scenarios as shown in Figure 1. One challenge in stability assessment is to determine the evolutionary state and sequence of the stream. The *cause* of the instability is as important to understand as well as the *consequence*.

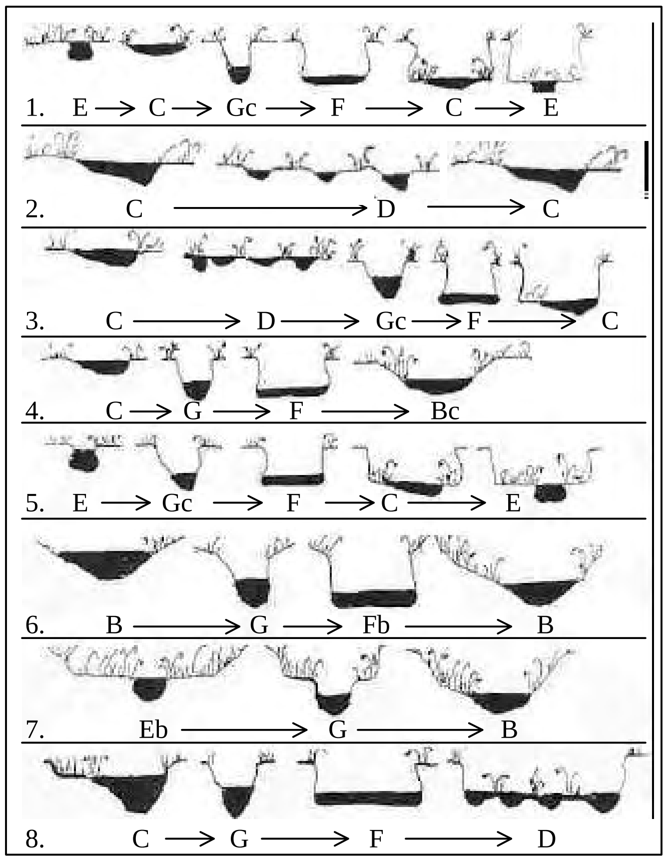

Figure 1. Various Stream Type Evolution Scenarios

OBJECTIVES

To prevent and or to correct stream channel instability, it is imperative to understand the mechanisms causing the shift in morphological variables and stability indices. The diversity of opinion has made it difficult to conduct consistent quantitative river stability assessments. It is not uncommon to have five individuals all "trained in these matters", simultaneously standing on the same bank of a river, having five divergent and conflicting opinions. Unless there are documented measurements, coupled with consistent, quantitative indices of stability, these subjective opinions will persist. Understanding of these complex processes can only come with a program of detailed measurements so that observations, stability indices and field assessment techniques can become effective. To meet this objective, the author set up a river inventory hierarchy (Rosgen, 1996), (Figure 2). This would allow an assessment at various levels appropriate to the level of inquiry. All 4 levels are used initially, until quantitative relations are established with the prediction methods. Initial stratification is accomplished at both levels I and II. This is not done to determine stability, but to stratify the reach by valley and stream type. Reference reach data is also obtained from adjacent stable reaches of the same valley and stream type. Reference reaches do not have to be pristine or relic sites, but meet the criteria of a stable river. Prediction of stability is made at level III, the "state" or condition level. Level IV is the validation inventory that requires the greatest level of measurement detail over a longer time period. For example, one may estimate vertical stability or bank erosion rate at level III, however permanent cross-sections are re-measured following runoff to verify bed elevation shifts, and erosion pins/toe pins are established at level IV to verify the actual erosion that occurred. This design allows prediction model validation at level III, thus, the prediction model can be extrapolated without the need to always accomplish level IV. Since these assessments involve large areas and many miles of river, this approach was designed to provide a prediction methodology with some credible validation.

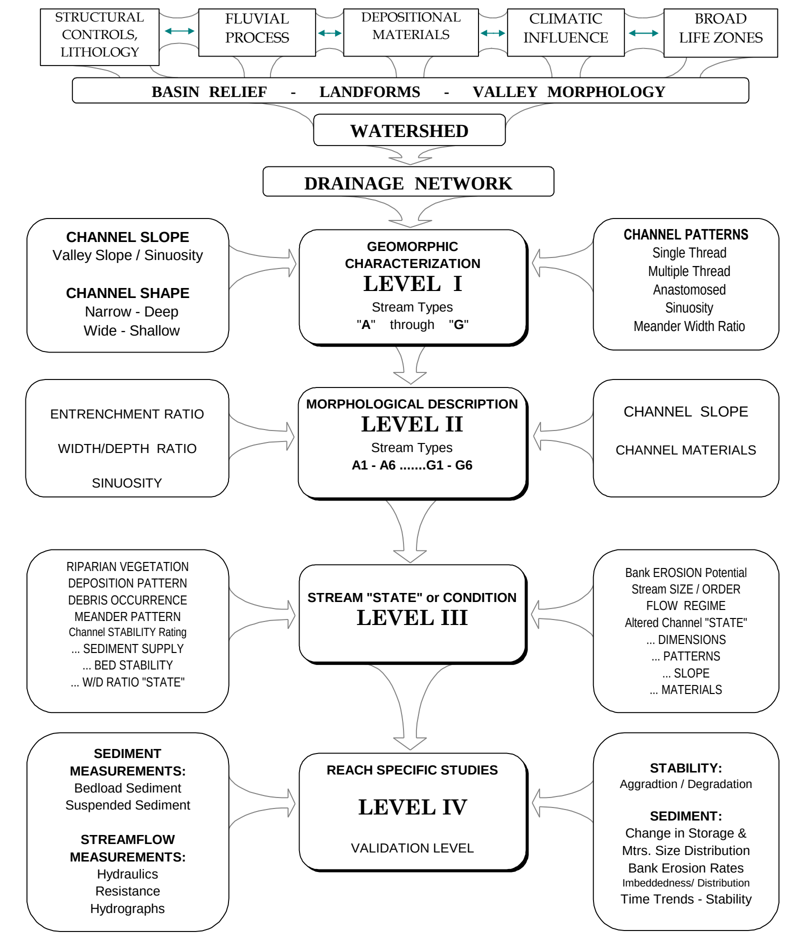

Figure 2. The Hierarchical River Inventory stratifying stability prediction and detailed river measurements by valley types and stream types for various levels of inquiry

METHODOLOGY

This section of methodology is meant to be a sequence of suggested steps for the field practitioner to use in reaching final conclusions and making recommendations for management and/or restoration. The stream channel stability assessment methodology is broken into the following ten major categories. Based on field inspection and measurements the categories of assessment are applied to the reference reach, as well as for impacted reaches. This provides a consistent comparative analysis of departure and assists in selecting evolutionary shifts in stream type and associated dimensionless sediment rating curves. A general summary of stability ratings and interpretations are included at the end of these categories of assessment.

1) Stream Channel Condition or "State" Categories: Determine condition categories from field inspection and measurement of stream channel condition characteristics. Specific categories are evaluated and documented based on the criteria for each variable. Detailed descriptions and examples for each category are presented in Chapter 6 (Rosgen, 1996) which will help completing these assessments. The seven categories and associated variables evaluated are: a) Riparian vegetation, (composition, density, and potential, climax riparian communities); b) Sediment deposition patterns (8 patterns); c) Debris occurrence (includes large woody debris); d) Meander patterns (8 patterns); e) Stream size/Stream order; f) Flow regime (perennial, ephemeral, intermittent, subterranean, snowmelt, stormflow, rain-on-snow, spring-fed, glacial-fed, tidal, diversions, and reservoir regulated, and; g) Altered states due to direct disturbance (dimension, pattern, profile and materials such as, channelization, straightening, levees, concrete, rip-rap, etc.). These seven major condition states provide insight into specific characteristics of the reference reach, as well as the stream type being assessed.

2) Vertical Stability/Degradation/Aggradation: From field measurements of bank height and entrenchment ratios and documented observations of excessive erosion and/or deposition, determine vertical stability of the stream reach. The degree of incision involves a measurement of bank height ratio (Table 1). It is measured as the ratio of the *lowest* bank height of the cross-section divided by maximum bankfull depth. For example a stream could be incising and not yet abandoned its floodplain or flood-prone areas. Bank height ratios of 1.2 and 1.3 are characterized by both streambanks eroding as the bank height is often below the rooting depth of the riparian vegetation. To determine if the stream has incised to the extent that the stream has abandoned its floodplain is determined by the entrenchment ratio, which indicates vertically containment. The entrenchment ratio is calculated by first determining the elevation of the flood-prone area as measured at twice the maximum bankfull depth. The floodprone area width at this elevation is then divided by the bankfull width. If the Entrenchment ratio is less than 1.4 (+ or- 0.2), the stream is entrenched (Rosgen, 1994,1996). Additional indicators of incision/degradation are: both left and right stream banks actively eroding, depositional features are being scoured, decrease in width/depth ratio corresponding with increase in bank height ratio, and mobilization of largest size D-100 of bed material (see category 8). The aggradation category is determined from a summary of the depositional patterns, coarse deposition on floodplains and very high to extreme width/depth ratios. Longitudinal profiles of the reach showing elevations of the bed, water surface, bankfull and *lowest* bank height indicate if the incision is advancing downstream or if a head-cut is advancing from the downstream direction. Profiles and cross-sections should be permanently monumented and read annually to verify the prediction of vertical stability.

Table 1. Conversion of bank height ratio (degree of incision) to adjective ratings of stability

Stability Rating	Bank Height Ratio
Stable (low risk of degradation)	1.0 - 1.05
Moderately unstable	1.06 - 1.3
Unstable (high risk of degradation)	1.3 - 1.5
Highly unstable	> 1.5

3) Lateral Stability: Determine the degree of lateral containment (confinement) and potential lateral accretion. The categories used for lateral stability are: a) Meander width ratio (degree of confinement) and b) Streambank Erosion Hazard Index (BEHI) and Near-Bank Stress (NBS) (see Rosgen, 1996 and 2001, In Press). Meander width ratio is the meander belt width (lateral containment of the channel within its valley) divided by bankfull channel width. Values of meander width ratio by stream type are shown in Rosgen (1996, p.4 -9). Some streams can be confined, but not entrenched. This provides insight into channel adjustment processes by stream type and degree of confinement. Annual, lateral streambank erosion rates are multiplied times the bank height and stream length of specific BEHI and NBS ratings along the reach. These values are converted to tons/year in order to apportion sediment supply sources. Many miles of stream can be evaluated using this method of prediction. Level IV data involves installing toe pins and cross-sections to accurately measure streambank erosion rates/lateral accretion. This helps validate the model or revise estimates and better reflect actual rates. The validation work can also be used for effectiveness monitoring prior to and following restoration and/or streambank stabilization.

4) Channel Pattern: Measure meander width ratios (meander length/bankfull width), ratio of radius of curvature/bankfull width, sinuosity, meander width ratio (belt width/bankfull width), arc length and arc angle. Convert all values to dimensionless ratios for comparative purposes. Additional assistance can be provided in assessing channel pattern categories as shown in Chapter 6, Rosgen, (1996). Changes in pattern are compared using dimensionless ratios when the reference reach data for the same valley and stream type may be of a different size. Channel adjustment due to instability can often be interpreted from these variables such as accelerated down-valley meander migration and excessive near-bank stress due to ratios of radius of curvature/width less than 2.0. Level IV data utilizes aerial photo time/trends and cross-sections showing down-valley meander migration.

5) River Profile and Bed Features: A longitudinal profile is measured to determine changes in river slope compared to valley slope which is very sensitive to sediment transport, competence and the balance of energy. Pool to pool spacing, ratios of maximum depth of pools/mean depth of channel, and maximum depth of riffles/mean bankfull depth are also obtained from longitudinal profile data. When pools start to fill (decrease in max depth/mean depth ratio), and the stream is widening with a corresponding decrease in sinuosity and increase in slope, the stream is becoming unstable. The reference condition for the same stream type will have dimensionless ratios that are used for comparison of the magnitude of departure. Spacing of step/pools in steeper stream types are inversely proportional to slope and directly proportional to width, and as such, are shown as a ratio of bankfull width by slope categories. The total removal of large woody debris often increases the step/pool spacing and as a result the excess energy increases the potential for channel degradation. Level IV validation of prediction estimates are accomplished by installing permanent longitudinal profiles with bench marks tied into permanent cross-sections or stationing pins. Measurements taken on a thalweg survey provides data on maximum bankfull depths, the various bed features, including riffles and pools, and documents any change in slope. Elevation measurements of the bed, water surface, bankfull, and low bank height also identifies changes in degree of incision along the profile as presented above in assessment item 2). Data summaries including dimensionless ratios for bed features and river profile can be recorded and analyzed in the "Reference Reach Field Book", (Silvey and Rosgen, 1998.

6) Channel Dimension Relations: Determine changes in the bankfull width and mean bankfull depth (width/depth ratio). This ratio indicates departure from the reference reach and is very sensitive and diagnostic of instability. Increases in width/depth ratio are often associated with accelerated streambank erosion, excessive sediment deposition, streamflow changes, channel widening due to evolutionary shifts from one stream type to another (i.e., G4 to F4 to C4), and direct alteration of channel shape from channelization, etc. The degree of width/depth ratio increases are shown as a departure from the reference condition of the stable stream type to establish stability ratings (Table 2). A *decrease* in width/depth ratio departure analysis will have a proportionate reduction in width/depth ratio values. This reduction from the reference condition is only applied when *the bank height ratio is greater than 1.0.* For example a "*moderately unstable"* rating for a stream channel with a bank height ratio greater than 1.0 would have a width/depth ratio *decrease* of 0.8 to 0.6 This is associated with a width/depth ratio that is *decreasing* as the stream is *incising* (i.e., C4 stream type conversion to a type G4). The corresponding reduced width/depth ratio creates excess shear stress in an incising stream type, which is adjusting toward an unstable condition. The level IV analysis establishes permanent, monumented cross-sections to determine the rate and extent of change in both the width/depth and bank height ratios.

Table 2. Conversion of width/depth ratios to adjective ratings of stability from reference conditions

Stability Rating	Ratio of W/D Increase
Very stable	1.0
Stable	1.0 - 1.2
Moderately unstable	1.21 - 1.4
Unstable	> 1.4

7) Stream Channel Scour/Deposition Potential (Sediment Competence): Compute critical dimensionless shear stress to determine the size of sediment particle that can be moved. Relations modified from Andrews (1984) and Andrews and Nankervis (1995) are used for this computation. Subsequent calculations using a Shields relation compares the existing slope and depth of a stream to be able to transport the largest size made available annually (during bankfull stage) to the channel. The procedure involves sampling the bed material on the riffle to obtain d_{50}, excavate a core sample of bar material (located on the lower 1/3 of meander on the point bar midway between the thalweg and the bankfull stage). The bar sample is used to obtain d_{s50} of the relation as a surrogate of the sub-pavement size distribution. Locations of this specific depositional feature and subsequent are shown in Chapter 7, (Rosgen , 1996). The bar also provides an interpretation of the size distribution of bedload at the bankfull stage and the largest size on the bar is used to obtain data representing the largest size of sediment frequently made available to the channel. The following calculations are used to make the competence prediction:

$$\tau_{ci} = .0834\,(d_{50}/ds_{50})^{-.872}$$

Where: τ_{ci} = critical dimensionless shear stress
d_{50} = median diameter of pavement or bed material on riffle
ds_{50} = median diameter of bar sample (sub-pavement)

The following equation is used to predict the depth and slope to move the largest size of sediment made available to the channel on a frequent basis: $\tau_{ci} = \frac{dS}{(\gamma_s)\,(Di)}$,

transformed to: $d = \frac{(\tau_{ci})\,(\gamma_s)\,(D_i)}{S}$

Where: γ_s = submerged specific weight of sediment
D_i = Largest diameter of particle on bar (use mm if depth is in meters)
d = mean bankfull depth of the channel
S = water surface slope at the bankfull stage

If the combination of depth and/or slope does not move the largest size, then potential aggradation or excessive deposition and corresponding high width/depth ratio is anticipated. If the depth and or slope exceeds that required to move the largest size, then potential degradation, or excess scour leading to incision has potential for instability. This procedure is verified by three methods at level IV: 1) Measured bedload size distribution of bedload at the bankfull stage, and corresponding slope and bankfull depth measurements at the bankfull stage, 2) Monumented cross-sections and vertical scour chains are installed before and after runoff. The scour chains give the depth of scour and subsequent change of particle size over chain. The largest particle over the scour chain exhibits the largest size of particle moved for the corresponding shear stress of the flows responsible. The cross-section shows net change of bed elevation, and specific changes over the scour chain, and 3) Annual replicate core samples at the same location on the bar shows sizes moved for a back-calculated shear stress as well as shifts in size distribution of bedload at the bankfull stage. It is the coarse fragment that determines channel morphology of gravel-bed streams (Leopold, 1992), thus it is important to be able to move the largest sediment clasts frequently made available to the river at the bankfull stage. These data can also be compared to measured bedload size distribution with the USGS, Helley-Smith bedload sampler. This field method has been tested on many rivers by the author with excellent success when compared to both scour chain and measured bedload data.

8) Stream Channel Stability rating (modified Pfankuch procedure): Determine channel stability ratings to predict potential state from the stable reference reach of the same stream or potential evolutionary type. The stability rating procedure evaluates the upper and lower banks and streambed for evidence of excessive erosion/deposition. The procedure has been used for 25 years by the USDA Forest Service and other Federal Agencies, (Pfankuch, 1975). The system evaluates mass wasting potential adjacent to the channel, detachability of bank and bed materials, channel capacity and evidence of excessive erosion and/or deposition. The larger the number, the greater the risk for instability. The risk rating of the classification was later converted to ratings by stream type. This modification was made to reduce the likelihood of applying the same numerical rating of "good" to C4 versus B4 stream types. Naturally, C4 stream types by their meandering nature, flatter slopes and point bars will obtain higher channel stability numbers than the steeper B4 stream types, even though both streams are very stable. In contrast, the channel stability ratings for a very stable B4, will be much lower than the C4 stream type, when both are stable. To remedy this dilemma, relations were developed to place numerical categories in adjective ratings by stream type (Table 3). For example a rating of "Good" for a B4 has a range of 40-64, whereas, the "good" rating for C4 stream types is 60-95. Applications for stability and sediment supply have been related to measured sediment rating curves. For example, the higher the stability rating number, the higher the intercept and steeper the slope of the suspended sediment rating curve as shown in Figure 3 for Redwood Creek, California (Leven, 1977). A similar analysis was performed on measured stream data in North Carolina (Coweeta Experimental Forest), Northern California, Idaho, Montana and Colorado (Rosgen, 1980). This is used in conjunction with dimensionless ratio sediment rating curves in the next category of assessment. Level IV verification involves the combination of measured sediment rating curves, cross-sections, longitudinal profiles and channel material size distributions. This level of assessment compares predicted to observed values of sediment and stability.

Table 3. Conversion of Stability Rating to Reach Condition by Stream Type

Stream Type	A1	A2	A3	A4	A5	A6	B1	B2	B3	B4	B5	B6
Good (Stable)	38-43	38-43	54-90	60-95	60-95	50-80	38-45	38-45	40-60	40-64	48-68	40-60
Fair (Mod. Unstable)	44-47	44-47	91-129	96-132	96-142	81-110	46-58	46-58	61-78	65-84	69-88	61-78
Poor (Unstable)	48+	48+	130+	133+	143+	111+	59+	59+	79+	85+	89+	79+
Stream Type	**C1**	**C2**	**C3**	**C4**	**C5**	**C6**	**D3**	**D4**	**D5**	**D6**		
Good (Stable)	38-50	38-50	60-85	70-90	70-90	60-85	85-107	85-107	85-107	67-98		
Fair (Mod. Unstable)	51-61	51-61	86-105	91-110	91-110	86-105	108-132	108-132	108-132	99-125		
Poor (Unstable)	62+	62+	106+	111+	111+	106+	133+	133+	133+	126+		
Stream Type	**DA3**	**DA4**	**DA5**	**DA6**	**E3**	**E4**	**E5**	**E6**				
Good (Stable)	40-63	40-63	40-63	40-63	40-63	50-75	50-75	40-63				
Fair (Mod. Unstable)	64-86	64-86	64-86	64-86	64-86	76-96	76-96	64-86				
Poor (Unstable)	87+	87+	87+	87+	87+	97+	97+	87+				
Stream Type	**F1**	**F2**	**F3**	**F4**	**F5**	**F6**	**G1**	**G2**	**G3**	**G4**	**G5**	**G6**
Good (Stable)	60-85	60-85	85-110	85-110	90-115	80-95	40-60	40-60	85-107	85-107	90-112	85-107
Fair (Mod. Unstable)	86-105	86-105	111-125	111-125	116-130	96-110	61-78	61-78	108-120	108-120	113-125	108-120
Poor (Unstable)	106+	106+	126+	126+	131+	111+	79+	79+	121+	121+	126+	121+

9) Dimensionless Ratio Sediment Rating Curves: Instability and the corresponding increase in sediment supply is often reflected in measured sediment rating curves. The source of this increase in sediment supply often is associated with channel adjustment, including degradation and lateral accretion (bank erosion). The variation in sediment rating curves is shown in Figure 4, reflecting differences in sediment supply for various Colorado streams (Williams and Rosgen, 1989, and Rosgen, 1996). Additional sediment rating curves by channel stability ratings indicating changes in stability and associated sediment supply are shown in Rosgen, (1980). On the Hatchie River in West Tennessee, Simon (1989) summarized the effects of channelization and corresponding stream stability change comparing evolution stages of channels to measured upward shifts in the slope of the measured suspended sediment rating curves. The sediment yields, from the Hatchie River, a stable, meandering, low width/depth channel with a well developed floodplain (E6 stream type) was 62.9 tons/km^2 (163 tons/year/mi^2). The South Fork Forked Deer River which, following channelization, became incised (F6 stream types) with resultant sediment yields of 961.4 tons/km^2 (2,490 tons/yr/mi^2). Simon (1989) was showing these changes in sediment yield associated with channel instability and adjustments using the channel evolution model (Shumm, et al 1984 and Simon and Hupp, 1986). The channel evolution model and stages of adjustment are related to quantitative morphological values corresponding to stream types (Rosgen, 1999). Both of these approaches are compatible at describing the consequence of channel adjustment. As the E6 stream type incises and changes to a G6 and eventually F6, the channel goes through an evolutionary adjustment of instability

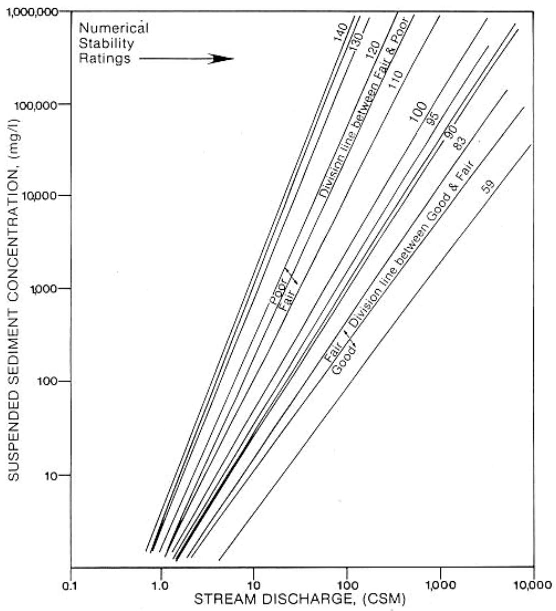

Figure 3. Suspended sediment rating curves by channel stability ratings (from Leven, 1977 in: Rosgen, 1980)

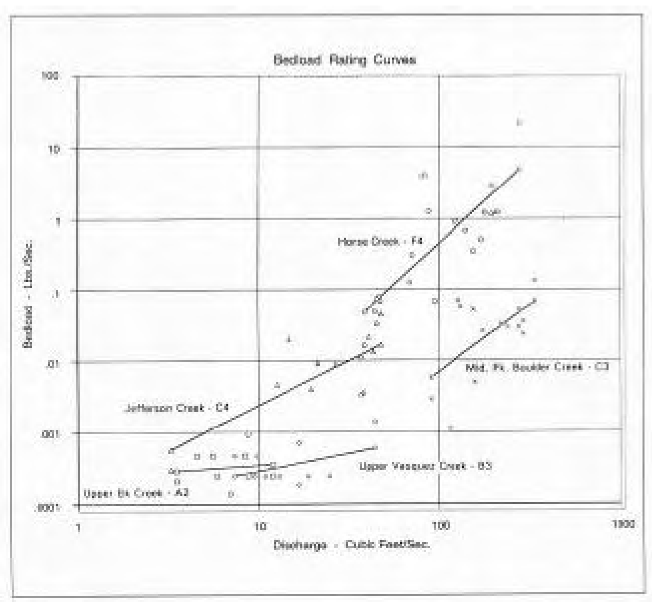

Figure 4. Bedload rating curves stratified by stream type (Rosgen, 1996, data from Williams and Rosgen, 1989)

and associated stream type change. The evolutionary sequence of the Hatchie and South Fork Forked Deer Rivers matches scenario #5 (Figure 1). When instability due to change in energy, sediment supply or direct disturbance occurs, the severity is such that stream types can change. This change reflects increases in sediment supply due to channel adjustment (streambed and streambank erosion) and eventual increased sediment yield as shown by Simon (1989). This does not infer, however, that all F stream types are unstable, such as the Colorado River in the Grand Canyon. The best assessment approach is a combination of stability analysis with stream morphology necessary to establish potential departure from the reference condition.

Reference reach sites representing stable stream reaches are used to establish dimensionless sediment rating curves by stream type and stream stability and as such, can be used to ascertain departure (Troendle, et al, In Press). Significant departure from the reference dimensionless sediment rating curves when comparing good and fair with poor stability ratings. Stream types that become unstable to the extent that they change morphological type are generally associated with poor stability and an increase in sediment supply. Sediment delivery ratios provide a means of extrapolation of sediment rating curves for rivers of different geology, size, stability, and associated morphology. These ratios are developed by taking the bedload and suspended sediment values and dividing them by the same units of sediment values at the bankfull discharge. Their corresponding discharges are also divided by the bankfull discharge to establish dimensionless ratio sediment rating curves. To convert these curves to actual numbers following extrapolation, sediment and discharge measurements at bankfull are obtained at the most detailed level of river stability assessment, then multiplied by the dimensionless ratios established for that stream type and stream condition. Sediment and flow data should also be collected at a lower flow to insure the slope and intercept of the dimensionless ratio sediment rating curve matches observed values. Confidence bands above the reference reach for the same, but stable stream type using the dimensionless ratios give a preliminary range of departure. In other words, the natural variability in sediment supply as shown in the sediment rating curves for the reference reach is documented to avoid the unwise tendency of trying to establish sediment TMDL's as a fixed value for a given stream. Also, natural geologic sediment rates can be established as reflected in the various stream types such as the A3a+ (steep, debris torrent channel incised in heterogeneous, unconsolidated landslide debris and or glacial till). These stream types have periodic and catastrophic, naturally high to extreme erosion rates due to their unlimited sediment supply and high energy. Sediment yields from these systems cannot and should not be altered, as the entire fluvial system has adjusted over time to accommodate such sediment loads. Efforts to restore "stability" in these channel types are fighting natural processes and face a high risk of failure.

10) Stream Type Evolutionary Scenarios: Determine the current state and evolutionary sequences as shown in Figure 1. The use of this relation requires the field observer to select not only the stream type, but the location in a particular sequence of evolution. This not only provides a current state evaluation, but provides an interpretation of the physical potential of this reach. A stability assessment can assist those doing restoration design. Often, unstable channels are "patched in place"…unfortunately it is often the "wrong place"…or perhaps, the wrong stream type. Another use of this specific assessment protocol is to be able to identify the potential stable stream type as opposed to the currently existing stream type. Restoration can speed up the adjustment or recovery period by obtaining the morphological data used from the reference reach of the appropriate stream type to match the stable form. Another application of stream type evolution is to specify a potential dimensionless sediment rating curve that would apply associated with the stability of a particular morphological evolutionary state as depicted in figures 5 and 6.

Summary of condition assessment and stability ratings: The summary of the ten major stability rating categories and condition variables of the level III prediction analysis are shown in Table 4. By completing each of the above assessments, the field practitioner can see the pattern of channel change and note a change in one variable is accompanied by changes in several others. The interpretation of the stability categories allows the observer to conclude as to the overall stability and potential state of sediment supply.

Table 4. Summary of stability condition categories for the Level III inventory

Stream Name ______________________________	Observers
Location ___________________________________	Stream Type _____________ Date
Riparian Vegetation, comp/density _____________	Flow regime
Stream size, Stream order ____________________	Depositional pattern
Meander pattern ____________________________	Debris/channel blockages
Channel stability rating (Pfankuch) ______________	Describe altered channel state
Stability category by stream type_________	__
Sediment supply (check appropriate category)	Dimension/shape:
Extreme ___________________________	Width
Very high ___________________________	Depth
High _______________________________	Width/depth ratio
Moderate ____________________________	Patterns (*show as function of W_{bkf})
Low ________________________________	Meander length*
Streambed (vertical) stability	Radius of curve *
Bank Height ratio ____________________	Belt width*
Aggrading ___________________________	Sinuosity
Degrading ___________________________	Arc angle
Stable _______________________________	Arc length* __________________________________
	Profile:
Width/depth ratio/condition:	Water surface slope
Excellent (stable) ____________________	Valley slope
Good _______________________________	Bed features: (Type and/or ratio max. depth/bankfull depth)
Fair ________________________________	Riffle
Poor ________________________________	Pool
Streambank erosion hazard:	Step/pool (p/p spacing)
Bank erodibility: Near-bank stress:	Convergence/divergence
Extreme _________ Extreme_________	Riffle/pool spacing *
High ____________ High ___________	Dunes/antidune/smooth bed
Moderate _______ Moderate ______	Describe channel evolution scenario:
Low ____________ Low ___________	Evolution type number
Very Low _______ Very Low ______	Existing state (type)
Annual streambank erosion rate _____________	Potential state (type)
Length of banks studied ________________	Competence calculation:
Tons/year ___________________________	Critical dimensionless shear stress
Curve used __________________________	Largest particle on bar
	Bankfull depth (existing)
Dimensionless Sediment rating curve:	Bankfull depth required
Normal ______________________________	Slope (existing)
Above normal _________________________	Slope required

CONCLUSION

Although this stability prediction method may seem onerous, it has been applied in watershed management and for geomorphological assessments for many years. The author has trained hundreds of individuals in this procedure that have collected both level III and IV data to help improve and validate the prediction relations. An additional application of this approach has been used for restoration proposals, where an understanding of the cause, consequence and correction of the problem involves an inventory that isolates the processes associated with stream channel instability. The recent requirement to establish TMDL's for clean sediment involves an understanding of natural rates, natural variability and documenting departure conditions leading to adverse consequence of instability and corresponding disproportionate sediment yields. If we understand the various processes of change, prevention through good management and application of mitigation measures can be appropriately applied to the problem. Continued field measurements are the key to improving upon these procedures and add to the collective understanding of these complex and valuable river systems.

"*A consistent chronicle of field observations and collected data is essential to the practice of hydrology. As with opportunities, good data are available only once."….* Luna B. Leopold.

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KING CREEK CHANNEL RESTORATION

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Abstract: King Creek is a small stream located in the Little Rocky Mountains of Montana. Gold mining operations during the past several decades caused the deposition of contaminated mine tailings within the stream channel. Removal of contaminated mine tailings was conducted under authority of the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA). Following mine tailing removal, restoration of the stream channel geometry was required. Design components evaluated include the determination of design flow rates and channel geometry features such as gradient, cross section, and planform. This paper presents some aspects of stream channel restoration design and discusses project constraints caused by existing project features and land use.

INTRODUCTION

The King Creek project is located near Hays, Montana, on the Fort Belknap Indian Reservation. Mining for gold and silver within the Little Rocky Mountains began in 1884. Originally, the tailings from the mining operation were confined to the headwater area of King Creek. A series of small dams were constructed during the progression of mining activities to contain the tailings. Major floods failed the dams on several occasions, most notably in 1952 and 1974. Flood waters transported and deposited mine tailings throughout the King Creek basin and downstream watersheds.

Mine tailing removal was conducted by the U.S. Environmental Protection Agency (USEPA) under the authority of CERCLA. EPA requested that the U.S. Army Corps of Engineers, Omaha District, execute the project through the Rapid Response Program. An Engineering Evaluation/Cost Analysis (EE/CA), was prepared to examine the removal action alternatives. Based on the EE/CA, the selected alternative consists of partial removal of mine tailings and disposal of the tailings at the existing mine site. A work plan was developed to describe in detail project objectives and planned activities. (USACE, 2000). Following completion of the work plan, mine tailing removal and channel restoration construction activities were conducted from June through October, 2000.

PRE-PROJECT SITE CONDITIONS

King Creek originates in Phillips County and enters Blaine County and the Fort Belknap Reservation approximately 1 mile from its headwater. Mine tailings were scattered throughout the basin with major deposits behind beaver dams and other natural detention areas. The area contains significant cultural resources of the Fort Belknap tribes, including the Pow Wow grounds and the Sun Dance area. King Creek project limits are illustrated in Figure 1.

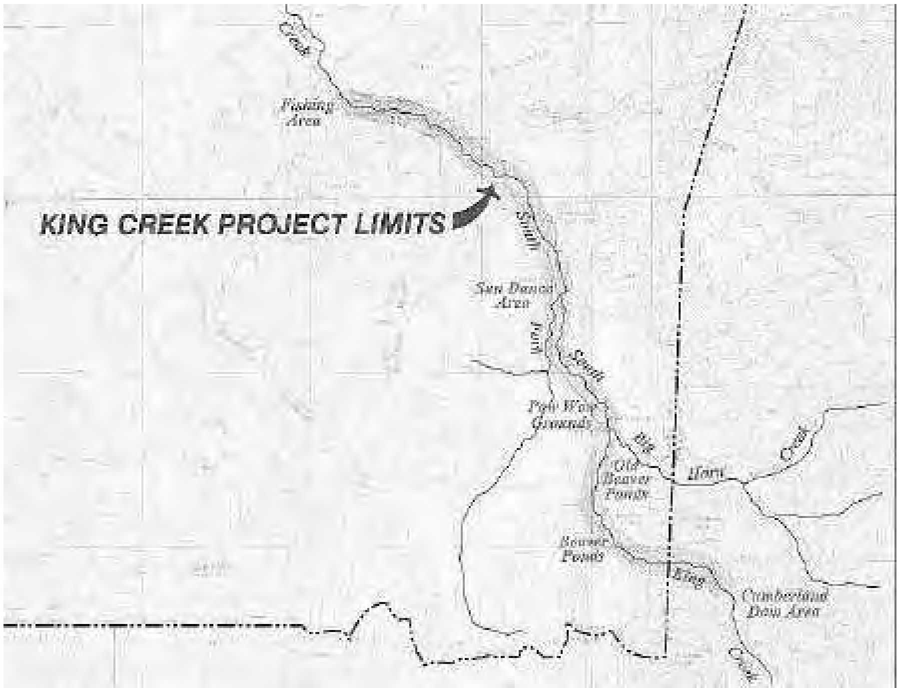

Figure 1

<u>Project Reach</u>: The area of the creek addressed during this project extends from the Cumberland Dam downstream to the Mission Canyon fishing area on the Ft. Belknap Reservation. Total stream length is approximately 5 miles. Within the project reach, King Creek is a tributary to South Big Horn Creek that in turn is a tributary to the South Fork of Little Peoples Creek. Previous investigations have located large tailing depositions at the Sun Dance area, at several beaver pond sites, at an existing swimming hole, and at the vicinity of the Cumberland Dam area. Total stream length included in mine tailing removal is approximately 3000 feet. Drainage basin size increases from approximately 0.1 square miles downstream of Cumberland Dam to over 7 square miles near the existing swimming hole at the downstream project limit. The pre-project channel has been severely impacted by the mine tailings migrating through the project reach.

<u>Mine Tailings:</u> Mine tailing deposits are easily visible within the project reach. The tailings are lighter in color than the native material. Mine tailings are uniformly sized in the range of a medium to coarse sand. The small grain size of the mine tailings combined with the steep stream slopes translates into high mobility. Tailings have been observed within Little Peoples Creek up to a distance of 20 miles downstream of the King Creek confluence.

Removal of mine tailings was not feasible throughout the drainage basin. Tailing depth varied from non-existent in some areas to over 10 feet in others. Access for tailing removal was also restricted in several areas by site topography. Tailing removal was concentrated to areas where the deposited thickness exceeded 6 inches. Mine tailings were removed employing conventional excavation techniques. Contamination levels within the mine tailings and the associated health risk did not require special excavation techniques.

Stream Classification: Of the various classification systems, the applied method is that developed by Rosgen (1994). Within the project reach, the channel slope varies from 2% to over 8%. Channel sinuosity is low and varies from 1.2 to 1.4. The entrenchment ratio varies from 1.2 to over 2. The width/depth ratio varies from low to moderate. Using a pebble count procedure (Rosgen, 1994), the D_{50} of the existing bed material was determined as 4 to 5 inches at the gage location and within an adjacent stream employed as a reference reach. Measurement of the bed material within the project reach for use with stable channel design was not applicable due to the tailings. According the Rosgen method, the upper reaches of King Creek are classified as A3 with the lower reach transitioning to B3.

DESIGN PROCEDURE

The general design procedure involved determination of discharge-frequency through the project reach, evaluation of channel planform, and computation of channel lining stability. Typical available references for channel restoration recommend employing the channel-forming or bankfull discharge to evaluate and design stable channel features. (FISRWG, 1998). The definition of channel-forming discharge and bankfull discharge principles are not universally accepted and subject to debate. For the purposes of this paper, the channel-forming discharge was evaluated using a flow recurrence approach. A detailed analysis employing recommended design procedures such as flow duration analysis and the derivation of a sediment transport rating curve was not performed.

Analysis Procedure: The simplified analysis procedure employed for this study consisted of determining the flow-frequency relationship at each location, compute the required channel geometry for the various project reaches, verify channel riprap lining stability, and design project specific features within each reach. The SAM analysis package (USACE, 1998) was employed to insure that mine tailing transport continued within the restored reach. Since complete excavation of mine tailings within the project was not possible, sediment retention dams were included in the design to trap tailings for future removal.

The design phase was extremely short for the project. Final topography within the restoration areas was not known until mine tailing excavation was completed. The construction time frame required that restoration construction begin almost immediately after tailing excavation. A detailed design and evaluation of project alternatives was not possible in the short time frame.

Design Constraints: Project design constraints include the Cumberland Dam upstream of the project, the construction of two additional sediment dams within the project reach, mine tailing excavation and removal of all suitable channel lining material, reaches of unexcavated tailings within the project, and limitations on channel alignment in several areas. The termination of tailing excavation was often stopped at undesirable hydraulic locations to avoid impacting trees and tribal cultural resources.

Stable channel design in the context of the King Creek project required modification from a traditional approach. A component of a stable channel is the capability to pass the incoming sediment load without significant aggradation or degradation. The Cumberland Dam plus the two additional sediment retention dams limits sediment inflow. After mine tailing excavation, the remaining channel lining material was generally silt or clay. Since channel slopes are generally in the range of 2-6%, additional lining was required to avoid degradation.

HYDROLOGIC ANALYSIS

An analysis was conducted of significant tributaries within the study reach to determine the design flow-frequency relationships at various locations through the project reach. Hydrologic analysis was conducted to determine the dominant discharge for use with design of stable channel features. Analysis was also required to determine less frequent events to evaluate stability of critical project features. Analysis methods employed gaging station records, available regional equations, and field measurements. Peak discharges were computed for the 2-, 5-, 10-, 25-, 50-, and 100-year events.

Gage Data Frequency Analysis: The USGS gage station number 06154410 is located downstream of the project on Little Peoples Creek near Hays, Montana. The period of record is from 1972 to the present. The annual peak discharge records were analyzed using the procedures outlined in WRC Bulletin 17B (USWRC, 1982) to determine a discharge-frequency relationship.

Regional Equations: Regional equations for Montana provide a second method for computation of discharge-frequency (USGS, 1992). The USGS report lists regression equations for computing peak discharge for the 2-year through the 500-year event. The regional equations compute peak discharge from drainage area and elevation. The equation for the 2-year event is

$$Q_{2yr} = 15.4 * A^{0.69} * (E/1000)^{-0.39} \qquad (1)$$

where Q_{2yr} is the computed peak flow (cfs), A is the drainage area (sq. miles), and E is the elevation (feet msl). Although not presented here, additional equations of the same form were employed for the remaining flow events.

Determined Flow-Frequency: Historical mining activities altered the upper drainage basin and reduced the King Creek drainage area by approximately 0.23 square miles. Although not part of this project, supplemental flows of approximately 1 cfs may be supplied to King Creek as mitigation for lost drainage area. Computed flows within the upper King Creek basin were increased by 1 cfs to account for the potential increase. Peak discharge at each location was determined by multiplying the gage values by the ratio of the drainage areas raised to the drainage area exponent. By using the USGS regional equation exponent in the ratio, the relationship of the different frequency events was preserved. The final peak discharge value at each site was determined by

$$Q_{design} = Q_{GageFreq} * \left(\frac{A_{Site}}{A_{Gage}} \right)^{USGS\ Reg.Exp} \qquad (2)$$

where Q_{design} is the determined design flow rate (cfs), A_{Site} is the total drainage area at the individual site, A_{Gage} is the gage drainage area of 13 square miles, and USGS Reg.Exp is the regional exponent applied to the drainage area in the USGS regional equation. The drainage area exponent varies from 0.69 for the 2-year event to 0.59 for the 100-year event. A summary of the determined discharge-frequency is provided in table 1.

Table 1. Computed Peak Discharge at Various Locations

Location	Dr. Area (sq. miles)	2-Year (cfs)	5-Year (cfs)	10-Year (cfs)	25-Year (cfs)	50-Year (cfs)	100-Year (cfs)
Cumberland Dam	0.08	2	6	9	16	22	30
Upper Beaver Ponds	0.92	9	24	41	70	97	129
Lower Beaver Ponds	1.06	10	26	45	76	105	140
Lower Sundance	6.86	33	89	145	238	324	422
Swimming Hole	7.13	34	92	149	244	332	432
Middle Fork	1.57	12	34	57	97	134	177
Gage Location (USGS Regional)	13.0	52	147	243	390	529	673
Gage Location (Freq. Analysis)	13.0	52	140	226	369	500	650

As table 1 illustrates, the gage data analysis and the USGS regional equations determined similar results at the gage station location. Results from the two methods compared very well with no difference at the 2-year event and an average difference of less than 5% for the remaining events.

USGS Gage Field Data Analysis: Cross sections and water surface elevations were surveyed in the vicinity of the USGS gage locations. The bankfull flow depth was estimated by field observations and transferred to the staff gage. Using the gage rating curve, the bankfull depth corresponds to a flow rate of 47 cfs. From the gage frequency analysis, the estimated recurrence interval is 1.9 years. Various references often state the bankfull discharge is estimated to occur with a recurrence interval between 1.0 and 2.5 years (FISRWG, 1998). Therefore, the analysis of the gage field data further supports the hydrologic frequency analysis.

Reference Reach: Within the Little Rockies, most of the streams have been impacted by the intense mining operations. Jeep trails and access roads have also impacted existing streams. A reference reach location was selected within the upper reach of the Middle Fork of Little Peoples Creek. At the reference reach location, measured cross sections, profile, and observations were collected. Using a normal depth analysis, computations determined a bankfull discharge of 11 cfs. The hydrologic analysis method determined a 2-year peak flow rate of 12 cfs for the drainage area. The two methods compare fairly well and indicate that the computed peak flow frequency is reasonable.

PROJECT DESIGN

Design Phase: The project design phase was extremely short. Following mine tailing excavation within a reach, survey data of the excavated channel was collected. The design channel alignment, section, slope, and location of transition features was determined as quickly as possible. Generally, construction of the restoration design began within 5 to7 working days after tailing excavation was completed. Coordination with tribal representatives further shortened the design phase. The short design phase limited the design detail.

Channel Cross Section and Planform: As a minimum, the channel cross section was sized to contain the 2-year peak flow rate. Within some reaches, the design flow rate was increased to the 25-year event. Higher channel capacity was required to provide increased reliability at critical locations. Stream restoration guidelines often recommend employing a limited channel capacity combined with floodplain capacity. However, the influence of sediment dams, limited stream access for repair in most reaches, the channel stability provided by the riprap lining, and the impact of mine tailings on stream stability all contributed to the desire for a larger channel capacity.

Channel width was evaluated using reference reach data. Flow area and channel capacity was confirmed with a normal depth analysis and evaluation with SAM (USACE, 1998). Many of the project areas have a design slope which greatly exceeds the reference reach slope. Following the excavation of mine tailings, the nearly vertical side slopes were backfilled to a stable slope between 1.5H on 1V and 2.5H on 1V. The flow channel was established within the remaining corridor. The channel section included a minimum 5-foot floodplain width between the side slope and constructed channel. Established channel section width varied from 5 to 8 feet with a maximum flow depth of 1.5 feet. Meander wavelength was determined using available relationships (Rosgen, 1994) and constrained by the reach topography. Generally, the meander length varied from 5 to 7 times the channel bankfull width. In order to reduce bend shear stress, the radius of curvature was limited to a minimum of 3 times the channel width.

Sediment Dams: Sediment dams were constructed at two locations within the project reach. The sediment dams are required to provide a trap for tailings that still remain within the project reach. The sediment dams were designed to be low height dams to minimize public risk. At each dam, the outlet consists of a rock lined chute. Chute width was determined as 10 feet. A 1.5 foot chute depth corresponds to the estimated flow depth for the 25-year event. For events greater than the 25-year, dam overtopping will occur. The sediment dams included a compacted clay core, upstream riprap facing, and a downstream slope of 5H on 1V. The low height and other design provisions reduce the risk of dam failure during events that overtop the dam.

Channel Riprap: Channel riprap was sized to provide stability for the design event. In the vicinity of the project, the nearest riprap quarry was over 80 miles distant. Therefore, an on-site source was located. An adjacent valley contained a talus slope of suitable material. Excavation of the rock determined that a reasonable gradation could be achieved. Using the channel riprap evaluation techniques provided by the U.S Army Corps of Engineers (1994) and the Federal Highway Administration (1994), the minimum rock size was computed. Minimum rock size was evaluated for each design reach using the radius of curvature, channel width, and flow parameters. Due to the small flow depth, the Federal Highway Administration method employed the Bathurst technique to compute Mannings' roughness value. Between the two methods, the minimum rock size was determined as a D_{50} of 0.45 feet.

The actual size of the material within the talus slope was difficult to determine. Therefore, the rock size was measured within the channel using a pebble count procedure. Figure 2 illustrates the specified rock gradation and the measured data. Based on field measurements, the talus slope rock was determined to be acceptable. It should be noted that in most cases, a performance specification would require the testing by weight to verify riprap size.

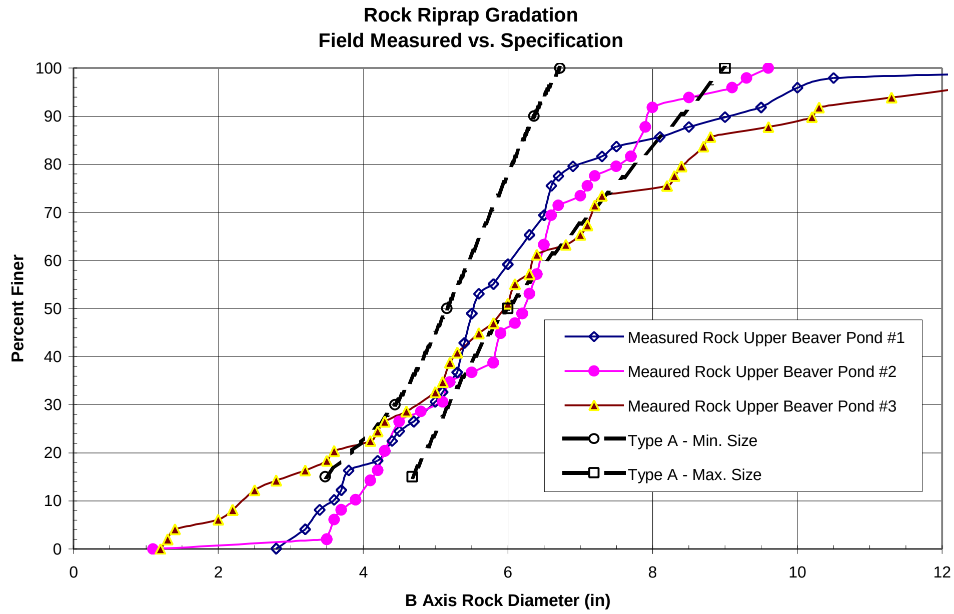

Figure 2

<u>Riprap Chutes:</u> Rock riprap chutes with slopes in the range of 10-20% were employed at several locations. Steep slope riprap chutes were required at the sediment dam outlets, upstream grade adjustment structures, and downstream grade control structures. Due to the critical location of the chutes, the design peak discharge was between 10-year and 25-year frequency. Rock riprap size was computed using steep slope riprap design guidance (USACE, 1994) which is stated as

$$D_{30} = \frac{1.95S^{.555}q^{2/3}}{g^{1/3}} \qquad (3)$$

where d_{30} is the riprap size of which 30 percent is finer by weight (feet), S is the bed slope (ft/ft), q is the unit discharge (cfs/ft), and g is the gravitational constant (ft/sec^2). The design methodology states many recommendations including multiplying the unit discharge by a flow concentration factor of 1.25.

The chute bottom width varied from 6 to 10 feet. Chute alignment was straight. Chute height varied from 3-5 feet. Computations determined a chute D_{30} rock size required for stability between 0.6 and 0.75 feet. The selected gradation used for all chutes corresponds to a D_{100} of 1.5 feet. Originally, the larger rock was attempted to be collected by visually sorting the talus slope. The gradation plotted in Figure 1 as Measured Rock Upper Beaver Pond #3 represents the larger sorted rock. The visually sorted rock was much smaller than required. Therefore, the chute riprap was purchased from a quarry. Although this involved considerable expense, chute stability is critical and could not be achieved with the talus slope rock. Following placement of the chute rock, surface voids were filled with 2-3 inch diameter limestone. The limestone improved the aesthetic appearance of the chute. A second benefit of the limestone was to provide a buffer for drainage from acid generating rock that occurs as a result of mining activity.

Plunge Pools: Plunge pools were employed as energy dissipators at the base of all the steep slope riprap chutes. Plunge pool design followed general design guidance (USACE, 1988). Variation from the standard chute design was required. In one location, alignment restrictions for cultural resources resulted in a chute that terminated at a 90 degree bend. To provide bank stability, a plunge pool was sited at the chute exit, rock riprap was placed on the channel bank, and the downstream culvert was designed to force backwater on the chute at higher flow events. The plunge pool width was approximately 2 times the channel width and the plunge pool length was approximately 3 times the channel width. Using the chute flow Froude number, plunge pool length and depth were also evaluated based on the hydraulic jump length and sequent depth.

SUMMARY

Mine tailing removal within the project reach was conducted by USEPA under the authority of CERCLA. The selected alternative consists of partial removal of mine tailings and disposal of the tailings at the existing mine site. Mine tailing removal and channel restoration construction activities were conducted from June through October, 2000. Determination of hydrologic frequency was presented. Channel planform and riprap lining features were determined for the project reach. The project reach also employed sediment dams, steep riprap chutes, and plunge pool energy dissipators.

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USE OF CHEVRON STRUCTURES TO CREATE DEPTH DIVERSITY IN THE MISSOURI RIVER

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Abstract: Construction of the Missouri River Streambank Stabilization and Navigation Project (BSNP) converted the river from a multi-channel stream that meandered across the flood plain to a single channel that is maintained along a pre-determined design alignment. The original channel that ranged from 1200 feet to 2 miles wide is currently 600 to 1100 feet wide. This narrowing resulted in the loss of over 100,000 acres of shallow open water habitat. The remaining open water consisted of an increased percentage of deep-water areas and shallow water areas are nearly non-existent, especially in the reach from Sioux City, Iowa to Rulo, Nebraska. This has lead to a dramatic decline in native river fish species, and the listing or proposed listing of three fishes as endangered. In an attempt to provide greater depth diversity to the river, while maintaining the navigation and stabilization functions of the project, the Corps of Engineers' Omaha District has constructed three chevron structures in a single bend. The channel in the vicinity of the chevrons has been monitored since 1993. This paper provides a case study of the chevrons placed in the Missouri River as well as a comparison of their performance with similar structures placed in the Middle Mississippi River.

INTRODUCTION

Background: The Missouri River from Sioux City, Iowa to the mouth (735 miles) has been channelized into a single channel through a series of transverse dikes and revetments. The discharges from the basin upstream of Sioux City are regulated by a series of six large reservoirs in Montana, North Dakota and South Dakota. The most downstream dam is Gavins Point Dam at Yankton, South Dakota. Gavins Point is a re-regulation dam that provides flow support for navigation and other uses. The objective of the BSNP is to prevent channel migration and provide a 300-foot wide by 9-foot deep navigation channel. Even with this control, bed material is still transported through the reach. Sources of bed material include the bed of the channelized reach, sediment inflow from the upstream reaches, and sediment inflow from tributaries. Sediment deltas typically form at the mouth of tributaries. High flows on the main stem river remove or redistribute these sediments throughout the system. In natural rivers, the tributary deltas presented navigation difficulties due to the localized instabilities, but were generally navigable. Channelized rivers are designed to provide a smooth and reliable sailing line as shown in Figure 1. Shoaling in the sailing line represents a severe hazard to navigation. Shoals associated with tributary deltas can occur relatively quickly, and if high flows on the main stem are attenuated by reservoir operations, or there is a regional drought, these deltas can remain for some time.

Problem Description: In the late 1980's and the early 1990's the upper Missouri River was in the midst of a severe drought. High flows that are typically associated with spring snow melt and precipitation events were stored in the main stem dams and metered out through the navigation season (April through November). The absence of high flow events, coupled with minor flood events in the Boyer River basin, allowed a shoal to develop at the mouth of the Boyer River during the 1991 navigation season. This shoal continued to increase in size and in 1992 presented a significant hazard to navigation. Depths over the shoal were as low at seven feet and the shoal at one time extended over 200 feet into the channel (Figure 3). This required commercial navigators to pass through a narrow channel adjacent to the end of the dikes (Figure 1). The normal response to these types of shoaling problems is to dredge or structurally modify the project. Structural modifications traditionally include increasing the length and/or elevation of the dikes. However, due to endangered species concerns, state and federal resource agencies were opposed to any alterations that would remove the shoal. The resource agencies would however, support a project modification that would move the shoal from the sailing line to some other location in the channel.

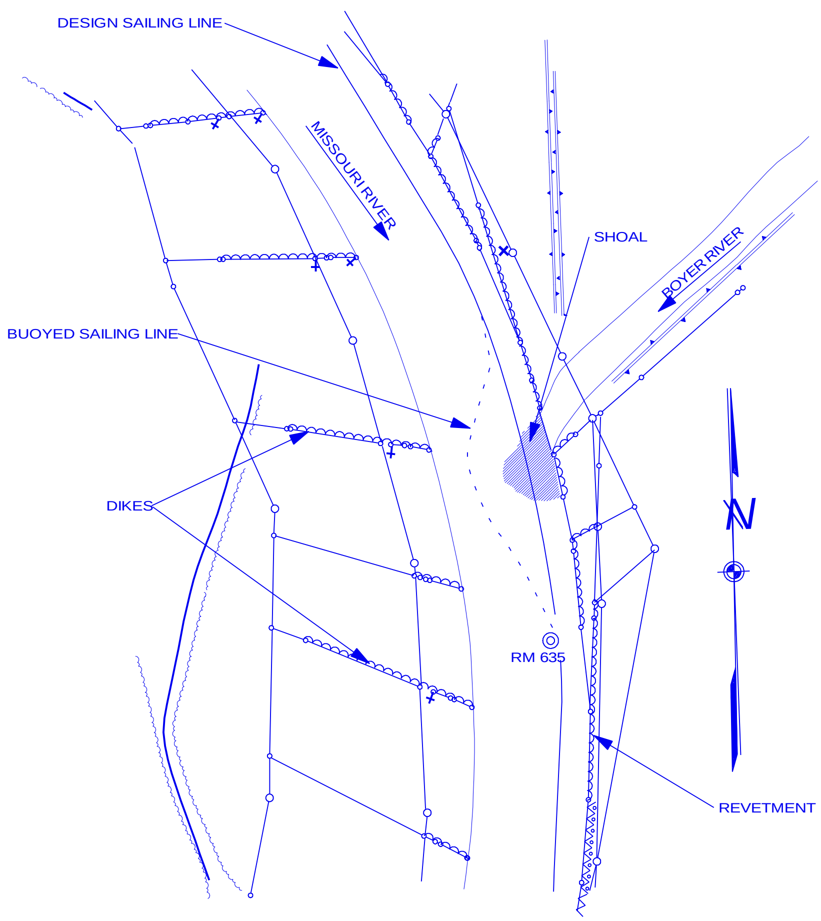

Figure 1. Plan View of Boyer Bend

DESIGN

<u>Concept:</u> Moving the shoal from the sailing line to the non-navigation side of the channel presented a number of challenges. First, channel capacity needed to be maintained. Although the flows in the areas are, for the most part, controlled by upstream reservoirs flooding is still a concern. Second, moving the shoal from the outside of the bend to the inside of the bend may

promote further high bank encroachment into the channel. This could eventually lead to a decrease in channel capacity. Third, any new structure would have to be placed as to not pose a hazard to commercial navigators. Fourth, because this particular area is heavily used by recreational boaters, any structure(s) needed to account for public safety. Lastly, the resource agencies desired the shoal to be higher in elevation, and even exposed during lower than normal flows, if possible.

To restore the sailing line within the constraints mentioned above, it was decided that a structure(s) needed to be placed in the channel to create a sandbar(s) that would not connect to the bank. Experience with vane dikes indicated a tendency to accrete sediments back to the high bank, even when submerged. Although initially vane dikes create a fair amount of shallow water, vegetation eventually establishes along the fringe of the sandbar and the area attaches to the high bank and the aquatic habitat value is quickly lost. Observations of rootless dikes indicated that a channel could be maintained between the high bank and the shoaling area. However, due to the narrowness of the river (600 feet) and the close spacing of the existing dikes, it was felt that rootless dikes would have to extend well beyond the riverward ends of the dikes to be effective. This is a concern to commercial navigation and recreational boaters. Experience with transverse vane dikes on the Missouri River in North Dakota indicated an ability to create a sandbar with a variable elevation in response to the changes in the hydrograph. Observation of chevron type structures on the Middle Mississippi River indicated an ability to maintain flow on both sides of the structure, even when placed out of the main channel. For these reasons, chevron type structures were developed to move the shoal from the sailing line to the opposite side of the river.

<u>Structure Layout:</u> Due to a short design schedule it was decided to develop structures that would remain within established guidelines for sill structures. Sills are underwater extensions of the dikes, which encroach into the channel a maximum distance of 100 feet. Because it was desired to maintain open water between the chevron and the right river bank, the landward wings needed to be short enough to allow a significant amount of the flow around the right side of the structure. Assessment of the dike field indicated that an inboard wing length of 60 to 75 feet would allow water to circulate between the chevron and the right bank of the river. A notch at the point of the chevron was designed to simulate placement of the transverse vane dikes in North Dakota. Three stone chevrons were placed between four consecutive dikes as shown in Figure 2. The crown elevation of the most upstream dike was set at the CRP elevation and the second the third dikes were set at one and two feet below Construction Reference Plane (CRP), respectfully. The CRP elevation is the water surface profile that approximates the discharge that is exceeded 75 percent of the time. The angle between the wings was set at 60 degrees, and each wing was 75 feet long. The side slopes of the chevrons were at the angle of repose for stone, and crown width of each structure was three feet.

PROJECT PERFORMANCE

<u>Pre-project Conditions:</u> At the time of construction, the channel and buoyed sailing line was as shown in Figure 2. Figure 3 compares a cross section surveyed prior to appearance of the shoal and a cross section obtained in 1992 when the shoal was near it's maximum size. From this it is clear to see that although a 300-foot wide by 9-foot deep channel existed, it was not located

along the designated sailing line. Most of the depths in the pre-shoal section are greater than 10 feet, with approximately 30 percent of the depths greater than 15 feet. There is almost no 0-5 foot deep water. While the post shoaling section exhibits a sizable increase in the 5-10 foot depths, the average depth remains nearly constant, and there is no increase in the 0-5 foot range. This resulted in a severe hazard to navigation, but provided little if any aquatic benefit.

Channel Response: Construction of the chevrons was complete in the fall of 1992. By the winter reconnaissance of January 1993, the shoal at the mouth of the Boyer River had reduced in size by approximately 50 percent. By the beginning of the 1993 navigation season the shoal was nearly gone and by the end of the 1993 season was completely removed. Figures 4 and 5 are channel cross section and profile surveys taken in December 1994. In Figure 4, the unaltered cross section is located approximately 0.75 miles upstream of Chevron 678.98, and is representative of a typical river channel (See Figure 3). Comparison of the three cross sections located downstream of the chevrons (Figure 4), indicates that there is a considerable increase in the amount of 0-5 foot depths relative to the CRP (15 to 25 percent of the section), and a decrease in the amount of 5-10 deep water, with relatively little change in the average bed elevation. The shoaling area along the left bank of the river has been removed, and subsequent reconnaissance inspections of the area show no evidence of the shoal reforming.

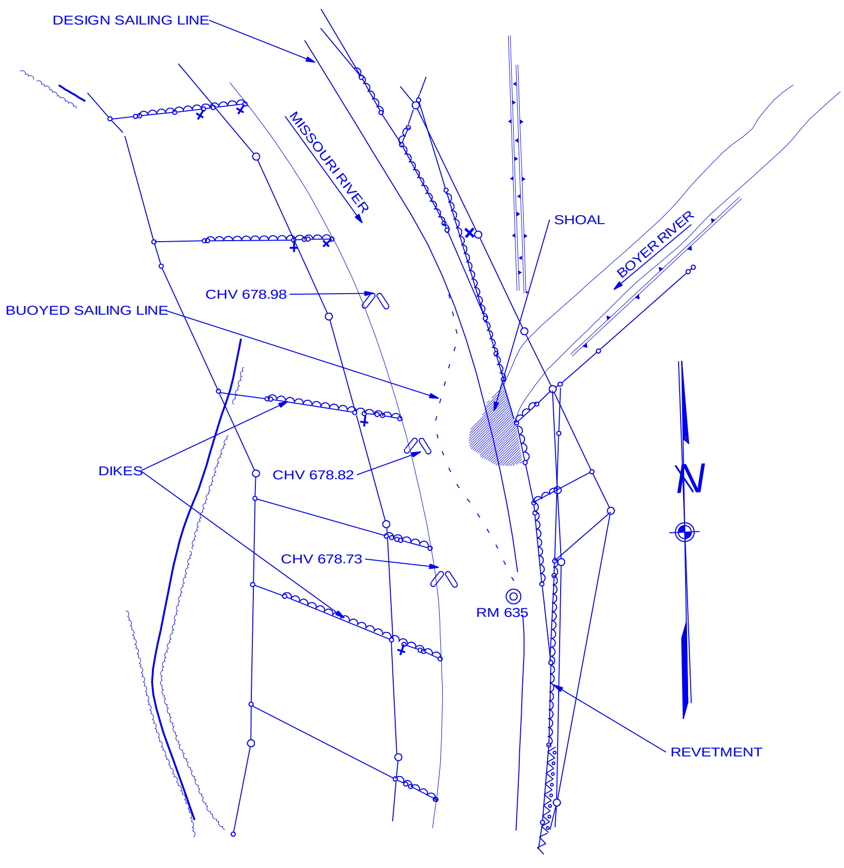

Figure 2. Boyer Bend with Chevrons.

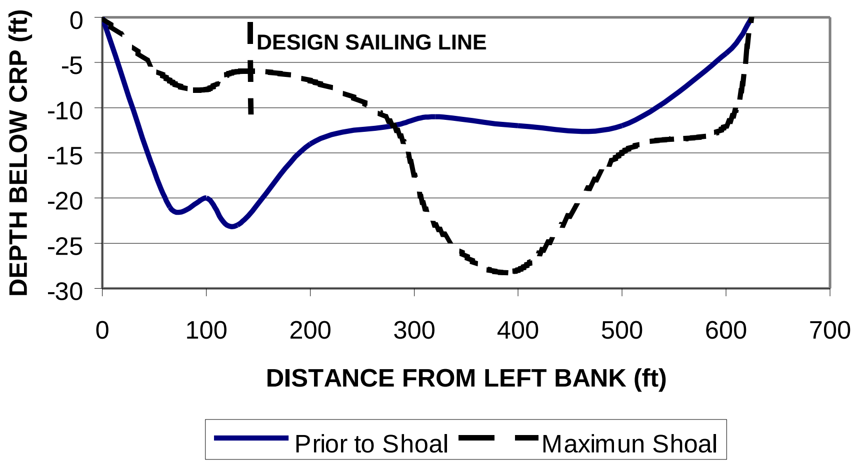

Figure 3. Pre and post shoaling cross sections at the mouth of the Boyer River.

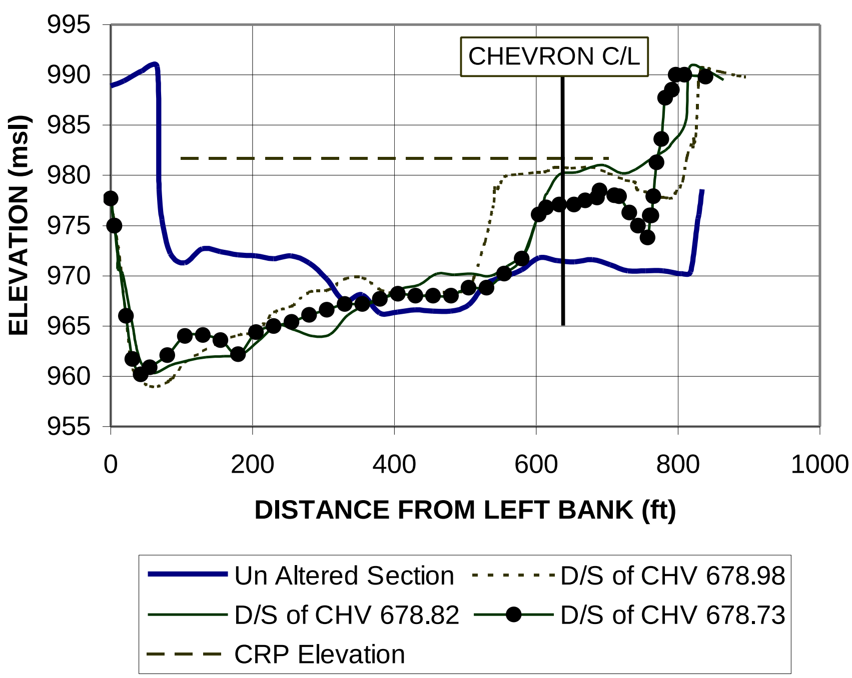

Figure 4. Post Construction Cross Section Surveys (December1994).

The profiles shown in Figure 5 were taken from approximately the end of the upstream dike, through the notch in the chevron, to the end of the downstream dike. These data indicate that the sandbars located downstream of the chevrons occupied 30 to 50 percent of the length of the between the dikes. While there are no pre-construction controlled surveys to compare these profiles to, reconnaissance inspections of the area prior to construction indicate almost no shallow sandbars.

The sailing line has been restored, and the sandbars have remained through an extended period of high flows (1993-1997). The elevations of the sandbars extended several feet above the crown of the chevrons after the 1995 and 1996 high flow years and were at or slightly below the crown elevations other years. The chevron with the highest crown elevation (CHV 678.98) maintained the highest and longest sandbar. However, it should be noted that the gap between the upper two dikes is considerably larger than the gaps between the downstream dikes. Re-circulation flows upstream of each dike would tend to truncate the downstream ends of the sandbars. This undoubtedly influences the length of the sandbars.

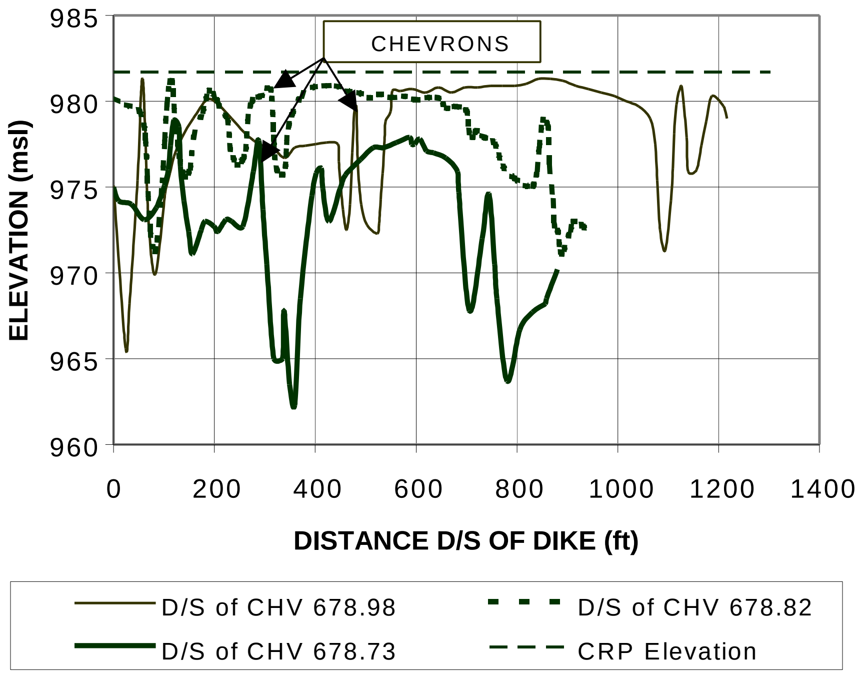

Figure 5. Post Construction Profile Surveys (December 1994).

<u>Performance Compared to Mississippi River Chevrons:</u> As mentioned above chevrons have been placed on the Middle Mississippi River. These chevrons serve as both channel improvements and dredge disposal holding areas. These structures also create riverine habitat for a variety of fish species (Corps). The basic design differences between the chevrons placed on the Middle Mississippi and the Missouri Rivers are: (1) the Mississippi River chevrons have a 'blunt nose" or rounded point, (2) the point of the Mississippi River chevrons is solid, and (3) the size of the Mississippi River chevrons is much larger. Some of the Mississippi River chevrons were placed between dikes and other were place in side channel or on the main channel fringe. The basic function of the chevrons also varies between the two rivers. On the Mississippi River dredge material is placed downstream of the chevrons in an attempt to prevent this sediment from re-entering the navigation channel, while the Missouri River chevrons are designed to capture sediment in order to build the sandbars.

The St. Louis District has monitored the Middle Mississippi River chevrons since 1995. The focus of the monitoring effort has been largely biological (fish sampling), and therefore no control surveys have been completed. However, depth information collected during fish sampling indicates the following: (1) a relatively deep scour hole is maintained immediately downstream of the point, (2) a channel is maintained on both sides of the chevron, and (3) the sandbars extend for a considerable distance downstream of the chevrons. The deep water immediately downstream of the point appears to be maintained by over topping and/or back currents from the downstream ends of the chevrons. The results of the biological monitoring indicate that riverine fish species are attracted to the areas between the chevron and the sandbar.

CONCLUSIONS AND RECOMMENDATIONS

The data indicate that chevrons can be effective in increasing the amount of shallow water habitat in the channelized portion of the Missouri River. Shallow water habitat (0-5 feet deep) increased from nearly zero to as much as 25 percent. However, this was at the expense of the 5-10 foot depths. Average depths remained essentially the same. It is debatable whether or not this is advantageous over a wide range of stages. As stage decrease below the CRP, the amount of 0-5 feet deep water decreases. Higher elevation chevrons produced higher and longer sandbars. Over a range of flows and multiple seasons the chevrons were able to maintain a channel between the landward wing and the adjacent river bank. This indicates that this type of structure can provide a consistent amount of sandbar habitat.

To provide an increase in both the amount and overall diversity of depths associated with chevrons, research should be conducted to determine the viability of placing these structures further riverward, and increase the size of the structure. This may possibly increase the size of the sandbar. Sloping crown elevations or smaller chevrons with variable crown elevations, arranged in series may increase the diversity of depths over the full range of discharges.

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DESIGN OF LARGE WOODY DEBRIS STRUCTURES FOR CHANNEL REHABILITATION

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Abstract: Described is a project intended to restore habitats along 2 km of a sand bed stream severely damaged by channel incision. The project consists of placement of large woody debris (LWD) and planting native vegetation. Design and construction of large woody debris structures are described. If successful, this approach will offer significant cost savings over traditional approaches involving stone bank protection structures.

INTRODUCTION

Stabilization of incising channels and their stream corridors can have major, positive ecological effects, particularly when the structures and methods used are designed to address habitat-limiting factors. Nationally, emphasis for stream and lake restoration has slowly increased over the last two to three decades. Additionally, restoration of aquatic systems to a functional state is the primary goal of the Total Maximum Daily Load (TMDL) approach to water quality now being applied nationally. The end-point of current federal water quality goals and new upcoming nutrient criteria is return to a functional ecological state, i.e. an aquatic ecosystem with ecological integrity. Thus, implementation plans focus on remediation of specific impairments. In many severely incised streams, primary impairments are lack of stable habitat including shifting bottom substrate, shallow depths with a lack of pools, and limited woody debris. Canopy and carbon are at a minimum since the stream is separated from its floodplain. Stabilization can improve both habitat potential and water quality.

Current practice for stabilizing watersheds destabilized by channel incision is based on combinations of grade control drop structures, in-channel stone structures, drop pipes, small reservoirs (floodwater retarding structures), and land treatment. Costs for treating an entire watershed range as high as \$750 ha^{-1}, and costs for channel stabilization as high as \$300 m^{-1} for the affected stream reach. Described herein are design and construction of a demonstration rehabilitation project with projected cost less than \$100 m^{-1}. The project is intended to quantify ecological effects of the low-cost measures, which were selected and designed with consideration of existing habitat deficiencies. Plans call for monitoring physical and biological response for up to five years.

STUDY SITE

The study site is located along 2 km of Little Topashaw Creek, a fourth-order stream in the Yalobusha River watershed in north central Mississippi. Contributing drainage area is about 37 km^2. Floodplain stratigraphy is characterized by dispersive silt and clay soils underlain by sand that overlies the consolidated cohesive material (Adams 2000). Sandy deposits are often found along the bank toe. The channel is tortuous, with an average sinuosity of 2.1, an average width of about 35 m, and an average depth of 6 m. At least two abandoned meanders suggest recent natural neck cutoffs. Channel bed materials are comprised primarily of sand with median sizes between 0.2 and 0.3 mm. However, cohesive materials occur as massive outcrops and as gravel-sized particles. Available evidence suggests mean width has increased by a factor of 4 to 5 since 1955. A geomorphic evaluation performed immediately prior to construction (Wallerstein 2000) indicated that the downstream end of the reach was in the aggradational stage V of the Simon (1989) conceptual model of incised channel evolution, while the middle part of the reach was stage IV, and the upstream fourth of the reach was still degrading (stage III). A knickpoint was located between zones classified as stage IV and stage III, with thalweg slopes ~0.003 upstream of the knickpoint and ~0.002 downstream. In general, concave banks on the outside of meander bends are failing by mass wasting subject to basal endpoint control, and sand is accreting on large point bars opposite failing banks (Figure 1). Outside of bends, eroding banks frequently invade adjacent cultivated fields, while inside bends and abandoned sloughs are vegetated with a diverse mixture of hardwood trees and associated species.

Large woody debris naturally occurring in the channel was mapped in the Spring of 1999 and 2000 using a differentially-corrected global positioning system. Debris diameter was measured using tree calipers, and in the 2000 census, orientation of tree boles with respect to flow direction was noted. When LWD formations were extremely complex, the perimeter of the formation was mapped rather than individual logs. Results (Figure 2) indicated that the channel contained more debris in 2000, perhaps because the 2000 map included smaller logs. Reach-mean debris density was 40.5 logs km^{-1} in 1999 and 60.5 logs km^{-1} in 2000 (formations omitted). Debris density was greatest in channel segments immediately downstream from the knickpoint. Debris stability during the period between the two maps was related to LWD orientation and channel evolution. Only about 29% of the debris mapped in 1999 remained in the same location in 2000, but about 70% of this debris was oriented roughly parallel to the flow direction with the butt pointing upstream. Debris stability was generally higher in stage V segments than in actively evolving stage III or IV segments.

Hydrologic data for the study reach is limited. A large storm in June 1999 produced overbank flooding along the reach, and a USGS gage (07282075) located on a larger stream about 1 mile downstream recorded a peak 11 m above base stage. Stage records obtained since this project was initiated indicate that storm hydrographs are typically brief (< 1 day) and have rise times on the order of 6 hours.

DESIGN AND CONSTRUCTION

Structural measures for stream corridor rehabilitation must be selected and designed to harmonize with the dominant geomorphic processes. For ecosystem rehabilitation, they must address the major factors inhibiting natural recovery. Accordingly, this project was designed to accelerate evolution of the existing system toward a sinuous two-stage channel with wooded berms that could be classified as Stage VI (Simon 1989). Bank stabilization structures made from large woody debris instead of stone were placed along the toe of eroding banks. The large woody debris structures (LWDS) were designed to resist displacement by interlocking, keying-in to banks, anchoring, and by inducing sediment deposition. The LWDS were intended to accrete and retain sediment and organic matter input both from adjacent mass wasting and material transported into the reach from upstream. Crest elevations were set high enough to stabilize existing near-vertical banks failing by mass-wasting (Simon 1998) by building a berm at the toe and preventing episodic cleanout of failure blocks. Since these structures will rapidly decompose in the humid, temperate climate, they are intended to provide suitable habitat for invasion of sediment deposits by plants that will secure and stabilize the channel margins over the longer term (Jacobson *et al.* 1999). In addition, since studies of degraded streams across the region have shown that habitat diversity (Shields and Smith 1992), invertebrate species richness and abundance (Cooper and Testa 1999), and fish species richness (Shields *et al.* 1998a) are associated in a positive fashion with LWD density, addition of LWDS should improve aquatic habitat and facilitate ecological recovery. Additional elements of the project include planting selected native plant materials to rapidly stabilize accreted deposits and gullies formed by runoff passing over the top of banks.

Design of LWDS was based on concepts from Edminster *et al.* (1949), Mott (1994), Abbe *et al.* (1997), Derrick (1997) and others adapted to our region (Figure 3). Structures simulate stable configurations of naturally-occurring debris (Wallerstein *et al.* 1997). Bed material gradations and thalweg profiles were available for design, as were limited cross-section survey data collected in 1997 and 1999.

About 1500 m of eroding banks were selected for LWDS protection. LWDS were constructed using either woody debris (~10%) or living trees (~90%) harvested from designated areas including the channel. Living trees were larger than 200 mm diameter at breast height (DBH). Living trees were harvested by grubbing in order to retain root balls and crowns intact. LWDS were constructed by stacking trees as shown in Figure 3. Members running across the flow direction ("key members") were ~9 m long and were keyed into the bank toes when bank slopes were gradual enough to permit key trench excavation. LWDS crest elevations were specified as either 2.4 m or 3.6 m above the adjacent streambed based on eroding bank height and channel alignment, but constructed LWDS were slightly lower. An average of 16 trees were used per LWDS. Structures were spaced to create nonuniformity, which is valuable for physical habitat recovery (Shields *et al.* 1998b), but aligned to enhance log stability and sediment deposition. In general, structures extended about 15 m in the streamwise direction, about 5 m transverse to the stream, and were spaced about 14 m apart. About one LWDS was constructed to protect each 20 m of eroding bank (Figure 2), which represented an order of magnitude increase in LWD loading.

For design, forces acting on the LWDS were partitioned into buoyancy and fluid drag. The buoyant force acting on each LWDS was computed using the formula:

$$F_b = \{\gamma_w (S_{wood}\, \Sigma \text{ volume of LWD} - \Sigma \text{ volume of displaced water})\} \qquad (1)$$

Where F_b = net buoyant force in N, S_{wood} = specific gravity of wood, and γ_w = specific weight of water in N m^{-3}. LWD stems were assumed to have volumes equal to cylinders with diameters equal to the mean DBH. This assumption overestimates LWD volume because it neglects stem tapering, but this factor is balanced by the volume of branches. The specific gravity of LWD was determined by collecting 89 samples of naturally-occurring debris prior to construction. Samples were weighed and volumes were determined by measuring the volume of water displaced by submerging each sample. Specific gravities were determined for *in-situ* conditions, after soaking in water for ten days, and after drying in an oven at 50^o C for ten days. *In-situ* S_{wood} varied from 0.30 to 1.39 for dead trees and from 0.67 to 1.14 for living trees. Means and standard deviations for dead and living trees were 0.82 ± 0.21 and 0.96 ± 0.16, respectively. Dried samples averaged about 73% lighter and soaked samples about 137% heavier than in-situ conditions.

For design, the depth of flow required for the LWDS to float (treating the structure as a unit) was computed by setting the weight of displaced water equal to the weight of wood. The volume of displaced water was determined by integrating the submerged volume of LWD using an approach similar to that of Braudrick and Crawford (2000). Root balls were treated as cylindrical disks for key members, but neglected for racked members. The weight of soil within root balls was neglected in order to be conservative. These depths were compared to those predicted by a steady flow model (Copeland *et al.* 1998) for typical cross sections using discharges determined for the two- and five-year return interval events using regional regression formulas (Landers and Wilson 1991). Results (Figure 4) indicated that LWDS stability is sensitive to wood density. Flow depths for frequent events are not adequate to submerge the upper portions of the structure, where the heavier parts of the members are concentrated. LWDS comprised of typical materials should be stable during frequent events. Discharges equivalent to the two-year event are required to float the LWDS if the specific gravity of wood = 0.75. Buoyant forces will be counteracted initially by the weight of fill in key trenches and by four earth anchors placed on opposite corners of each structure and load tested to a minimum of 4.4 kN. Anchors on opposite corners are attached by 6 mm wire cable. After a few flow events, buoyant forces should also be counteracted by the weight of sediments deposited on LWDS members.

The drag force on the LWDS was computed by:

$$F_d = 0.5\ V^2 A \rho_w C_D \qquad (2)$$

Where F_d = drag force in N, V = approach flow velocity in m s^{-1}, A = area in m^2 of LWDS projected in the plane perpendicular to flow, ρ_w = density of water in kg m^{-3}, and C_D = drag coefficient. Approach velocities for the two-year event were computed using the Manning equation (V_{mean} = 1.0 m s^{-1}) and verified using output from SAM (Copeland *et al*. 1998) and typical cross-sections (V_{mean} = 0.6 m s^{-1}). For design, the mean velocity of 1.0 m s^{-1} was increased by a factor of 1.5 to allow for higher velocities on the outside of bends. Drag coefficients were computed using the empirical formula for LWD formations presented by Shields and Gippel (1995), and ranged from ~0.7 to 0.9. Drag forces are expected to rapidly diminish with time during the first few high flow events as patterns of scour and deposition reshape the local topography (Wallerstein *et al.*, In Review). Results of LWDS force analysis for five-year discharge conditions are summarized in Table 1.

Materials available for LWDS construction were limited to LWD presently in the channel and trees growing in patchy stands on the floodplain. No clearing was permitted within 10 m of top bank. There was considerable uncertainty prior to construction regarding the quantity of LWD required to complete the project, and the area needed for harvesting the required materials. Regional data collected by Downs and Simon (1999) indicated stem densities of about 100 ha^{-1} and 800 ha^{-1} for trees with diameter > 30 and 18 cm, respectively. Accordingly, given a minimum DBH of 20 cm and assuming an average DBH of 25 cm, we estimated about 50 LWDS would be needed to protect 1500 m of eroding bank, which would require a total of about 1,200 trees harvested from 5 to 10 ha of forest. The finished project consisted of 72 structures built with about 1,200 trees, but these were obtained by clearing only about 3.4 ha. Cleared areas were primarily zones such as fencerows and ditches that landowners wanted cleared for cultivation.

Table 1. Computed Forces Acting on Submerged 2.4-m high LWDS (discharge = 57 $m^3 s^{-1}$ racked member diameter = 30 cm, key member diameter = 40 cm, S_{wood} = 0.75).

Quantity	Magnitude, kN
Total weight of LWD	115
Weight of displaced water	126
Submerged weight of backfill in key trenches	7
Force due to earth anchors	18
Drag force	14

Effects of rehabilitation measures on bed-material sediment transport were estimated by applying the SAM computer routine (Copeland *et al*. 1998) to a data set comprised of 11 cross sections. The total force option was used for determination of composite hydraulic properties, and the Yang equation was used to compute bed-material sediment load. Manning n values were set at 0.07 for vegetated banks and 0.04 for the central portion of the channel. These values reflect increases of about 30% to allow for effects of sinuosity (Chow 1959). At the estimated two-year discharge, the computed sediment transport capacity of the existing channel upstream from the project was about 1,200 kg s^{-1}, increased in the reach containing a 0.8-m high headcut, and then declined to about 800 kg s^{-1}. This pattern is consistent with the channel evolution model proposed by Simon (1989). The influence of LWDS was simulated by increasing the Manning n values to 0.15 for segments of the cross section covered by LWDS. Resulting composite n-values computed by SAM were consistent with values obtained using the approach of Shields and Gippel (1995). This modification influenced only a small fraction of the wetted perimeter, but reduced computed sediment transport for the reach by 50%. Effects for greater discharges were slightly less.

MONITORING

The Little Topashaw Creek corridor is being used as a study site for a variety of projects, all of which should provide insight into rehabilitation and response. Precipitation, stage and discharge and routine water quality parameters are continuously monitored. Thalweg profile and 39 strategically located cross sections are surveyed each winter. Fish, macroinvertebrates, and physical aquatic habitat information are collected from zones located upstream, within, and downstream from the study reach in Fall and Spring. Special studies are proceeding to describe the effects of local dewatering on stability of steep banks, the effects of LWDS on flow depths and velocities, and the effects of planting large native grasses on streamside gullies. A census of terrestrial plants was performed prior to construction, and bank soils data were collected to confirm a recently developed protocol for prediction of willow planting success.

CONCLUSIONS

LWDS hold considerable potential as low-cost measures for rehabilitating small (drainage area < 200 km^2) sand-bed streams damaged by channel incision. Successful application will result in decelerated erosion and ecosystem recovery. However, the structures are vulnerable after they are installed but before sufficient sediment has deposited within the woody debris matrix to counteract buoyant forces. Another hazardous period will occur when the structures decompose and disintegrate if colonization of the sediment deposits by woody vegetation is not rapid. Finally, the long term outlook depends upon a watershed-wide strategy to control grade and upstream sediment sources so that the new morphology developed by the LWDS will approach dynamic equilibrium with water and sediment inputs.

ACKNOWLEDGMENTS

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Figure 1. Typical bank erosion in study reach, 1999.

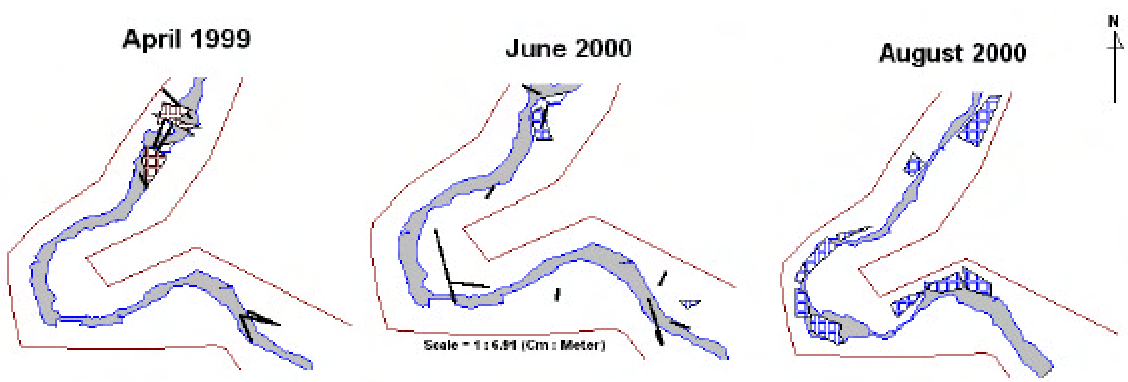

Figure 2. Map of selected segment of study reach showing top banks, water's edge at base flow, LWD formations and individual logs (April 1999 and June 2000) and LWDS (August 200)

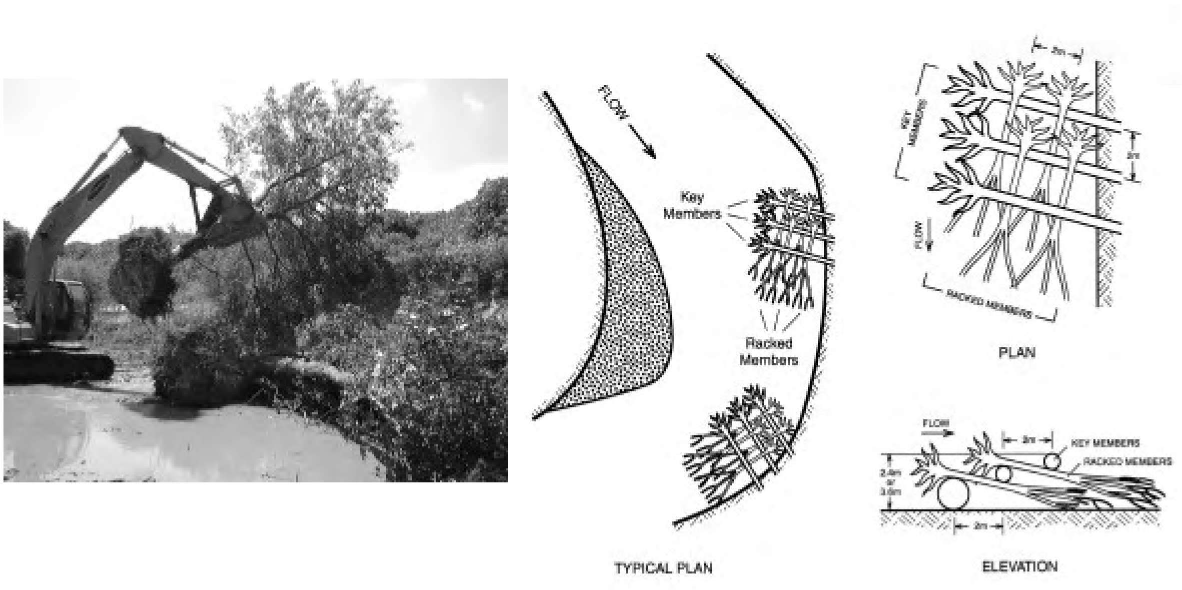

Figure 3. Typical plan and elevation of large woody debris structures. Inset photo shows LWDS under construction**.**

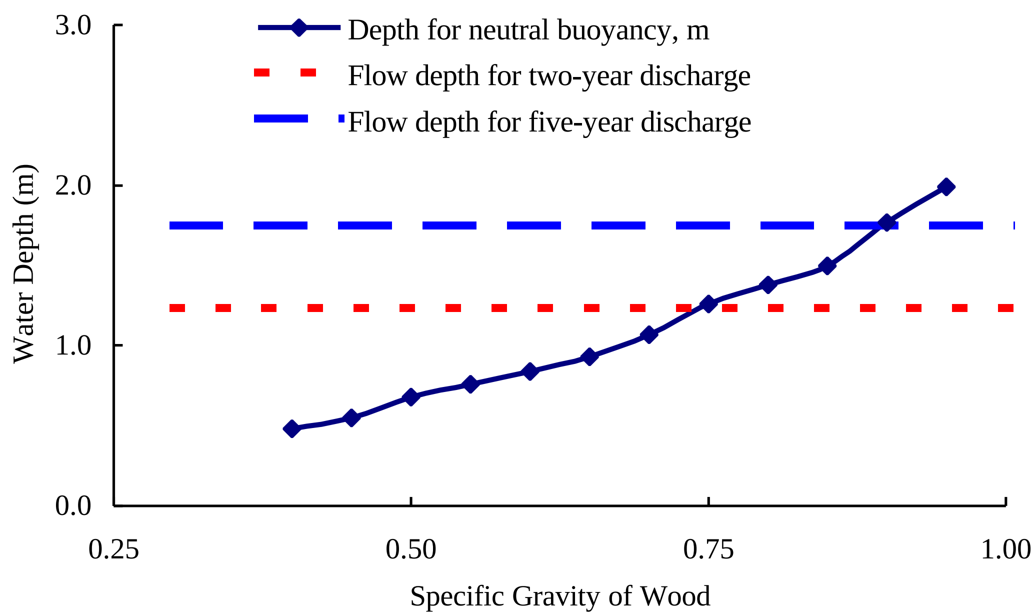

Figure 4. Flow depth required for buoyant force on large woody debris structure to equal weight of wood as a function of flow depth.

COMPUTATION OF BANKFULL AND FLOOD-GENERATED HYDRAULIC GEOMETRIES IN EAST PLUM CREEK, COLORADO

By Eleanor R. Griffin, Hydrologist, U.S. Geological Survey, Boulder, Colorado and J. Dungan Smith, Hydrologist, U.S. Geological Survey, Boulder, Colorado

INTRODUCTION

On June 16, 1965, an extreme flood occurred along East Plum Creek, south of Denver, Colorado. This flood caused the channel to change its form from meandering to braided (unravel) all the way from Larkspur 34 km downstream to its confluence with West Plum Creek at Sedalia (figure 1). The intense rainstorm that caused this flood also resulted in unraveling of Plum Creek from Sedalia to its mouth at the South Platte River, and it caused extensive flooding of the South Platte River through Denver (Matthai, 1969). The geomorphic effects of the flood along Plum Creek, near Louviers, were examined in detail by Osterkamp and Costa (1987), and some of their data were used in this study to reconstruct a flood hydrograph for East Plum Creek (figure 2) and to provide an additional data point for the downstream hydraulic geometry presented here. Most of the rain fell in the East Plum Creek drainage basin. However, consequences of the flood along East Plum Creek had not previously been examined in detail. Our computation of the flood and pre-flood bankfull hydraulic geometries presented here provides a basis for determining the discharge at which this stream began to unravel.

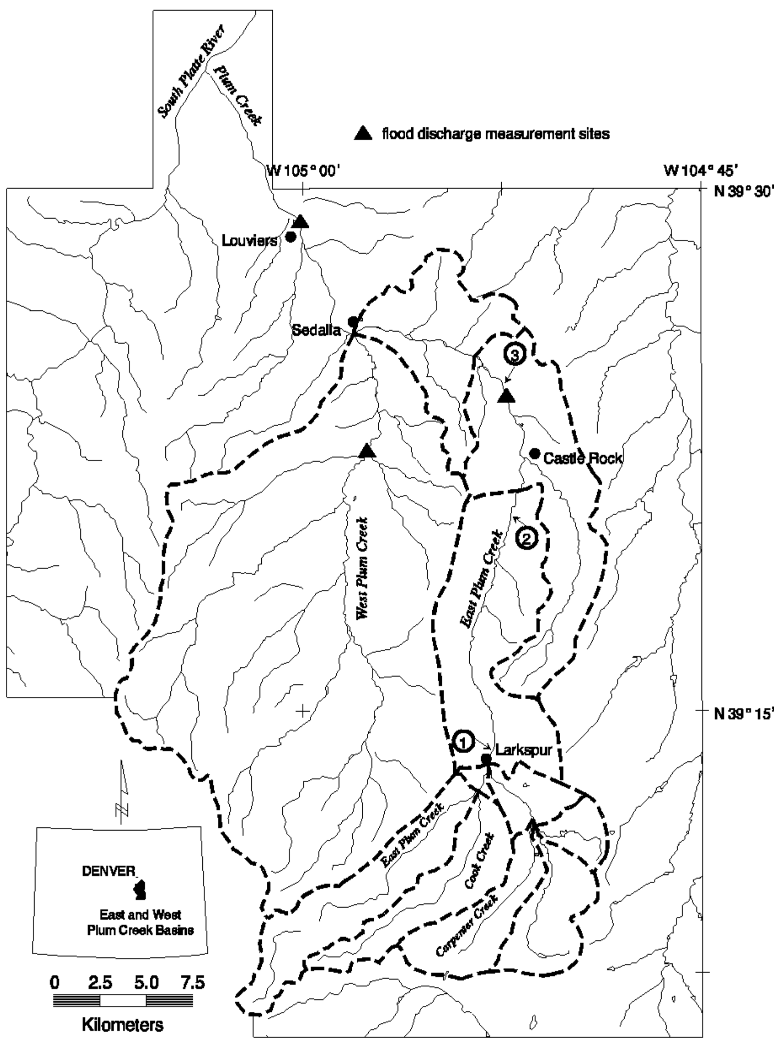

Figure 1. Location of the East Plum Creek drainage basin, south of Denver, Colorado. The dashed lines delineate drainage basin boundaries.

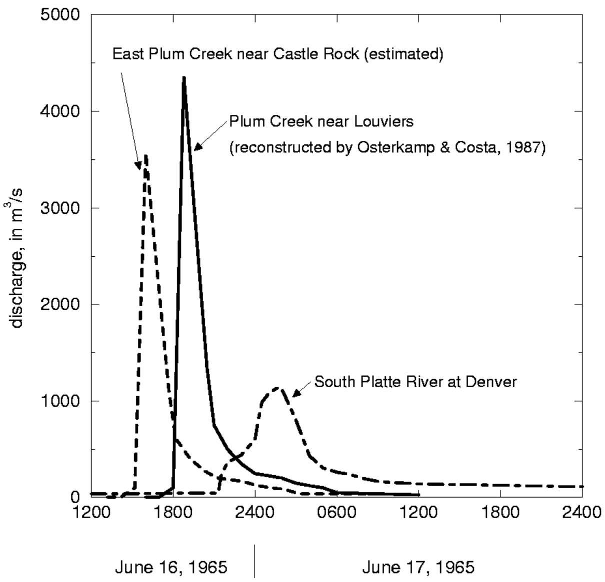

Figure 2. Reconstructed flood hydrographs for Plum Creek and East Plum Creek (our estimate), and the gage record for the South Platte River at Denver.

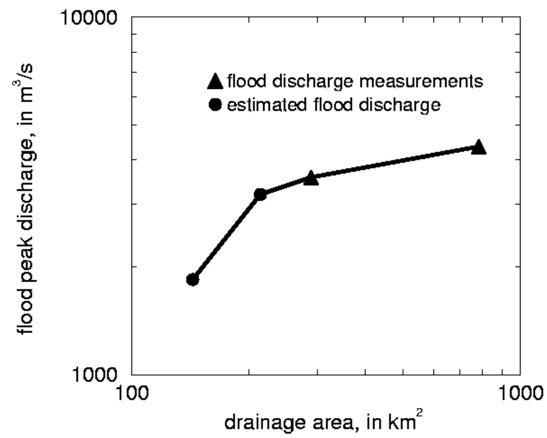

Figure 3. Flood peak discharge along East Plum Creek and at Plum Creek near Louviers.

East Plum Creek flows along major highways from Larkspur to Sedalia, and the Colorado Department of Transportation obtained large-scale aerial photographs (about 1:4,000-scale) of the highway corridor, including much of East Plum Creek, immediately after the flood and at various other times. Using a subset of these photographs, we determined mean channel width and sinuosity both before (May and June, 1956) and immediately after (June 18, 1965) the flood for three long (more than 1 kilometer) reaches. The numbered arrows on the map (figure 1) generally point to the reach locations. These data were then used, along with reach-averaged slopes determined from large-scale digital map data and some assumptions concerning flow conditions, to compute both the flood-generated hydraulic geometry and the pre-flood, bankfull hydraulic geometry.

In order to develop the hydraulic geometries, estimates of both flood peak and pre-flood, bankfull discharges were required for each of the three locations. An indirect measurement of the flood peak discharge was made at location 3, north of Castle Rock (Matthai, 1969). We estimated the flood peaks at upstream locations (1 and 2) using total rain depths and reported timing of rain and flood events (Matthai, 1969), the geometry of the basin, and a calculated soaking depth (figure 3).

We estimated the pre-flood, bankfull discharge at the location north of Castle Rock using available gaging station records for East Plum Creek at Castle Rock (station number 06708750) and Plum Creek near Louviers (station number 0670950). Both streams have highly variable flow with a low base flow, and bankfull discharges are associated with spring snowmelt and summer rainstorm flood peaks. The mean annual peak flow for the 42-year period of record for Plum Creek, excluding the 1965 flood, was assumed to be close to the bankfull discharge near Louviers. We then developed a flood flow frequency curve for Plum Creek using the methods of the Interagency Advisory Committee on Water Data (1982) and determined that the mean annual peak discharge (26.1 m^3/s) is about a 3-year flood. Although only a short (4-year) record is available for East Plum Creek, the 4 annual peaks and average of the daily mean flows are proportional to those of Plum Creek near Louviers, so the mean annual peak for Louviers could be scaled by that proportion to estimate the bankfull discharge of East Plum Creek north of Castle Rock (13.0 m^3/s). The estimated bankfull discharge for East Plum Creek north of Castle Rock also corresponds to about a 3-year flood at this location. The bankfull discharge calculated for location 3 was then compared to drainage area to obtain a relation between bankfull discharge, Q_{bf}, and drainage area, D_A,

$$Q_{bf} = 0.105 D_A^{0.85} \tag{1}$$

In equation 1, the value of the exponent was determined from the 1965 flood peak discharge as a function of drainage area along East Plum Creek. We then used equation 1 to estimate bankfull discharge along East Plum Creek upstream from Castle Rock (figure 4).

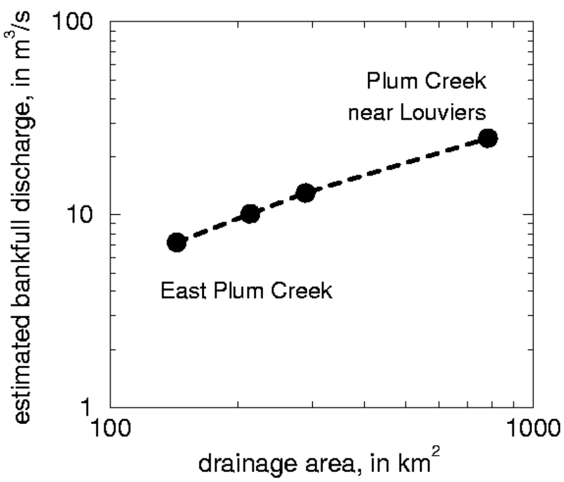

Figure 4. Estimated pre-flood bankfull discharge at the three locations along East Plum Creek and for Plum Creek near Louviers.

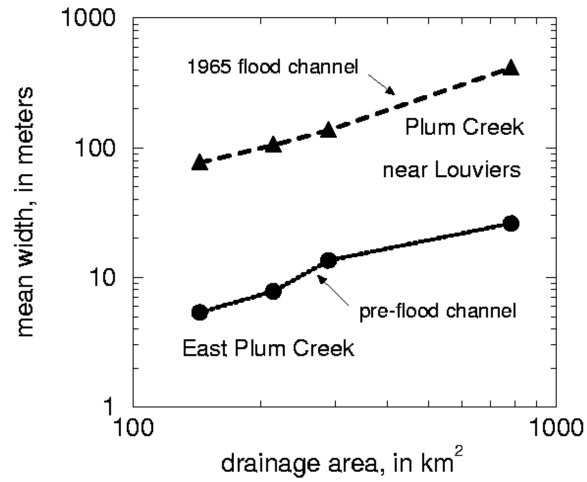

Figure 5. Mean flood and pre-flood bankfull widths determined from large-scale aerial photographs.

HYDRAULIC GEOMETRY

Flood channel: We determined mean channel widths from large-scale aerial photographs and found they increase systematically as a function of drainage area for both the pre-flood and flood-generated channels (figure 5). While mean width, slope and flood peak discharge are known or can be estimated directly, mean flow depth and channel roughness are unknown. However, flow can be assumed to have been about critical during the flood peak because of the presence of upstream propagating antidunes on the bed, clearly indicated by wave trains seen in flood photographs. Another indication flow was close to critical is the degree of unraveling that occurred during the short duration of the flood peak. According to Osterkamp and Costa (1987), the valley floor near Louviers was inundated for only about 2.5 hours.

Upstream propagating antidunes develop when the Froude number (F_R) is close to critical (i.e., 1). As the antidunes build, form drag increases, increasing the depth until the waves break. Once the antidunes are fully developed, a backwater forms upstream, where flow is less than critical. When the antidunes break, they wash out, and flow becomes supercritical until the antidunes begin to build again. Therefore, where there are fully developed upstream propagating antidunes, the Froude number remains about 1 on average. Specification of the flow Froude number in this manner is more accurate than guessing the roughness or trying to calculate roughness caused by the upstream propagating antidunes.

Reach-averaged mean flow velocity, U, can be calculated for turbulent flows that are steady and horizontally uniform in the mean using the equation

$$U = \frac{u_*}{k}\left(\ln\left(\frac{h}{z_o}\right) - 0.74\right) \tag{2}$$

when channel roughness, z_o, and mean flow depth, h, are known. In this equation, u_* is the shear velocity, $(ghS_o)^{1/2}$, k is von Karman's constant, 0.408, S_o is the mean channel slope, and g is acceleration due to gravity. Equation 2 is a quasi-logarithmic velocity profile, where a parabolic profile in the outer part of the flow (away from the bed) is matched to a logarithmic profile for the inner part of the flow (near the bed) at two tenths of the depth (Wiberg and Smith, 1991). However, mean flood flow depth and velocity are unknown, so we applied an iterative approach to find the mean depth and velocity that resulted in the known discharge when the Froude number was 1. We first assumed a depth, then solved the equations for mean flow velocity (equation 2) and Froude number, where

$$F_R = \frac{U}{\sqrt{gh}} = 1 \tag{3}$$

to find the channel roughness that resulted in a Froude number of 1 at that depth and velocity.

The equation for mean flow velocity was then substituted into $U = Q / A$ to develop a relation for discharge that could be solved using mean channel width, b, and slope, S_o, along with the depth and z_o found using equations 2 and 3, above, which is

$$Q = b \cdot \frac{\left(gh^3 S_o\right)^{1/2}}{k}\left(\ln\left(\frac{h}{z_o}\right) - 0.74\right) \tag{4}$$

By iteratively adjusting the depth, the calculated discharge ultimately matched the measured (north of Castle Rock) or estimated (south of Castle Rock and north of Larkspur) discharge, and the Froude number was 1 (table 1). Calculated roughness values, z_o, were converted to an equivalent Manning's n using the calculated flow depths and discharges. The channel roughness and mean slope counteract each other, so that in the steeper reach south of Castle Rock, the calculated roughness is higher than in the other two reaches.

Table 1. East Plum Creek flood-generated hydraulic geometry.

Location along East Plum Creek	Flood peak discharge (m^3/s)	Mean channel width (m)	Calculated values				Slope	Sinousity
			Mean depth (m)	Mean velocity (m/s)	z_o (cm)	Manning n		
North of Castle Rock	3,570	136.1	4.12	6.36	1.4	0.032	0.0068	1.04
South of Castle Rock	3,200	105.2	4.55	6.68	2.7	0.038	0.0087	1.05
North of Larkspur	1,850	76.6	3.90	6.19	1.2	0.032	0.0065	1.04

Pre-flood, bankfull flows: For pre-flood, bankfull flows, only the discharge, mean channel width, and average slope are known for each location along East Plum Creek. In order to calculate the hydraulic geometry, we examined various forms of roughness, including that produced by bedload transport of sediment, and determined their effects on flow depth and velocity. We first looked at effects of bed grain roughness alone, then calculated effects of saltating particles and dunes on the bed. This procedure enabled a physically-based assessment of the effects of roughness elements rather than relying on a guessed roughness parameter. The most likely conditions are presented in a table at the end of this section.

A minimum possible bankfull depth was first calculated by estimating roughness for steady, horizontally uniform flow over a poorly sorted bed as $(z_o)_{sf} = 0.1 * D_{84}$ (Whiting and Dietrich, 1989). For East Plum Creek, D_{84} was estimated as D_{50} plus one standard deviation of the bed sediment sample sizes from Plum Creek provided by Osterkamp and Costa (1987). Bed material in East Plum Creek is assumed to be similar to that of Plum Creek because of similarities in sediment sources and mean channel slope along both streams. The value of D_{84} is 0.3 cm, so the initial estimate of $(z_o)_{sf}$ was 0.03 cm. Equation 4 was then solved to find the mean flow depth, h, from the estimated roughness, bankfull discharge, and mean channel width and slope determined from the aerial photographs and digital map data. At the location north of Castle Rock, the mean flow depth needed to result in a discharge of 13.0 m^3/s when b = 18.1 m, S_o = 0.0063, and $(z_o)_{sf}$ = 0.03 cm was 0.33 m. Converting the roughness to an equivalent Manning's n at that depth results in n = 0.017, a lower than expected value for this natural channel (Barnes, 1967). The roughness parameter used in this calculation only accounts for grain roughness. Therefore, the depth calculated here (0.33 m) is a minimum possible depth of flow for the pre-flood bankfull discharge.

Bedload transport occurs when the boundary shear stress, τ_b, is greater than the critical shear stress, τ_{cr}, for the particles present on the bed. The critical shear stress for a given particle diameter, D, can be determined from Shield's diagram, which gives non-dimensional critical shear stress, $(\tau_*)_{cr} = (\tau)_{cr} / (\rho_s - \rho)gD$, as a function of the friction Reynolds number, $R_* = u_*D / v$, where v is the kinematic viscosity, 0.0131 cm^2/s. When the bed shear stress is high enough to cause saltation of particles on the bed, the magnitude of the roughness is proportional to the thickness of the saltating layer, δ_B (Wiberg and Rubin, 1989). The thickness of the saltating layer can be determined from a relation between the transport stage, $T_* = \tau_b / \tau_{cr}$, and δ_B /D (Wiberg, 1987). Also, the maximum saltation height occurs with particles of size D_{50} or slightly smaller, because particles larger than D_{50} will roll rather than hop. Therefore, D_{50} is used rather than D_{84} to calculate the saltation height.

Wiberg (1987) developed an expression for δ_B /D as a function of the non-dimensional shear stress from a theoretical model of saltation and bedload transport, which is

$$\frac{d_B}{D} = \frac{a_1 t_*}{1 + a_2 t_*} \qquad (5)$$

where a_1 and a_2 are coefficients that vary with grain size, and τ_* is the non-dimensional shear stress determined from the flow depth, slope, and grain size. The values of the coefficients were determined from a best-fit of trajectory heights calculated for specific grain diameters using the model of saltating particles and bedload transport (Wiberg, 1987). The saltation height determined by this method is 0.97 cm. The relation between δ_B and the roughness parameter, z_o, is $z_o = 0.1 * \delta_B$ (Wiberg and Rubin, 1989), which results in a z_o of 0.097cm, or about 0.1 cm. Using this value of roughness along with the known bankfull width, slope and discharge, equation 4 can be solved again to find mean depth. This procedure was applied iteratively until the depth calculated from $z_o = 0.1 * \delta_B$ led to the same value of δ_B using equation 5. The calculated mean flow depth at the location north of Castle Rock is 0.37 m, the mean flow velocity is 1.9 m/s, and equivalent Manning's n is 0.021.

Ripples begin to form at a transport stage of about 2 to 2.5 (Wiberg and Smith, 1989), and an upper limit for purely bedload transport of well-sorted sediment is a transport stage of about 20 (Wiberg, 1987). The stability field for dunes is with transport stages from 5 to 36 (Mohrig, 1994). With a mean flow depth of 37 cm, the transport stage north of Castle Rock is about 10.4, well within the range in which dunes are likely to be present on the bed. Therefore, the next step is to assume there are dunes on the bed and calculate a total roughness including form drag due to the presence of dunes.

The maximum height of the dunes, H, would be about 1/5 the depth (Wiberg and Smith, 1989), or 7.4 cm, and the wavelength, λ, would be on the order of 20 times the height (i.e., H/λ = 1/20; Mohrig, 1994). With dunes present on the bed, the total shear stress is

$$(t_b)_T = t_{sf} + t_D \qquad (6)$$

where τ_{sf} is the component of shear stress due to skin friction and τ_D is the component due to form drag, where

$$t_D = \frac{F_D}{lb} = \frac{1}{2} \cdot \frac{rC_D U_{ref}^2 H}{l} \qquad (7)$$

In this equation, F_D is the drag force and C_D is an empirically-determined drag coefficient.

In separated flow, the value of C_D is 0.212 (Smith and McLean, 1977). The equation for the reference velocity, U_{ref}, (the velocity that would be present if the dune were removed) can be integrated from z_o to the dune height, H, to get the equation

$$U_{ref} = u_* \left[\frac{1}{k} \left(\ln \frac{H}{(z_o)_{sf}} - 1 \right) \right] \qquad (8)$$

Following the procedure of Smith and McLean (1977), the ratio of total shear stress, $(\tau_b)_T$, to shear stress due to skin friction, τ_{sf}, can be determined by substituting equation 8 into equation 7, and dividing through by τ_{sf}.
Using (z_o)sf = 0.1 * δ_B, or 0.10 cm (Wiberg and Smith, 1989), velocity profiles for the inner (next to the bed) and outer layers can be matched at the estimated dune height, 7.4 cm. Rewriting $(\tau_b)_T$ and τ_{sf} in terms of velocity at the point where the inner and outer profiles match yields

$$\frac{\left(\ln\frac{H}{(z_o)_{sf}}\right)^2}{\left(\ln\frac{H}{(z_o)_T}\right)^2} = 1 + \frac{C_D}{2k^2}\left[\ln\frac{H}{(z_o)_{sf}} - 1\right]^2 \frac{H}{l} \qquad (9)$$

(Smith and McLean, 1977; Kean, 1998). Equation 9 was then solved iteratively to find the values of depth, $(z_o)_{sf}$, and $(z_o)_T$ that resulted in the estimated bankfull discharge. For the reach north of Castle Rock, the calculated value of $(z_o)_T$ was 0.20 cm and $(z_o)_{sf}$ was 0.11 cm. Depth calculated from $(z_o)_T$ and the mean channel width and bankfull discharge at this location is 40 cm, and the value of Manning's *n* increases to 0.024 (table 2).

Table 2. East Plum Creek calculated bankfull hydraulic geometry.

Location along East Plum Creek	Bankfull discharge (m^3/s)	Mean channel width (m)	Calculated values						Slope	Sinousity
			Mean depth (m)	Mean velocity (m/s)	$(z_o)_{sf}$ (cm)	$(z_o)_T$ (cm)	Manning n			
North of Castle Rock	13.0	18.1	0.40	1.78	0.11	0.20	0.024		0.0063	1.13
South of Castle Rock	10.1	8.3	0.54	2.25	0.12	0.24	0.025		0.0073	1.25
North of Larkspur	7.2	6.9	0.53	1.96	0.11	0.23	0.025		0.0055	1.18

Bar form drag also may have been a factor in the channel roughness, but pre-flood bed topography is not known. However, because the pre-flood sinuosity was low (about 1.2), bar form drag is expected to be low. Also, either form drag from dunes or bar form drag is likely to dominate the roughness, and the value of $(z_o)_T$ determined using equation 9 is likely to be about the maximum possible roughness for the pre-flood channel at bankfull discharge.

<u>At-a-station hydraulic geometry</u>: The calculated bankfull and flood-generated hydraulic geometries at each of the three locations along East Plum Creek (tables 1 and 2) give points from which mean channel width, depth and velocity can be estimated for other flows between bankfull discharge and the discharge of the extreme flood that occurred in June, 1965 (figure 5). In addition, when one of these parameters is known at a particular location, discharge can be estimated for the associated flow. For example, if the reach-averaged width of a flood flow greater than bankfull but less than the 1965 flood peak is known at a location, the discharge associated with that width can be estimated by interpolating between the widths at bankfull and during the flood peak.

CONCLUSIONS

Although the duration of the 1965 flood was extremely short, it established what appears to be a consistent hydraulic geometry for East Plum Creek. The value of the exponent in the relation for flood peak discharge as a function of drainage area (equation 1) led to a reasonable calculation of bankfull discharge along East Plum Creek and the calculation of a pre-flood, bankfull hydraulic geometry. Together, these hydraulic geometries provide a basis for calculating the discharge at which the stream began to unravel.

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TIDAL WETLAND RESTORATION: ACCELERATING SEDIMENTATION AND SITE EVOLUTION IN RESTORING DIKED BAYLANDS

By Kevin Knuuti, P.E., Hydraulic Engineer, U.S. Army Corps of Engineers, San Francisco, California

Abstract: In the 1800s, farmers in the San Francisco Bay area constructed dikes on bayfront tidal salt marshes (baylands) to hold back the flood of the tides. Over time the diked baylands dried, people cultivated or developed the land, and the land gradually subsided. In many areas, the former baylands subsided several feet below the mean lower low water tidal datum. Growing environmental awareness in the last twenty years has led to a desire to restore some of the 170,000 acres of tidal baylands that were lost due to diking, farming, and development. Restoring these former baylands is more complicated than simply opening them back up to tidal flooding. Because of the amount of settlement behind the dikes, opening the baylands to tidal flooding would result in salt-water ponds that are not suitable habitat for species that reside in tidal wetlands. Natural sedimentation would eventually create the desired habitat, but this process would be extremely slow. To accelerate the sedimentation process, a team of local, state, and federal agencies proposed partially filling the subsided lands with dredged material from navigation projects throughout the San Francisco Bay area. The U.S. Army Corps of Engineers (Corps), San Francisco District, is actively involved with two such restoration projects: Sonoma Baylands and Hamilton Army Airfield (Hamilton).

The Corps initiated the restoration of Sonoma Baylands, a 348-acre site on the northern edge of San Pablo Bay, in 1995 to demonstrate the use of dredged material in accelerating restoration of diked baylands. Partially filling the subsided lands with dredged material prior to opening the site to tidal action and estuarine sedimentation is allowing the tidal marsh system to develop naturally over a relatively short time period while minimizing construction costs. The Corps has been monitoring the evolution of the Sonoma Baylands site since it was opened to tidal action in 1996. We are analyzing the data that have been collected, along with our observations, and are applying the lessons we learn to the design of the Hamilton wetland restoration site.

Hamilton is a 988-acre site on the western edge of San Pablo Bay. In 1998, the Corps, in partnership with the California State Coastal Conservancy, completed a feasibility study for restoring the site to tidal and seasonal wetlands. The Corps is currently in the design phase of this restoration project and plans to use 10,600,000 cubic yards of dredged material to accelerate the natural sedimentation process. Site preparation and dredged material placement will take six years and will cost approximately $56,000,000. The restored wetland site will consist of approximately 800 acres of tidal wetland, 64 acres of seasonal wetland and 85 acres of transitional upland area. This paper describes the lessons learned from Sonoma Baylands and the design process and problems associated with the Hamilton wetland restoration.

INTRODUCTION

In the mid-1800s, farmers and developers in the San Francisco Bay area began diking off portions of San Francisco Bay (including areas of South Bay, San Pablo Bay, and Suisun Bay) for agricultural use and urban development. Currently, over 170,000 (seventy percent) of the Bay's original 240,000 acres of baylands have been diked and separated from tidal action (Goals Project, 1999). Of the 170,000 acres, approximately 50,000 acres exist as "diked historic baylands" or ponds, fresh-water wetlands, and farmland that have never been dredged or filled (Bay Institute, 1987).

Recognizing the need to manage its coastal environmental resources, the State of California began regulating Bay Area coastal activities in 1965 and formed the California State Coastal Conservancy (SCC) in 1976. Since its inception, the SCC has, among other things, sought to acquire, protect, and restore the Bay Area's remaining diked historic baylands. Around 1990, the Sonoma Land Trust and the SCC began working together to restore the 348-acre Sonoma Baylands site to tidal salt marsh. The SCC sought assistance from the Corps and, in 1993, the Corps began working on the design and execution of the restoration project. The Corps finished engineering and construction of the Sonoma Baylands project in 1996 and is currently monitoring the evolution of the site. Since 1998, the Corps has been working with the SCC on the Hamilton project. In an attempt to improve the methods used to restore tidal wetlands in the Bay Area, the Corps is using the lessons it learns from Sonoma Baylands to improve its design at Hamilton.

SONOMA BAYLANDS

Pilot Project: The Sonoma Baylands Wetland Demonstration Project (Sonoma Baylands) was conceived as a pilot project under the Coastal America initiative of 1991. The restoration project was multi-purposed, with the goals being "to restore tidal wetlands, provide habitat for endangered species, expand the feeding and nesting areas of waterfowl along the Pacific flyway, and demonstrate the use of suitable dredged material as a resource, facilitating the completion of San Francisco Bay dredging projects in an environmentally sound manner" (WRDA, 1992). By using dredged material from local navigation channel maintenance dredging projects to partially fill the subsided ground at that project site, the Corps hoped to demonstrate a beneficial reuse of dredged material while accomplishing the goal of habitat restoration. The Sonoma Baylands project was the first Corps project in the Bay Area in which dredged materials were used to accelerate the restoration of a tidal salt marsh instead of being disposed of at an open water or upland disposal site. This restoration process is now part of the Corps' Long-Term Management Strategy (LTMS) for dredging and dredged material disposal in the San Francisco Bay/Sacramento-San Joaquin Delta Estuary.

Basis for Design: The Sonoma Baylands project design was initially based on a detailed reevaluation of completed local tidal marsh restoration projects, some of which were completed over twenty years ago. Features of these projects, some intentional and others incidental, that had been shown over the years to be beneficial were identified and used as the basis for the Sonoma Baylands design. Sonoma Baylands is thus considered to be the first second-generation design in the Bay Area. One of the lessons learned from studying the previous restoration sites was that allowing natural forces to shape the final form of a marsh restoration site results in the greatest potential for long-term success.

Due to decades of subsidence, the ground elevation at the project site was up to six feet below the mean higher high water (MHHW) tidal datum, the original marsh elevation. To eliminate the possibility of flooding adjacent lands, the Corps constructed a levee around the perimeter of the restoration site. Levee material was excavated from within the restoration site, as a cost-saving measure, and resulted in some areas of the restoration site being ten feet lower than the original marsh elevation. The project designers determined that placing clean dredged material from nearby shipping channels would be the best method of restoring the site to a suitable elevation for marsh development. Partially filling the site prior to restoring it to tidal action would accelerate the development of the marsh and using dredged material would avoid the transportation costs and the adverse environmental effects of aquatic disposal that otherwise would have occurred. Partial filling would also allow natural sedimentation to complete the restoration of the site, enabling a complex system of natural channels to form over time. Based on the anticipated rate of sedimentation in the restoration site, the design team suggested filling the site to approximately two feet below MHHW. This provided a beneficial use of nearly 2,000,000 cubic yards of dredged material, enabling it to be disposed of in an ecologically sound manner (U.S. Army Corps of Engineers, 1994).

Design Features: To decrease the possibility of wind waves resuspending recently deposited sediments in the restoration site, the Corps constructed interior peninsulas throughout the site. Originally, these peninsulas were to be permanent features, providing periodic upland habitat throughout the site. Public concerns, however, about predator access to the site resulted in significant alterations to the design of the internal peninsulas. Rather than constructing permanent peninsulas, the Corps constructed the peninsulas with uncompacted soil, placed at an elevation only slightly higher than MHHW. The relatively low elevations, combined with their makeup of loosely placed earth, should allow the interior peninsulas to naturally erode to the final design marsh elevation while continuing to limit the size of wind waves that can form in the site.

The Sonoma Baylands project site is divided into two sections, a small pilot unit and a main unit, each of which is connected to San Pablo Bay by its own natural tidal channel. These natural channels are small enough that they restrict tidal action in the restoration site. In an attempt to minimize disturbance of existing habitat, local resource agencies requested that the channels be left in their natural condition and not enlarged. The design team agreed to this request, thinking that the channels would rapidly scour and widen to the point where they would allow full tidal action in the restoration area.

Offshore from the Sonoma Baylands restoration site is a large expanse of mudflat. The nearshore elevation of this mudflat is higher than the thalwag of the tidal channel between the Bay and the restoration site's main unit. The design team considered the need for dredging a channel through the mudflat, from the main unit's tidal channel to a suitable depth in the Bay, but concluded that the ebb and flow of water through the tidal channel to the site would naturally scour a channel through the mudflat. The tidal channel leading from the pilot unit empties into the Petaluma River, which is deep enough that a significant channel through the river's adjacent mudflat would not be necessary (U.S. Army Corps of Engineers, 1994).

Adaptive Management: The Corps anticipated complete restoration of the Sonoma Baylands site would take approximately twenty years. An important aspect of the design, however, was an adaptive management approach that included monitoring channel and marsh development after completion of initial construction. The goal of the monitoring was to identify areas of the design that needed to be modified early enough to allow us to complete those modifications prior to full restoration of the site. As an assurance that natural sedimentation at the site would not be delayed by narrow channels muting the tidal regime, the Corps established a monitoring program that included measuring water levels continuously and measuring channel cross-sections and channel thalwag profiles every six months. Additionally, numerous transects across the interior of the site were measured and compared to vegetation surveys to monitor sedimentation in the site, interior channel development, and the development of various types of habitat.

Lessons Learned: The Corps has just completed its fourth year of monitoring the development of the Sonoma Baylands site. The most obvious determination has been that the tidal channels connecting the project site to San Pablo Bay have not scoured or widened nearly as quickly as the design team projected. This has resulted in a continuously muted tidal regime in the restoration area and much slower sedimentation than the Corps had anticipated. Originally, the Corps thought that after five years the restoration site would be experiencing a full tidal regime and would have experienced sedimentation to the point that pickleweed (*Salicornia virginica*) would be colonizing a significant portion of the site. The restricting tidal channels have slowed this sedimentation and colonization significantly.

While the process has been slow, the tidal channels between the restoration site and San Pablo Bay have been evolving. Analysis of cross sections (see Figure 1) shows that the channels are experiencing a cycle of deepening and widening. Scour in the channels first deepens them to the point where the channel banks become unstable. Slump blocks then fall into the channels, widening them but also causing them to become shallower. The channel bottoms then scour again and the process continues.

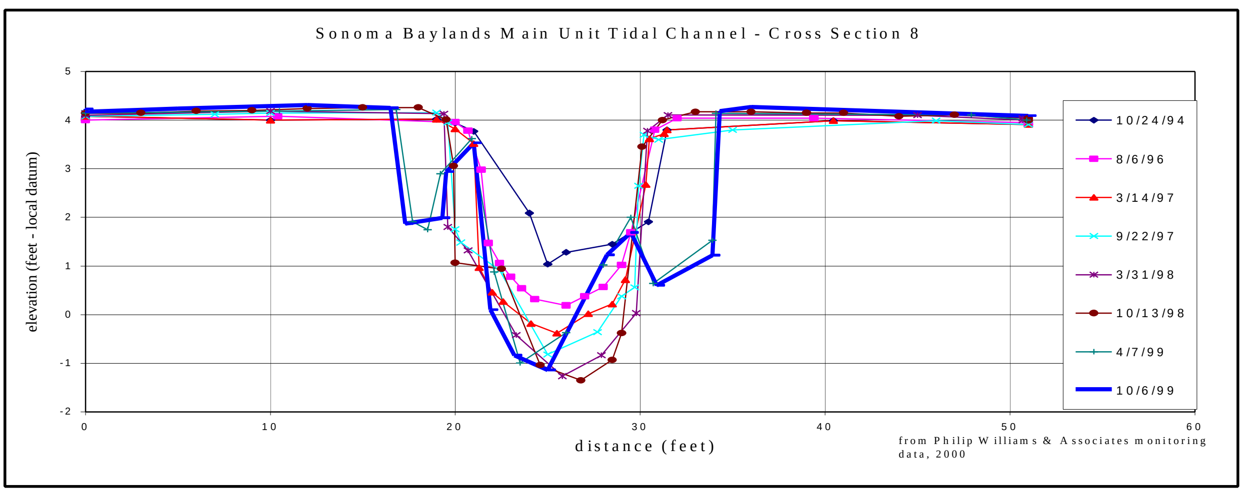

Figure 1 – Sonoma Baylands tidal channel evolution

Slower than anticipated sedimentation in the restoration site has also slowed the formation of interior subtidal channels. While interior channels started to form by April 1999, they have only become visibly noticeable since March 2000 (see Figure 2).

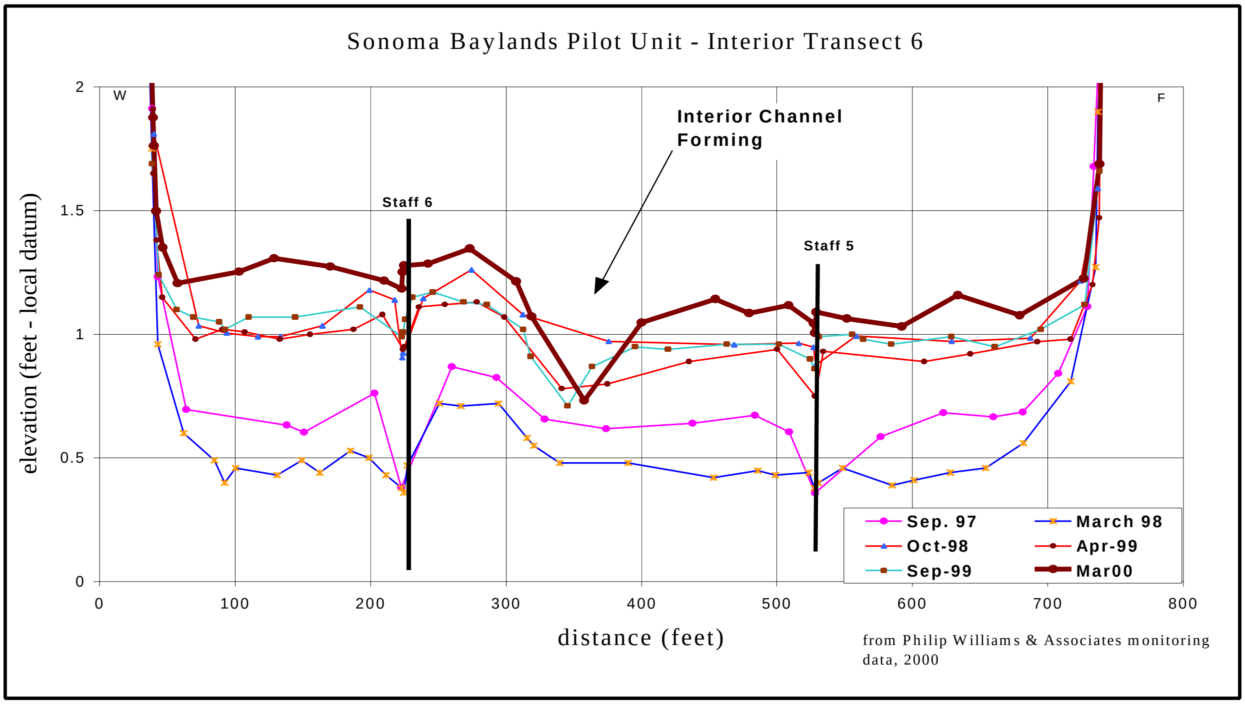

Figure 2 – Sonoma Baylands transect showing the start of interior channel formation

The interior peninsulas have generally been performing as expected. Through an error in construction, these peninsulas were built to a higher elevation than was originally planned. They have reduced wave action in the restoration site but are still allowing wave action that is sufficient to partially resuspend recently deposited sediments. The interior peninsulas are also eroding faster than the design team planned. We do not currently know the reason for the more rapid erosion but suspect that it may be due to the slow sedimentation and muted tidal regime allowing deeper water to persist at the site. This deeper water could result in larger waves at the site with more erosive power than was originally anticipated.

A small, but significant channel has been scouring through the mudflats at the mouth of the main unit's tidal channel. We are not quantitatively monitoring this channel but are observing it to ensure that it is sufficiently conveying the water that enters and leaves the restoration site.

HAMILTON ARMY AIRFIELD

General: The Hamilton Wetland Restoration Project is located on a 988-acre site on the west side of San Pablo Bay, approximately 6 miles south of Sonoma Baylands. Like Sonoma Baylands, the Hamilton site was diked by farmers in the mid-1800s and subsided well below its original marsh elevation of MHHW. Some areas of the site subsided as much as nine feet below MHHW. In the early twentieth century, the U.S. Army (Army) built an airfield on the site. This airfield has been relatively inactive since 1970. The Army is currently in the process of

transferring the airfield site to the SCC as part of the base realignment and closure process. In 1998, the SCC approached the Corps, seeking assistance with its plan to restore the Hamilton site to a tidal salt marsh in the same manner that was used at Sonoma Baylands.
Due to the desire of local residents and resource agencies, the SCC and the Corps chose to design twenty percent of the site as upland area and seasonal wetland and the remaining eighty- percent of the site as tidal salt marsh. The seasonal wetland and upland area will provide a transition and a buffer between the tidal salt marsh and the existing upland areas and urban areas that surround the Hamilton site (U.S. Army Corps of Engineers, 1998).

Initial Design: The basic design of the Hamilton site is very similar to that of Sonoma Baylands (see Figure 3). Initially, a levee will be constructed around the site to protect abutting property owners from flooding. This levee, however, will be over 200 feet wide and will slope gently down from an upland type habitat to seasonal wetlands and then to the tidal salt marsh. The seasonal wetland will also cover the entire northern and southwestern portions of the site, where local stormwater runoff will be directed. This area will be completely filled with dredged material, to within a few feet of its final elevation. Because the dredged material is mostly silty clay, with some silty sand, and is unsuitable for the final vegetative cover of the seasonal wetland area, the final cover will be composed of loamy topsoil from adjacent farmland.

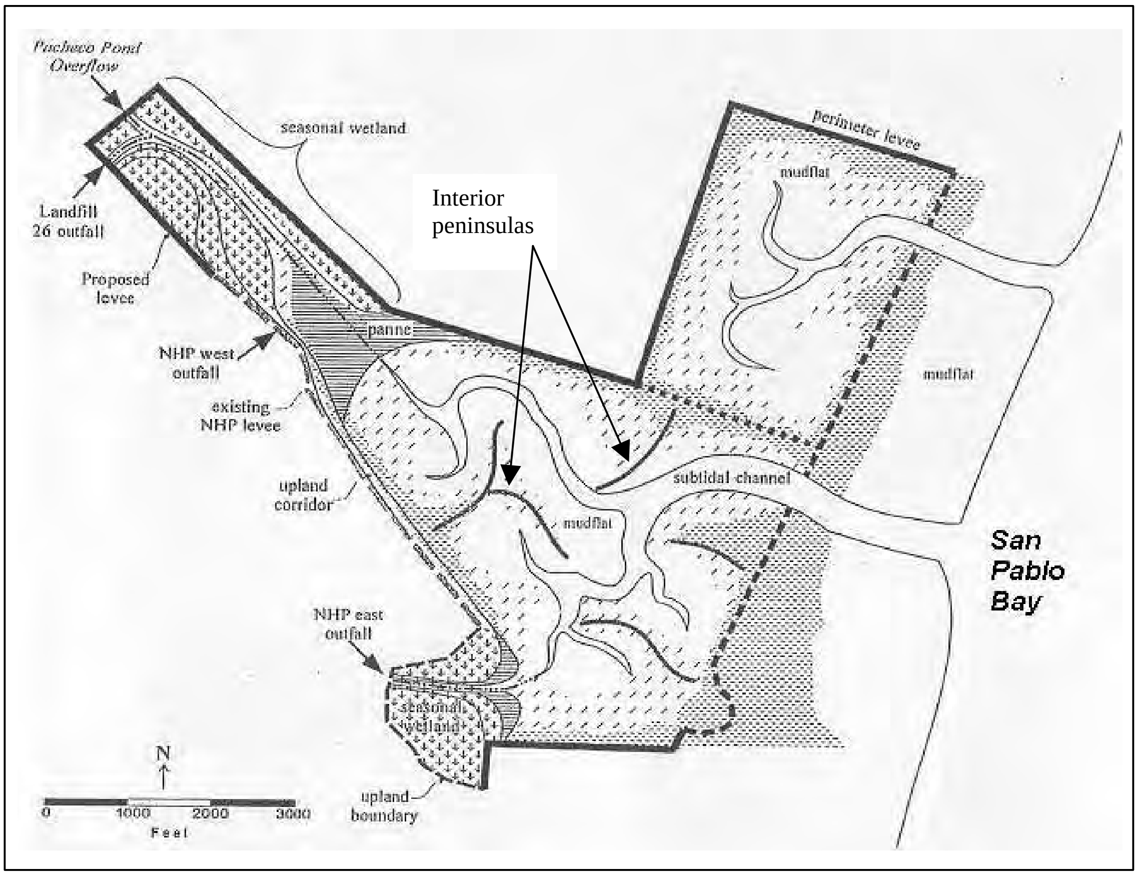

Figure 3 – Hamilton wetland restoration project conceptual plan

The tidal salt marsh area will be filled with dredged material to within 1.5 feet of its desired elevation, MHHW. Similar to Sonoma Baylands, this area will also include a series of interior peninsulas that will limit wave action in the restoration site, thereby reducing the likelihood that recently deposited sediments will be resuspended. In addition to limiting wave action, the interior peninsulas will also direct the flow of water and assist with the formation of the interior subtidal channels.

The Hamilton site has a much shorter outboard marsh area than is present at Sonoma Baylands. This marsh area does not include any significant natural tidal channels, so the Hamilton site's connection to San Pablo Bay will have to be fully excavated. The Hamilton design team sized the channel and its inlet based on a comparative analysis of eight local tidal wetland sites around the San Francisco Bay area. The analysis compared drainage area and tidal prism to channel width and depth. The breach and inlet channel for the 790-acre tidal wetland portion of the Hamilton site were thus designed to have a final equilibrium width of 280 feet and a final equilibrium depth of six feet below mean lower low water (MLLW). The initial design for the inlet channel at Hamilton involved excavating it to a size smaller than its anticipated equilibrium dimensions and, as at Sonoma Baylands, allowing it to scour and widen naturally.

The offshore mudflat area in front of the Hamilton site is similar to that near Sonoma Baylands. In the case of the Hamilton site, however, the much wider and deeper levee breach at the site inlet will require a correspondingly larger channel through the offshore mudflat. In the initial design for the site the design team assumed that the ebb and flow of water through the Hamilton site inlet would naturally scour a channel across the mudflat, as has been occurring at Sonoma Baylands.

Design Modifications: Currently, we have not yet finished evaluating the data from the past four years of monitoring at Sonoma Baylands. We have, however, come to a few preliminary conclusions in terms of design modifications:

- We are now planning to excavate the inlet channel and breach at Hamilton to close to their equilibrium dimensions. We do not consider slowing the natural sedimentation at the site in order to allow the outer channel and inlet to widen and deepen naturally to be consistent with the short and long-term restoration goals of the project.

- We will be reevaluating the placement of the interior peninsulas in order to ensure that they sufficiently reduce the development of wind waves and the potential resuspension of recently deposited sediments.

- We are planning to collect samples from the offshore mudflat and to perform laboratory testing on those samples. The goal of this testing will be to quantify how much mudflat scour could result from the ebb and flow of water through the Hamilton site inlet.

Additionally, we have recently completed a detailed analysis of the tidal datums at the Hamilton site, based on the most recent tidal epoch (1980-1998). This new datum information shows MHHW, mean tide level (MTL) and MLLW to all be higher at the site than we had previously calculated by interpolating between datums (1960-1978 epoch) at nearby tide stations. After reexamining our initial evaluation of sea-level rise potential in the area, and combining this with the new tidal datums, we will be able to finalize the heights of the perimeter levees and interior peninsulas at the site.

CONCLUSIONS

Sonoma Baylands was developed as a demonstration project, to try a new method for restoring tidal salt marsh in diked subsided areas. This method consists of partially filling the site with dredged materials, constructing interior peninsulas to limit wave induced resuspension of recently deposited sediments, and allowing natural sedimentation to create the final marsh surface and channel network. The initial design of the Hamilton project followed the same procedure that was used at Sonoma Baylands, with the understanding that the design would be modified to incorporate lessons learned from construction and monitoring of the Sonoma Baylands site. We are now at the point where we have enough monitoring data from Sonoma Baylands to begin quantifying which aspects of the design have performed as expected and which have not. Final design of the Hamilton project should be completed by late summer, 2001.

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NEW RESTORATION APPROACH FOR STEEP GRADIENT STREAMS: SPOKANE RIVER CASE STUDY

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Abstract

The Spokane River is a steep-gradient river that flows through the City of Spokane in eastern Washington. Large-sized sediments are eroded and transported down the river, with significant quantities deposited upstream of Avista Corporation's Monroe Street Hydroelectric Dam intake in the downtown area of Spokane. The impacts are river degradation, erosive damage to equipment, and disturbance of normal intake operations. Consequently, work crews must remove the sediment, which has typically been accomplished with disruptive and costly dredging operations. The major thrust of this study is the proposition of a new strategic plan for river restoration not based solely on qualitative assessment, but developed using quantitative analysis. This research utilizes two-dimensional hydrodynamic modeling, FESWMS, to map the flow patterns in the river. The results are used to identify and isolate areas of accelerated sediment erosion, and to develop river restoration methods to control upstream erosion.

INTRODUCTION

Project Site Description: The Spokane River is a steep-gradient river that drains an area of 17,200 square kilometers (km) in northern Idaho and eastern Washington (EBASCO, 1984). The river originates at the outlet of Coeur d'Alene Lake, Idaho, approximately 63 km upstream from the City of Spokane, Washington. Outflows from the lake are partly regulated by the Post Falls Dam located near the mouth of the lake. The Spokane River proper flows from the north end of the lake through the City of Spokane, and continues northwesterly for a distance of about 161 km to enter Franklin D. Roosevelt Lake on the Columbia River.

Average annual rainfall is approximately 43 centimeters (cm) in Spokane, and the average annual snowfall is 135 cm. Peak flows on the Spokane River and its tributaries may occur at any time in the period of October through June. Floods in the Spokane River basin usually occur in the spring as a result of the rapid snowmelt following rises in air temperature. Elevations in the project area range from about 518 to 579 meters (m) above sea level.

Based on surficial geological inspection in the study area, the bedrock is fractured basalt overlain in places by a layer of soil less than 3 to 5 m thick. The basalt observed is crystalline, fractured basalt. It is typically fresh, dense, gray to black, and fine-grained. Small columnar jointing has been observed in areas less than 3 m high by 6 m wide in slopes along the Spokane River (EBASCO, 1984). The irregular jointing produces a "hackly" or jagged texture in the basalt.

From Coeur d'Alene Lake to Spokane, the river flows through a valley of glacial gravel and silt. This is underlain by large expanses of basalt formation which outcrop at various locations in Spokane. These outcrops create the Spokane Falls (Upper Falls and Lower Falls) in downtown Spokane. The Upper Falls zone consists of two channels flowing around Canada Island in the middle of the river. Even outside of the falls zones, the river gradient is relatively steep.

The study site is the approximately 526-m long section of the river and riverbanks in a highly developed section of downtown Spokane, from just downstream of the Washington Street Bridge down to the Monroe Street Hydroelectric Development (HED). The study site (shown in Figure 1) includes Canada Island and the Upper Falls. The existing Monroe Street HED is located on the Spokane River at River Kilometer 119 (River Mile 74), upstream from the Columbia River. The Monroe Street HED is between the Upper Falls and Lower Falls in downtown. It was the first plant built by the utility company Avista Corporation (Avista), and has been in operation since 1890. There are six other hydroelectric plants on the Spokane River, five of which are owned and operated by Avista. The Upper Falls HED is located approximately 823 m upstream of the Monroe Street HED and is upstream of the study area; however, the study area does include the penstock outfall from the Upper Falls HED, which discharges into the river at the downstream end of the Upper Falls zone.

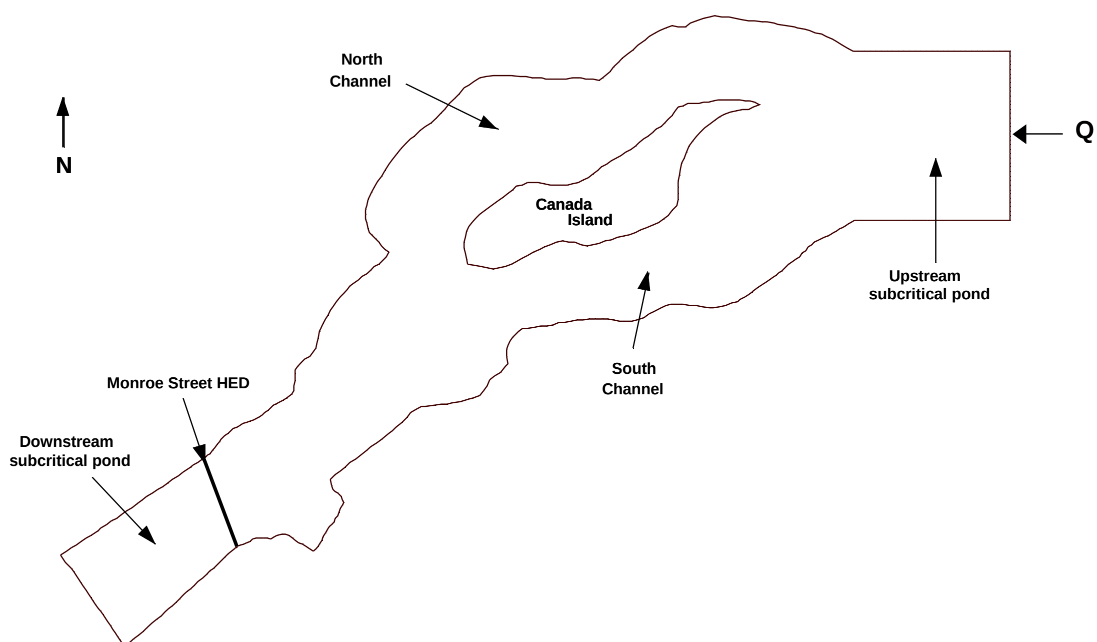

Figure 1. Spokane River Study Site

<u>Objectives</u>: Large cobble and boulder-sized sediments are eroded and subsequently transported down the Spokane River within the study area. The impacts are river bed degradation, erosive damage to equipment, and disturbance of normal intake operations. These sediments are deposited in the forebay area of the Monroe Street HED intake. As sediment accumulates behind the intake screens, it interferes with the normal flow of water through the intake structure. Some of the gravel particles with size less than 5 cm in diameter pass through the screens and cause impact and erosive damage to turbine blades. Consequently, work crews must remove the sediment, which has typically been accomplished with disruptive and costly dredging operations. Current Department of Ecology instream flow requirements for the Spokane River preclude any further use of dredging as a means of sediment removal.

The goals of this study are (1) to find alternative methods for preventing sediment accumulation at the intake structure, and (2) to suggest remediation measures to control the sediment erosion process well upstream of the intake structure. For these purposes, a two-dimensional hydrodynamic finite element model is employed to depict the flow velocity field within the study area, and to obtain better insight about the flow patterns and possible mechanisms triggering sediment erosion. The research utilizes the Finite Element Surface Water Modeling System (FESWMS) computer program, developed for the Federal Highway Administration, to map the transcritical flow patterns within the river channel. FESWMS offers considerable advantages over other existing numerical models since it provides information about the two dimensional velocity vector and shear stress distribution for supercritical flows (Froehlich, 1994). The use of this model allows the detection of those areas within the forebay that exhibit high local flow acceleration, which in turn results in local scour of the river bed and banks. A sediment transport model applicable to high-gradient streams will be used to predict the amount of sediment transported downstream.

The major thrust of this study is the proposition of a new strategic plan for river restoration not based solely on qualitative assessment, but developed using quantitative analysis. The results are used to identify and isolate areas of accelerated sediment erosion. The quantitative results of this analysis will be used to evaluate optimal strategies for bedload control in the vicinity of the Monroe Street HED. The final product of this study will be the development of a river restoration technique that yields successful sediment entrapment for a variety of flow conditions. Depending on the flow conditions, channel gradient, river geomorphology, and substrate material, several alternative structures will be developed to control upstream erosion and to minimize sediment deposition and passage though the intake structure.

METHODOLOGY

Historical Analysis: The first step in applying restoration techniques to a serious problem in a river system is to perform a qualitative historical analysis. An historical analysis has the potential to highlight what later may seem like obvious causes of or obvious solutions to the problem. Even if the obvious solution is not reasonably attainable, it is imperative to understand the underlying mechanisms of what is causing the problem in the first place in order to develop an appropriate solution. It is important to perform this analysis even in the case where there is little documentation of what used to be the historical condition of the river system, if only to document the fact that this information was sought and not found (Kondolf, 1985).

Available documentation concerning the Spokane River and the development of the sediment issue at Monroe HED was reviewed in order to evaluate the impacts that several hydraulic and non-hydraulic structures, built along the stream corridor, had on the historically natural state of the Spokane River.

FESWMS: The Spokane River Restoration model consists of two components, the hydrodynamic analysis and the sediment transport analysis. The focus of the present study is on the hydrodynamic component.

In this investigation a two-dimensional hydrodynamic model is used to depict the flow field (i.e., velocity). The two-dimensional velocity vector will be used to determine the shear stress distribution along the longitudinal and transverse directions. This information is important to accurately determine sediment transport rates and to provide solutions for stream restoration.

Specifically, FESWMS version 2 is used here. This model, developed for and sponsored by the Federal Highway Administration, is applicable to both subcritical and supercritical flow conditions (Froehlich, 1994). FESWMS is used to compute water surface elevations and flow velocities at nodes in a finite element mesh (FEM) representing a body of water such as a river, harbor, or estuary. Both steady state and transient solutions can be determined with FESWMS, so it is possible to model static flow conditions such as a river with constant flow, or dynamic flow conditions such as river flows with hydroelectric dam flow releases. The commercially available software SMS (Surface Water Modeling System) was used to pre- and post-process the model input and output data. This software enables the user to quickly construct a finite element grid, and allows for quick and easy evaluation of the model results.

To run FESWMS, the FEM is first constructed for the reach of interest along the river. Data defining the boundary conditions and material properties are then required. FESWMS applies the finite element method to solve the system of equations that describes two-dimensional depth-averaged surface water flow in a horizontal plane (defined by the longitudinal and transverse directions). The method of weighted residuals using Galerkin weighting is applied to the governing flow equations to form the algebraic form of the finite element equations. Newton's iterative method is then used to solve the non-linear terms of the momentum equations (Froehlich, 1994). The flow field is determined using the river channel bathymetry, the river's boundary conditions, and the river's resistance to flow. The upstream boundary condition used (known as the natural condition) was a specified upstream flow. The downstream boundary condition used (known as the essential boundary condition) was the corresponding downstream water surface elevation for the specified flow. The river's resistance to flow is quantified by the river channel's value for Manning's roughness coefficient n.

Spokane River Model Data: The FESWMS model was applied to a portion of the Spokane River approximately 526 m in length, as previously described. The data used to develop the detailed bathymetric data for the model base map was mainly derived from topographic mapping based on aerial photography of the project area. Prominent, defining features of the study area were used to determine the boundaries of the FEM.

Flow data for the Spokane River were obtained from the United States Geological Survey (USGS) stream flow gaging station No. 12422500 (Spokane River at Spokane, Washington), located approximately 1.6 km downstream of the Monroe Street HED. A Log-Pearson Type III hydrologic analysis of the annual peak runoff for water years 1891-1998 was performed to determine design flows for the upstream boundary condition. The normal depth corresponding to each flow was calculated using a uniform flow equation to determine the downstream boundary condition.

A number of methods exist for estimating the Manning's roughness coefficient. Typically a field investigation is conducted, and the conditions encountered in the field are compared with published photographs of reaches for which *n* has been determined. In addition, tabular data describing reach characteristics and associated *n* values can be used to refine initial estimates of the roughness coefficient. For this project, field investigations were conducted during the 1999 Spring runoff period (late April and May) to visually inspect hydrologic and hydraulic conditions. Photographs taken of the channel reach in previous years during periods of low flow were also reviewed to determine river bottom and riverbank hydraulic resistance. This information was considered in conjunction with photographic comparisons and tabulated data to determine the representative *n* values for modeling the project study area.

RESULTS

Historical Setting: The Spokane River has historically carried a significant sediment load, predominantly bedload. Some of its current sediment load is thus part of the natural state of the river. The river was also historically a very braided channel. Many branches of the Spokane River are no longer in existence due to the development in the city, which has occurred along the banks and restricted the lateral migration of the river. The gradual constriction of the river to its current state suggests that the encroachment of development has had an impact on the increased erosion and subsequent sediment transport.

Comparing topographical maps from approximately 1915 with aerial photography and contour maps from 1983, it can be seen that the longitudinal slope of the river in the study area has remained approximately the same. However, the actual ground elevations have dropped on average 3 m, and up to 4.6 m in some locations. The areas of elevation change are mainly in the river bed around Canada Island and just downstream of the spillway (found at the Monroe street dam). This suggests that the area around Canada Island may be a source of some of the sediments being eroded and transported downstream.

Spokane River Flow and Roughness Characteristics: The results of the annual peak flow analysis and normal depth calculations are shown in Table 1.

Table 1. Spokane River Boundary Conditions

Exceedance Probability %	Return Period (years)	Total Peak Flow (cubic m/sec)	Downstream Water Depth (m)	Water Surface Elevation (m)
99	1.01	222	1.5	545.4
95	1.05	330	1.9	545.8
90	1.11	400	2.1	546.0
50	2	698	3.0	546.9
10	10	1,033	3.7	547.7
2	50	1,220	4.1	548.1
1	100	1,280	4.3	548.2

Table 1. Spokane River Boundary Conditions
Notes Peak flows and exceedance probabilities determined from hydrological analysis Downstream location in artificial pond area at model boundary Water depths determined from uniform flow calculation Water Elevation = [ground elevation] + [water depth] Datum in pond = 544 m Avista Corp. datum

These design flows were chosen to represent the typical flood events that result in sediment aggradation in the Monroe forebay, and several higher return period flood event flows which have occurred historically. Table 2 shows the roughness coefficients used in the model for the different materials defined.

Table 2. Material Parameters in River Channel

Model ID	Material Name	Additional Description	Channel "n"	Wall "n"
01	Main Channel	natural substrate	0.046	0.051
02	Steep Bank	natural substrate	0.104	0.104
03	Canada Island - steep bank	natural substrate	0.108	0.108
04	Canada Island	partial development	0.064	0.071
05	Floodplain - developed		0.034	0.037
06	Floodplain - mixed	part natural, part developed	0.048	0.053
07	Steep Bank - mixed	part natural, part developed	0.074	0.082

Model Results: The finite element mesh developed for the Spokane River Model is shown in Figure 2. Several elements were added to the upstream end of the mesh in order to “smooth” the elements at the inflow and to encourage model convergence. Additionally, a large artificial pond was added to the area downstream of the Monroe HED where in reality the Lower Falls are located. These subcritical regions were added since FEWMS tends to prefer this type of flow at the inflow and outflow boundaries. Since downstream of the Monroe HED is outside of the area of interest for the erosion issue, the artificial pond does not interfere with results and facilitates model convergence.

Preliminary results for the 2-year, 10-year, 25-year, and 100-year storm event flows have been determined. Larger velocities have been observed in the southern channel around Canada Island (see Figure 3) and along several riverbank areas for all of the flows modeled. These areas will be the focus of further sediment transport modeling and restoration measures.

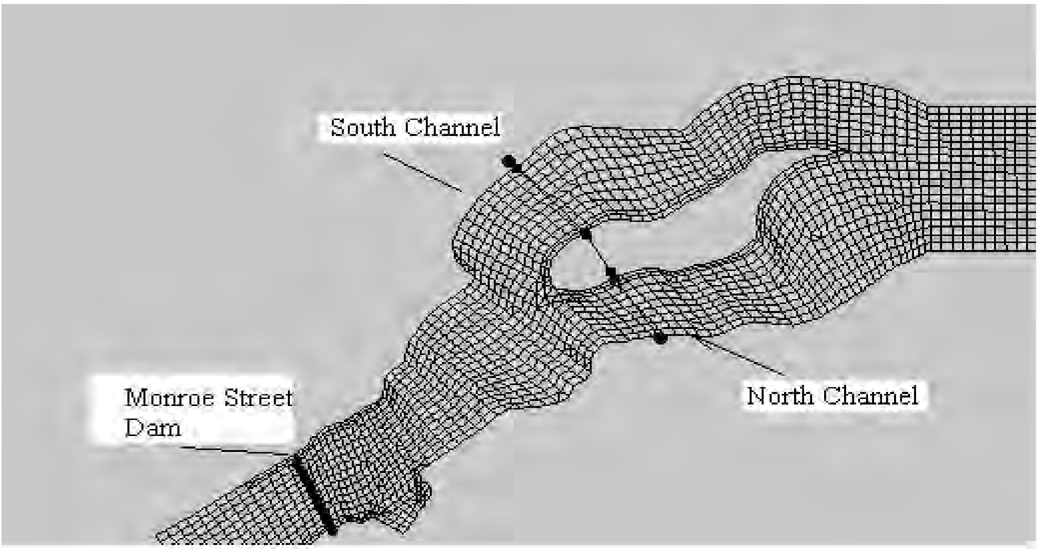

Figure 2. Spokane River Finite Element Mesh

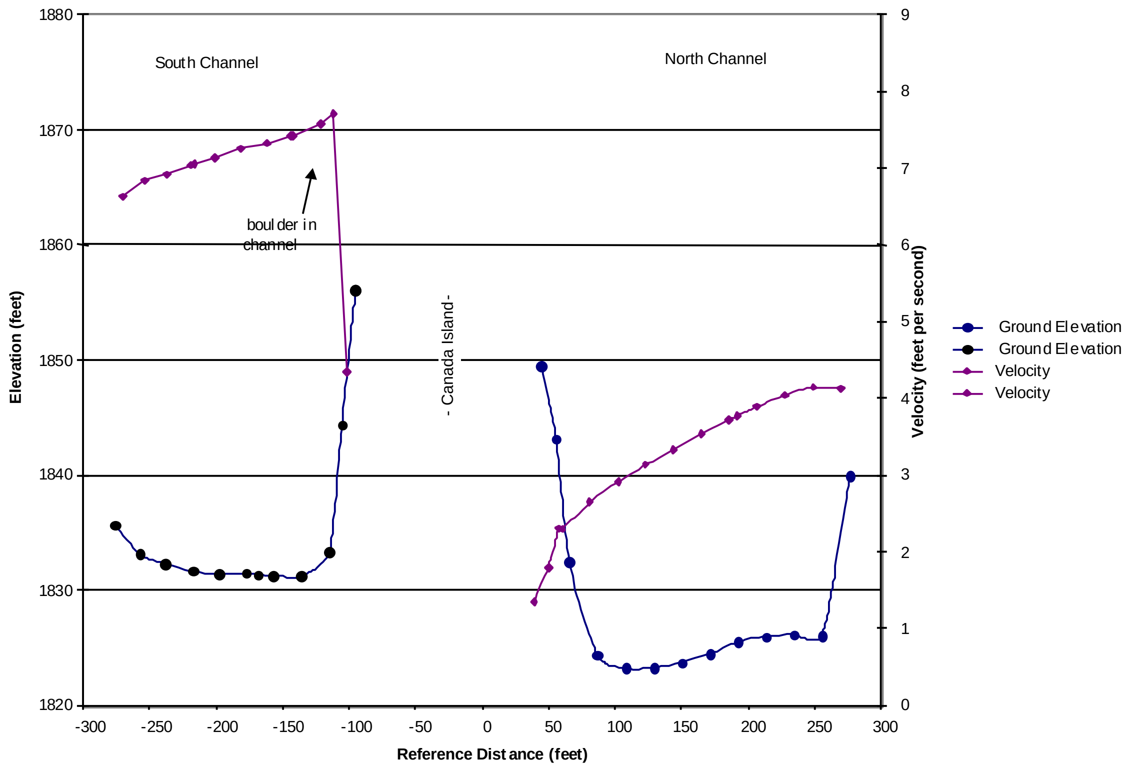

Figure 3. Spokane River Cross Section: 2-Year Flow Velocities

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DEVELOPING A "REFERENCE" SEDIMENT TRANSPORT RELATIONSHIP

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INTRODUCTION

In 1980, the Environmental Protection Agency (EPA) published "An Approach to Water Resources Evaluation of Non-Point Silvicultural Sources" (WRENSS). The document represented a then state-of-the-art approach for watershed analyses and prediction of the impact of non-point silvicultural activities on water quality. Land and water management practices continue to be one of the dominant contributors to water quality impairment through impacts on sediment loading to channel systems and transport process. This report addresses an attempt to develop a "Reference" sediment transport relationship, utilizing existing sediment transport data, that once developed could be used as the basis for documenting departure in impaired watersheds.

OBJECTIVES

There are two objectives, or specific hypotheses:

a. H_o: The reference condition (Natural Range of Variability) sediment transport relationship for stable systems (systems capable of carrying the sediment being delivered without change in dimension, pattern, or profile) can be defined.
b. H_o: In disturbed systems, departure of the sediment transport relationship from the reference condition can be documented.

METHODOLOGY

Attempts to develop a "reference relationship" for sediment transport have been an iterative process that initiated with the historical sediment transport data collected by D. Rosgen and several others and culminated by relying on fewer data sets from intensively studied watersheds elsewhere.

D. Rosgen provided sediment transport data (both bedload and suspended sediment) from approximately 160 watersheds located in Colorado, Wyoming, Alaska, Montana, and Idaho. Not all, but most, of the sites had data for both suspended sediment and bedload. For each of the watersheds the data consisted of a series of paired samples of sediment transport, either concentration of suspended sediment (mg/l) or rate of bedload movement (kg/s), through a cross-section and the discharge rate (cfs) at the time the sediment sample was collected. In addition, descriptive metrics that included channel type (Rosgen 1994), Pfankuch (1975) stability rating, and an estimate of bankfull discharge (the 1.5-year return interval discharge rate) were provided.

The expectation was that the historical, or Rosgen, data set could be useful in defining a reference sediment transport curve, stratified by stream type and stability rating, that could be used to document departure when compared with other systems. At the onset of this analysis, a basic assumption was made that the transport $\times$ discharge relationship needed to be presented in a dimensionless format. An assumption made because previous experience (EPA 1980) indicated virtually every stream has a unique sediment transport signature that reflects watershed size, stream type, discharge rate, sediment supply, etc. and transforming the data into a dimensionless form was necessary to diffuse much of this variability.

The process of dimensionless transformation is described in the following sequence for each watershed and sediment type (suspended, bedload). In their dimensional form, suspended sediment is a concentration and expressed in mg/l, bedload transport is a rate and is expressed in kg/s. Discharge, also a rate, is expressed in cfs.

1) Initially a linear model ($y = a + bx$) was fit to the discharge (x) and sediment (y) pairs for each watershed. If the model was not significant, meaning b was determined not to be different than 0, it was concluded that no slope exists in the relationship between sediment transport and discharge for that watershed based on the data available. Therefore, the mean value for transport rate or concentration estimates transport at all flow levels.

These watersheds were dropped from further analyses. If the slope proved to be significantly different than 0, the next step was to determine if a linear model or a power model best described the data by fitting a power function ($y = a + bx^c$) to the data. If c was significantly different than 1.00 it was concluded the data were nonlinear in nature. If c was not significantly different than 1.00 it was concluded that the linear model sufficiently described the data. As part of the fitting process the studentized deleted residuals from the models were used to test for outliers (Neter, Wasserman, and Kutner 1990). The p-value was set to 0.0001 so that only very extreme outliers were identified. These outliers were then graphically interpreted to only eliminate points that very obviously detracted from the model fit or form.

2) The fitted dimensional transport model for each sediment type and watershed were then used to predict the sediment transport that would occur at the predetermined estimate of "bankfull" discharge, or the 1.5-year event. Each value of sediment transport (y_i) for that watershed was subsequently made "dimensionless" by dividing it by the predicted value of sediment transport at bankfull discharge. The corresponding value of discharge (x_i) for each sediment and discharge pair was also made dimensionless by dividing by the estimate of bankfull discharge. The model fitting process explained above was then repeated for the transformed, dimensionless, data. In virtually every instance the model form remained the same.

   At times, when fitting the power function, there were difficulties in getting the b coefficient and the c exponent to converge. In these instances, the G-4 option in SAS (1989), which uses the Moore-Penrose inverse in the parameter estimation, was implemented to facilitate the process. Convergence problems can occur when fitting a power model if the coefficient b and exponent c are correlated. The G-4 option helps with convergence because a singular-value decomposition algorithm is used. It is important to note that for data with a strong nonlinear relationship, the model form and fit are exactly the same with or without using the G-4 option.

3) While finding the best dimensionless transport models for each watershed, the model residuals were examined for homogeneity of variance and normality. Watersheds containing sediment measurements distributed across all levels of discharge did not typically have a serious normality or heterogeneous variance problem. However, there were many sites, some with as few as 5 data points, for which these data were not distributed across all levels of flow. In these instances the range of data and amount of data collected did not allow us to adequately examine residuals, but the models fit appeared appropriate. Sites were then grouped by stream type and stability rating for further analysis. Tests were run to determine if the grouped, or combined, models within stream type and between stability rating classes (GOOD, FAIR, POOR) were similar by using an extra sum of squares analysis for nested models that determines if two or more models are significantly different than the pooled model (Bates and Watts 1988). The subsequent series of analyses compared bedload and suspended sediment transport within stability class, between stability classes, and within and between stream types.

4) The desired end product of the analysis was to be the definition of a reference expression for sediment transport, stratified by stream type, that would define the range of variability in sediment transport over a wide range in flow and from which human induced departure, of a test watershed, could be detected.

RESULTS

Historical Data Sets

Dave Rosgen, (Wildland Hydrology, Pagosa Springs, CO) provided suspended and bedload sediment transport data from approximately 160 watersheds located in Colorado, Alaska, Idaho, and Montana. The data, stored in both his and U.S. Forest Service archives, represented sampling done over the past 30 years. James Nankervis (Blue Mountain Consultants, Berthoud, CO) coordinated the electronic entry of the data, oversaw the verification or revision of either the channel type, as currently characterized by Rosgen, or the Phankuch (1975) stability rating, and determined initial model form. The final data set consisted of the sediment × discharge pairs, stream type, stability rating, and an estimate of bankfull discharge (approximately the 1.5-year maximum instantaneous flow).

Once model fitting for bedload (kg/s) and suspended sediment (mg/l) was completed as outlined, analysis was conducted to determine if the sediment transport relationships characteristic of the watershed were related to either the classification of stream type or the assignment of the Phankuch (1975) stability rating. Most of the more common, but not all stream types, were represented in the watershed sampling and fewer GOOD streams were sampled than FAIR or POOR streams (Figure 1). At first, some of the results of the analysis appeared somewhat unexpected, but understandable. As expected, bedload and suspended load respond to discharge differently and at different rates with suspended sediment being the larger component of the total sediment transport. For bedload transport, however, we failed to show significant differences between dimensionless transport within and between stream types for both GOOD and FAIR stability ratings. Streams with POOR stability ratings exhibit differences within and therefore, by default, between stream types. This implies POOR streams exhibit significantly different dimensionless sediment transport characteristics (departure) from each other as well as from either FAIR or GOOD streams. This pattern was not as clear, or well demonstrated, for suspended sediment transport as there appears to be more inherent variability both within and between stream types and stability rankings. However, the tendency to fail to show departure among streams and between stream types is still quite strong as 86% of all comparisons failed to document departure. Although differences in dimensionless suspended sediment transport appear to exist within stream types, we cannot conclude they are differences attributable to stream type.

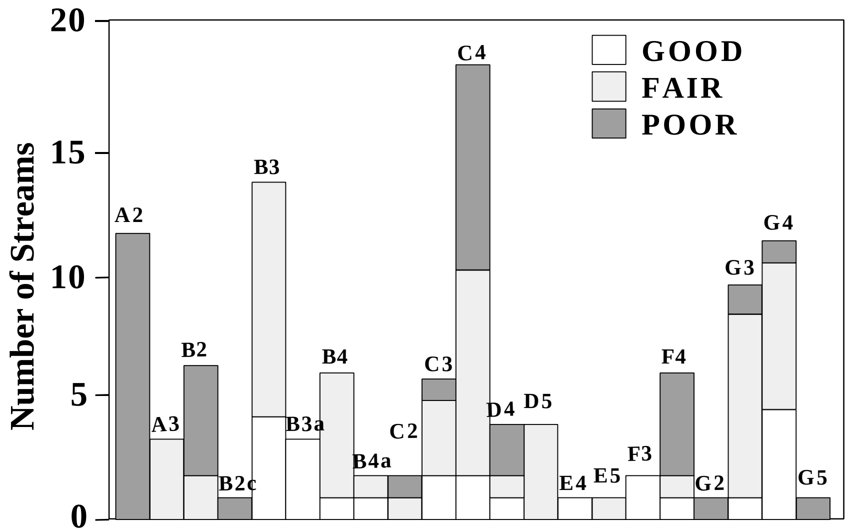

Figure 1. Stream type and channel stability rank.

It must be remembered that differences in the dimensional, or absolute, sediment transport/stream discharge relationship do exist, both within and between stream types and between stability ratings for both suspended and bedload transport. Every stream has its own sediment signature. It would appear that the transformation of data into a dimensionless format compensates for those differences. The similarity of transport response, apparent in the dimensionless format, reflects the continuity of mass and equilibrium that must exist within the system. Intuitively, one would expect continuity across or through stream types, if the system is in equilibrium That continuity would be disrupted by poor stability caused by human intervention or a catastrophic event, as evidenced by the departure of POOR streams. This instability, or change in sediment supply, could cause either a departure from the dimensionless sediment transport relationship or a change (evolution) in stream type.

Transforming the data into a dimensionless format by dividing all sample pairs by bankfull discharge and sediment transport at bankfull discharge, forces all models through (1,1) on the (X,Y) axis. Although not specifically constrained in the fitting process, all models also tend toward (0,0). Only a few of the watersheds sampled in the historic data set were sampled at flow levels in excess of bankfull, therefore the preceding analysis was not very robust as the models tend to become constrained at both ends of the data range. Thus, there was little opportunity for differences to be documented when the observed range in data did not exceed bankfull, especially if the “best fit model” was linear. To evaluate the significance of this concern, and determine whether definition of a reference relationship was a truly viable concept, data from experimental watersheds at the Fraser Experimental Forest and from the East Fork of the Encampment River in Wyoming were added to the database.

Fraser Experimental Forest (FEF) Data Sets

Suspended sediment and bedload transport have been intensively sampled on numerous FEF experimental watersheds since 1993. To date, 100 or more data pairs for both suspended and bedload transport, are available for each watershed with the distribution of samples virtually encompassing the entire range of flows observed to have occurred over the past 60 years (Troendle and Olsen 1994, Ryan and Troendle 1996, Troendle et al. 1996, Wilcox et al. 1996). Troendle et al. (1996) noted that for only 12 days between 1943 and 1995 did mean daily stream flow from East St. Louis Creek exceed the maximum flow value for which sediment transport had been monitored. Maximum instantaneous flows on the same watershed have exceeded the highest flow sampled less than 25 times. The experimental watersheds vary in size: Deadhorse Creek (DHOMA) is 640 acres, Fool Creek (LFCRK) is 714 acres, Lexen Creek (LEXEN) is 307 acres, East St. Louis Creek (ELOUI) is 1984 acres, and East Fork Encampment (UEFXS) is 2200 acres. These watersheds also have different geology (granites, sedimentary) and stream order (1^{st} – 2^{nd}). Bedload transport has also been monitored at 6 additional locations along St. Louis Creek, the 4^{th} order stream draining the 23,000-acre Fraser Experimental Forest (Ryan and Troendle 1996). Contributing areas for the 6 sites range from 8,300 to almost 14,000 acres. Bedload transport was monitored from 1992 to 1997 at these cross sections, 3 located above and 3 located below points of water diversion. The range of flows sampled is as intensive as the other FEF sampling. Suspended sediment was not available at the Main St. Louis Creek sites. In general, streams associated with each of the 11 cross sections have a GOOD stability rating with drainage areas from 600 to almost 14,000 acres and Rosgen (1994) Level 1 stream types of A, B, or C.

As with the historical data, sediment transport was transformed and dimensionless models estimated in the manner described earlier. Subsequent analysis failed to demonstrate differences in dimensionless transport $\times$ dimensionless discharge models for either suspended sediment (Figure 2) or bedload transport (Figure 3), when compared with the pooled model for all watersheds. A single dimensionless model, one each for suspended sediment (Figure 2) and bedload transport (Figure 3), best describes sediment transport. The datasets for suspended and bedload transport contain 293 and 1124 data pairs respectively. These data extended over a range in discharge that exceeds 2 times bankfull discharge and approaches the 1-in-25 year event. Due to lack of normality and homogeneous variance, bootstrapping methods were used to generate both the regression models (SPSS 1993) and the 95% individual prediction intervals (Stine 1985). The prediction intervals were generated using 5000 bootstrap iterations and then smoothed for presentation purposes. These intervals can be used to assess departure of individual transport $\times$ discharge pairs, sampled on other watersheds, from the reference models.

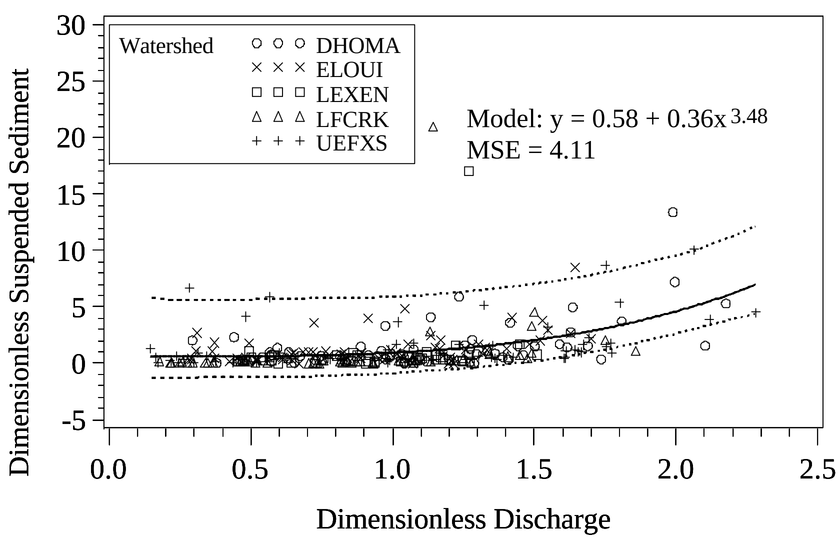

Figure 2. Reference suspended sediment transport model with 95% individual bootstrap prediction intervals.

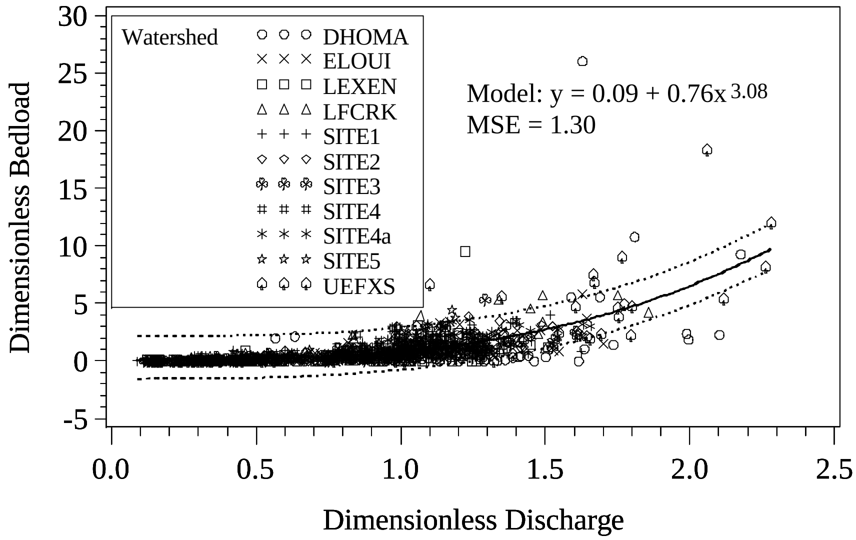

Figure 3. Reference bedload transport model with 95% individual bootstrap prediction intervals.

Testing the Reference Curves

The analysis to this point leads us to acceptance of the hypothesis that a single dimensionless sediment transport model, for either suspended sediment or bedload represents reference dimensionless transport for undisturbed systems. Two additional analyses were conducted as a further test of this hypothesis.

Sediment transport data for the GOOD and FAIR B3 and C3 streams in the historical data set were selected to compare with the reference curves for both suspended and bedload transport developed from experimental data. Dimensionless transport models for the historical data sets for suspended sediment (p=0.95) and bedload transport (p=0.99) were found to be the same as the respective reference relationships (Figures 4 and 5). As noted earlier, one of the deficiencies of the historical data is the limited range in discharge over which the historical data was collected and that limitation is apparent in Figures 4 and 5 as the sampling is skewed to lower flow levels of discharge relative to the reference curve.

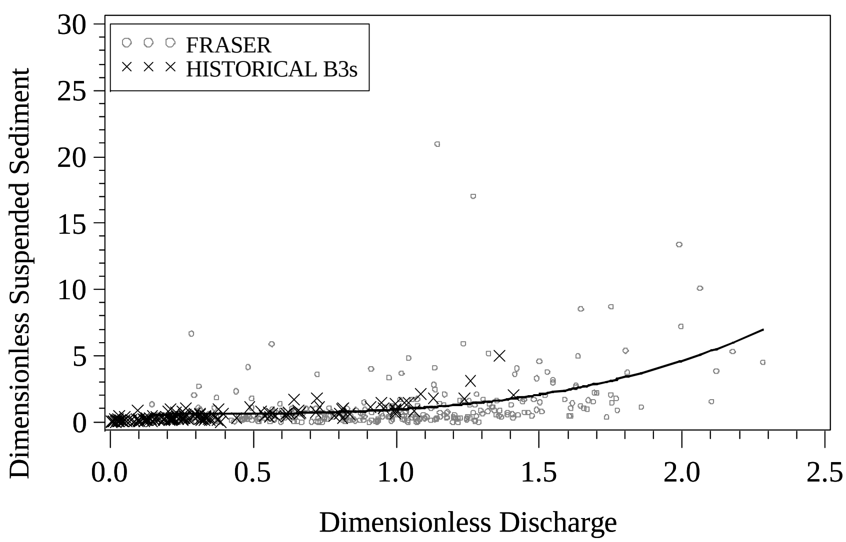

Figure 4. Suspended sediment for all historical B-3 streams plotted over pooled model for reference streams.

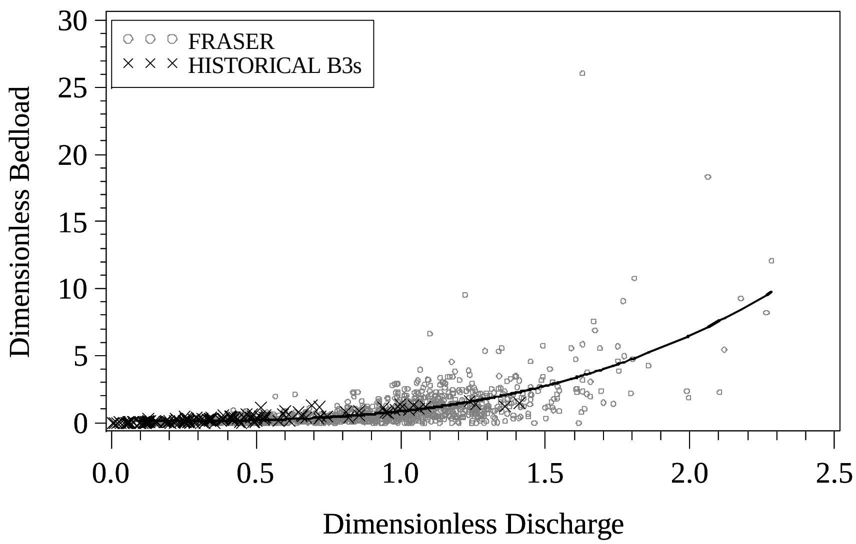

Figure 5. Bedload transport for all historical B-3 streams plotted over pooled model for reference streams.

Although not presented, the outcome was similar for all historical C3 streams as well (p=0.65 for suspended sediment test and p=0.99 for bedload test). This implies little difference exists between the reference curve and the historical data for GOOD and FAIR B3 and C3 streams.

The final assessment of the reference curve evaluated dimensionless suspended and bedload transport data from Coon Creek. Coon Creek is a 4,000-acre partially harvested watershed on the Medicine Bow National Forest, adjacent to Upper East Fork of the Encampment River, one of the reference watersheds. In 1989, roads were constructed in Coon Creek to allow a total of 24% of the watershed to be harvested in small clearcuts in 1990, 1991, and 1992. Suspended sediment and bedload transport from Coon Creek were not monitored until after timber harvest. Both suspended sediment and bedload transport data were collected on Coon Creek in 1993 and 1995 (Wilcox et al. 1995)

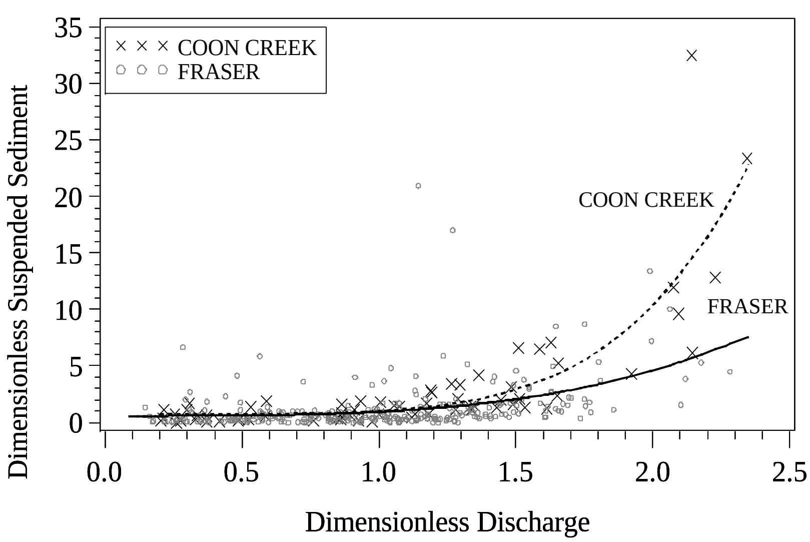

Figure 6. Comparison of the dimensionless suspended sediment model for Coon Creek and the pooled model for reference conditions.

Dimensionless suspended and bedload transport models were estimated for Coon Creek and compared with the pooled model for the respective reference condition. The dimensionless supended sediment model for Coon Creek significantly differs ($p<0.0001$) from those comprising the reference condition (Figure 6). In contrast, the dimensionless bedload transport relationship for Coon Creek (Figure 7) does not significantly differ from the reference curve. However, it should be noted that the p-value associated with differences between the 12 streams comprising the reference bedload transport curve is 0.94. Adding the 13th stream, Coon Creek, causes the p-value to drop to 0.17. The addition of the Coon Creek model increased departure, although not to the level of significance at $p = 0.05$.

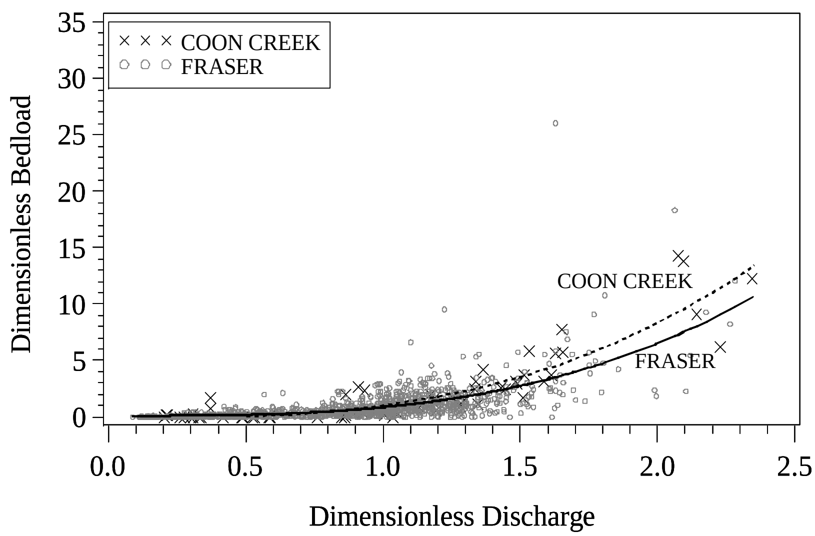

Figure 7. Comparison of the dimensionless bedload transport model for Coon Creek and the pooled model for reference conditions.

SUMMARY

The "Reference Sediment Transport" functions for suspended sediment and bedload transport appear to function well and indicate departure can be demonstrated. An interesting outcome of the analysis was the lack of ability to show differences in dimensionless sediment transport attributable to stream type. At first, this seemed inconsistent with expectations. Upon review, the outcome seems intuitively appropriate. If bankfull discharge is in fact the flow that maintains channel geometry and if streams in equilibrium are those that carry the sediment being delivered to them, then there should be continuity, even similarity, in the dimensionless sediment transport functions. Thus, sediment passing through one reach is passed on to the next, and so on, in a continuum. The same is true with flow. Unless a particular reach is unstable, the material is passed on with minimal deposition or scour. Where there is instability, aggradation or degradation occurs and the channel is in disequilibrium and the Rosgen Level 1 channel type changes. As channel morphology changes and stream type evolves, a new equilibrium is reached, and continuity appears to resume. As a working hypothesis, the reference curves should be a useful prototype in detecting departure while instability is present. If data pairs are available for a study stream and dimensionless transport $\times$ discharge can be calculated, the dimensionless data pairs can be plotted individually on the reference curve. The determination can then be made as to whether the individual point falls within or outside of the 95% prediction interval. If departure occurs, this may imply the study watershed is impaired and the sediment transport model for the study watershed warrants further investigation. Once a channel type change occurs (e.g. a C3 degrading to a G4) the sediment transport for the study watershed may not indicate departure. In the latter case, departure is better defined by knowing the channel type has been forced to evolve. Analysis of the historical data, as well as the Coon Creek test case, documents that departure in sediment transport from impaired streams from the reference condition can be detected. Departure implies channel instability, often do to either a change in sediment supply or flow regime. Instability may foster an evolution in stream type.

The dimensionless sediment transport curves can be transformed into a dimensional form, specific to any watershed, by reversing the transformation process. At a minimum, an estimate of the bankfull discharge and at least 1 sediment transport × discharge pair must be available for the watershed of concern, and ideally the discharge pair taken should be at or near bankfull discharge. In any event, the dimensionless discharge for the sediment sample collected is calculated by dividing the sample discharge rate by the bankfull discharge rate. The ratio, or dimensionless discharge estimate (x), can then be entered on the appropriate dimensionless sediment transport curve to determine the dimensionless sediment transport associated with the sample discharge. An alternative is to use the prediction model also presented on Figure 2 or 3 to estimate the dimensionless sediment transport directly. Once the dimensionless sediment transport ratio (sediment transport divided by the sediment transport at bankfull) is determined, the observed estimate of sediment transport (dimensional) is divided by the sediment transport ratio to estimate the dimensional sediment transport that should be expected at bankfull discharge. The scale on the (X,Y) axis on the dimensionless reference curve can be transformed into dimensional values by multiplying by the estimate of bankfull discharge (X axis) or by the estimate of sediment transport at bankfull discharge (Y axis). The dimensional curve represents an estimate of the sediment transport curve for the watershed of interest. Any additional sediment × discharge pairs can then be plotted over the predicted curve to validate appropriateness of fit**.**

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RIVER RESTORATION ON THE MIDDLE RIO GRANDE: OPPORTUNITIES AND CHALLENGES

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Abstract: The historic Middle Rio Grande has been aggrading (i.e. rising river bed due to sediment accumulation), with a slightly sinuous plan form, shifting sand substrate, and low banks. Frequent channel avulsions caused the channel to migrate across the valley floor. In the late 19th and early 20th centuries an era of channel control began. The goal of this work was to narrow the river channel, improve the sediment transport carrying capacity of the river to reverse the aggradation trend, and reduce flooding. These works have effectively accomplished these objectives from an engineering perspective. The channel has narrowed and degraded (i.e. lowering river bed due to sediment removal). This has resulted in a more stable channel, that does not shift across the existing floodplain. However, these changes have also led to the decline in populations of native fish and terrestrial species resulting in several species being listed as endangered species.

In this paper, the history of the Middle Rio Grande is briefly reviewed, historic and current geomorphology presented, together with several restoration goals and activities. The goal of these restoration projects generally is to create wider shallower flow conditions, re-connect the main channel with a flood plain, establish riparian areas comprised of native species, and provide for some channel migration within geographic limits.

INTRODUCTION

The alluvial Middle Rio Grande has long been recognized for it=s striking characteristics. The surrounding desert lands, large snowpack and summer thunderstorms produce wide ranges of water and sediment flows. As the river flowed from the mountainous region to the flatter Middle Rio Grande Valley sediment was deposited resulting in river bed aggradation (i.e. river bed raising by sediment accumulation). The ancestral Middle Rio Grande was a relatively wide, aggrading channel with a shifting sand bed and shallow banks. The planform was braided, relatively straight, or slightly sinuous (Crawford et. al. 1993). It is estimated that the Middle Rio Grande Valley in New Mexico (Figure 1) has been aggrading for the last 11,000 to 22,000 years (Leopold et. al. 1964, and Hawley, et. al. 1976). Thus, not being in a state of dynamic equilibrium. The maximum degradation is believed to have occurred about 22,000 years ago, when the Rio Grande was about 60-130 ft. below the current valley floor. Since then, the Middle Rio Grande has been slowly aggrading because tributary inflows contribute more sediment than the river can remove. (Crawford et. al. 1993). Attempts have been made for many hundreds of years to divert the Rio Grande by brush and rock dams.

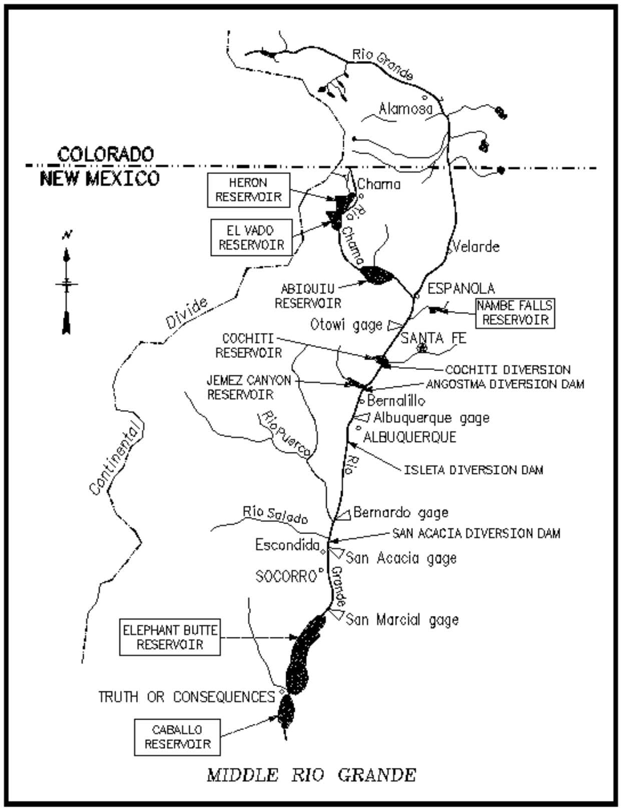

Figure 1. Middle Rio Grande

Completion of the irrigation works by the Middle Rio Grande Conservancy District (MRGCD) in 1936 marked the start of the period of active river control. After the formation of the MRGCD, non-engineered spoil levees were constructed parallel to the river. These levees were protected by sediment fences and vegetation but were often washed out during high spring runoff flows. Due to a variety of reasons, including unusually high flood flows in the early 1940's and the aggrading stream bed throughout the Middle Rio Grande Valley, the MRGCD sought assistance from the federal government. This assistance was provided by the United States Bureau of Reclamation (Reclamation) and the Army Corps of Engineers (Corps) through the Middle Rio Grande Project authorized by the Flood Control Act of 1948 and supplemented by the Flood Control Acts of 1950 and 1960.

The two agencies set out to accomplish the assignment through a variety of means including the construction of large dams and channel rectification to control floods and sedimentation. The channel has narrowed and degraded (i.e. lowering river bed due to sediment removal). This has resulted in a more stable channel that does not shift across the existing floodplain. However, these changes have also contributed to the declining populations of native fish and terrestrial species resulting in the Rio Grande silvery minnow and southwestern willow flycatcher being listed as endangered species. Current emphasis is being placed on understanding the recent geomorphic and hydraulic geometry changes, their respective effects on the endangered species, and developing action plans that will restore more suitable habitat while still meeting water delivery obligations and protecting important riverside facilities.

In this paper, the history of the Middle Rio Grande is briefly reviewed, historic and current geomorphology presented, together with several restoration goals and activities. The goal of these restoration projects generally is to create wider shallower flow conditions, re-connect the main channel with a flood plain, establish riparian areas comprised of native plant communities, and provide for some channel migration within geographic limits. The connection between the channel geomorphology and these ecosystem goals will be reviewed. Features of these restoration plans include constructing Gradient Restoration Facilities (GRF), terrace lowering, channel widening, constructing deformable bank-line protection, removing exotic tree species, and planting native plant species.

GEOMORPHOLOGY

Hydrology and Sediment: Water and sediment flows have changed dramatically effecting channel pattern, sinuosity, and hydraulic geometry. At the SanMarcial gage from 1896 to 1945 annual flows averaged about 1,100,000 acre-ft, from 1946 to 1978 annual flows averaged about 570,000 acre-ft, and from 1979-1993 annual flows averaged about 1,100,000 acre-ft. In other words, 33% of the record is about 570,000 acre-ft., and 67% of the record is about 1,100,000 acre-ft. Annual peaks have reduced during the period of record from a range of about 20,000 to 30,000 cfs down to less than 10,000 cfs. Typical spring runoff peaks are reduced by control at Cochiti Reservoir as shown in Figure 2. Table 1 shows the reduction in sediment load at four gages on the Middle Rio Grande.

Channel Response: These changes in hydrology and sediment have initiated a channel response similar to the qualitative model developed by Schumm (1977):

$$Q^{-}, Q_{sb}^{-} \rightarrow w^{-}, d^{\pm}, (w/d)^{-}, \lambda^{-}, S^{+}, s^{\pm} \qquad \text{Eqn. 1}$$

where: Q = water discharge (e.g. mean annual flood)
Q_{sb} = bed material load (expressed as a percentage of total load)
w = channel width
d = channel depth
w/d = width/depth ratio
λ^{-} = meander wavelength
S = sinuosity
s = channel slope

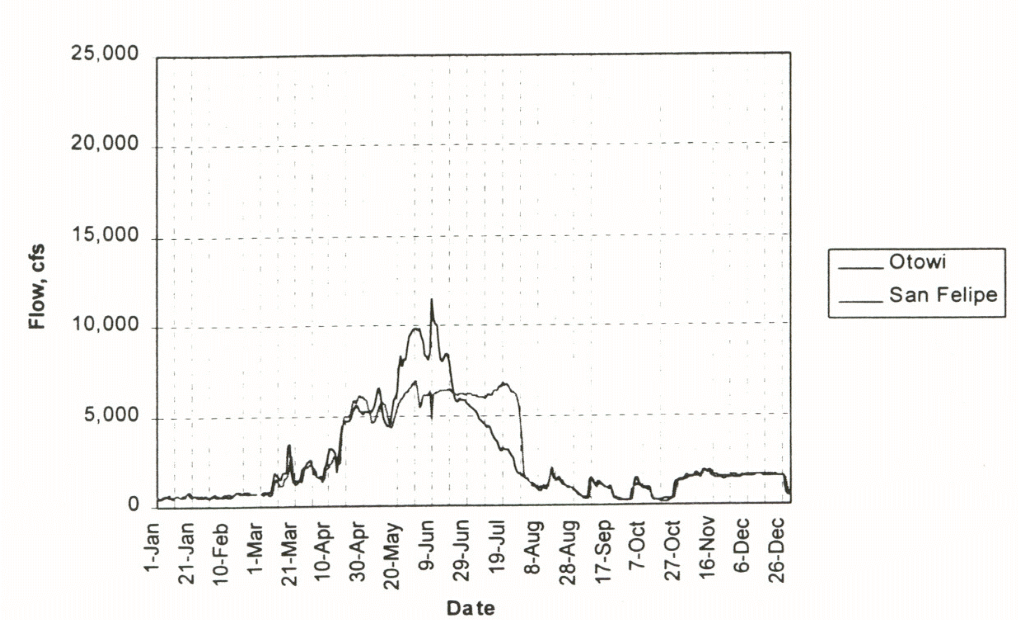

Figure 2. Cochiti Reservoir Inflow (Otowi) and Outflow (San Felipe) 1979 Hydrograph

Table 1. Sediment Load Changes on the Middle Rio Grande

Gage Name	Period of Record	Ave. Sediment Concentration (mg/L)	% of Historic Sediment Supply
Albuquerque	1970-1974 1974-1996	3,750 580	 15%
Bernardo	1965-1977 1977-1996	2,760 740	 37%
San Acacia	1946-1978 1978-1997	13,300 2,600	 20%
San Marcial	1925-1974 1974-1997	12,100 3,800	 32%

The A-A and A+@ superscripts indicate a reduction or increase in each of the values. Equation 1 shows that with a decrease in both water and bed material load, the channel width and width/depth ratio decrease as well. The depth may increase or decrease depending upon the relative magnitude of the changes in Q and Q_{sb}. For this case the depth increases indicating that the importance of the bed material decrease has more influence on the depth than does the reduction in discharge.

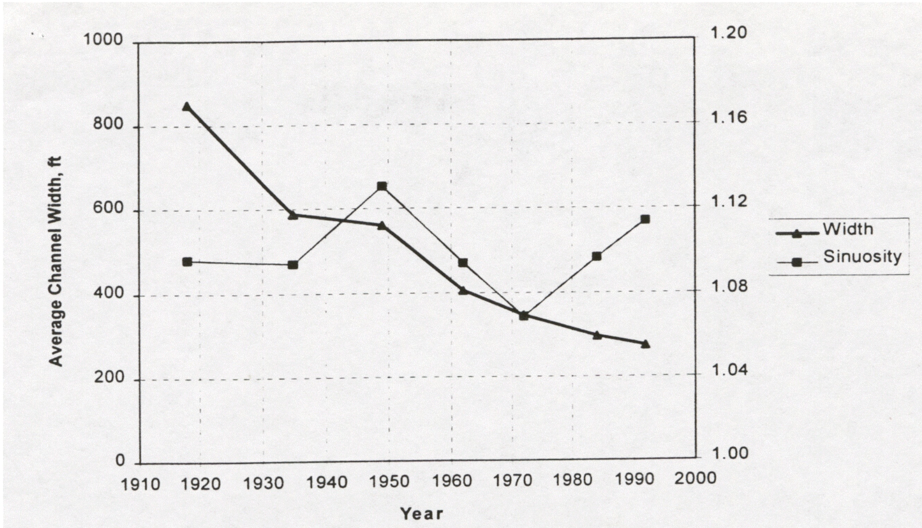

Figure 3. Active Channel Width and Sinuosity from Cochiti to Bernalillo

Figure 3 shows the change in width and sinuosity for the reach from Cochiti to Bernalillo. Table 2 shows the width narrowing trends for the reaches indicated (see Figure 1 for location along the river). Width decreases range from 65 to 77% of either 1918 or 1972 widths. Table 3 shows the increases in flow depth range from 45 to 125% of 1962 depths. Table 4 shows velocity increases ranging from 22 to 34% for the 25-30 year periods of record shown.

Table 2. Width Changes on the Middle Rio Grande

Reach	Period of Record	Reach Average Width (Ft)	% of Historic Width
Cochiti to Angostura	1918 1962 1992	850 400 250	29%
Angostura to Bernalillo Bridge	1972 1995	1150 400	35%
Rio Puerco to San Acacia	1918 1962 1992	1750 700 400	23%
San Acacia to San Marcial	1918 1962 1992	1600 500 425	27%

Table 3. Depth Changes on the Middle Rio Grande

Reach	Period of Record	Reach Average Flow Depth (Ft)	% Increase
Angostura to Bernalillo	1962 1999	1.6 3.6	125%
Rio Puerco to San Acacia	1962 1992	2.5 3.6	45%
San Acacia to Escondida	1962 1999	2.0 4.0	100%

Table 4. Velocity Changes on the Middle Rio Grande

Reach	Period of Record	Reach Average Velocity (Ft/s)	% Increase
Angostura to Bernalillo	1971 1995	3.2 4.3	34%
Rio Puerco to San Acacia	1962 1992	3.7 4.5	22%
San Acacia to Escondida	1962 1992	3.7 4.5	22%

In many reaches the river has not yet reached a new dynamic equilibrium condition, the bed elevation is still lowering, and the channel is continuing to narrow. Table 5 shows the amount the average bed elevation has lowered and Figure 5 shows the average bed elevation trend for the San Acacia to Escondida reach from 1962 to 1999.

Table 5. Average Bed Elevations on the Middle Rio Grande

Reach	Period of Record	Average Bed Elevation Lowering (Ft)
Angostura to Bernalillo	1971-1995	7.3
Rio Puerco to San Acacia	1962-1992	3
San Acacia to Escondida	1962-1999	9.6

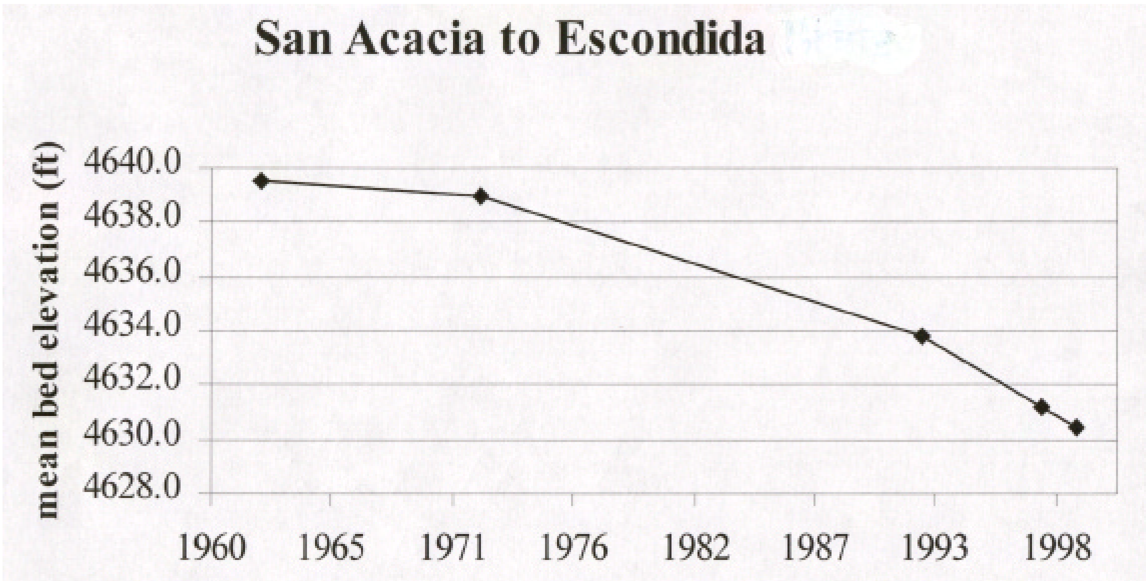

Figure 5. San Acacia to Escondida Average Bed Elevation

The bed material size has also changed over time. The Angostura to Bernalillo reach is now a gravel bedded channel where it was historically sand, the Rio Puerco to San Acacia reach is now a partially gravel bedded channel, as is the San Acacia to Escondida reach. It is estimated that in the San Acacia to Escondida reach the bed will be entirely gravel in about 3 years.

Other Influences: There are several other factors that affect the channel hydraulic geometry in addition to water and sediment changes. During the period of the late 1800's and early 1900's overgrazing in the watershed likely increased the sediment supply. In addition, during this same period water diversions occurred in the Alamosa, Colorado area (see Figure 1) thereby reducing discharge. These water diversions coupled with increased sediment supply may have resulted in a wider channel during this period. Large floods during the early part of this century also contributed to the large channel width. In addition, channel narrowing in the period before 1962 also resulted from channelization activities that included constructing large Kelner Jetty fields, and levee construction in the reach from San Acacia to San Marcial. However, since 1962 the primary effects are the lower flood peaks and lower sediment loads as a result of upstream reservoir construction and reduced sediment delivery in the entire basin.

CHANNEL RESTORATION

Restoration Goals: There is a lot of unknown information about the hydraulic geometry of the Middle Rio Grande prior to the 20th century, therefore, restoration is envisioned to help the channel achieve conditions conducive establishing and maintaining viable populations of the Rio Grande silvery minnow and the southwestern willow flycatcher. Restoration goals include: 1) increasing sediment supply, 2) increasing peak flows, 3) reconnecting the main channel with the abandoned floodplain, 4) storing sand sediment in the main channel, and 5) creating wider, shallower depth, slower velocity habitat.

Restoration Activities: Several restoration activities have been undertaken on an experimental basis. One goal is to monitor these restoration sites and evaluate their performance and learn from the empirical data to enable more effective restoration to be achieved. Potential restoration activities include: 1) channel widening, 2) terrace or overbank lowering, 3) constructing gradient restoration facilities (GRF), 4) constructing high flow side channels, 5) restoring the native riparian habitat mosaic, including salt grass, shrub, and Bosque communities, 6) constructing deformable bank-line protection, and 7) combinations of items 1-6.

Restoration Opportunities, and Challenges: The non-equilibrium conditions over the last several thousand years, dramatic changes in hydraulic geometry, and channel pattern changes present the largest challenges for river restoration. The amount of sediment remaining is not enough to maintain a wider channel with a shallow depth and sand bed that are vital to re-establish viable populations of the silvery minnow. While this provides a huge challenge for effective restoration, it provides numerous opportunities for applying innovative techniques, and monitoring the results and adapting later restoration efforts. Estimating future equilibrium conditions after restoration is also presenting challenges because the available width estimation techniques do not readily correspond to the Middle Rio Grande.

CONCLUSIONS

The Middle Rio Grande has reduced to 23 to 35% of the historical width, the depth has increased 45 to 125%, many reaches that were previously sand bedded are now gravel bedded, and the channel pattern is changing to meandering. Many of these changes were goals of the Middle Rio Grande Project as authorized by Congress in 1948 and 1950. From an engineering perspective the project has been successful. However, the channel changes have contributed to the decline in the number of aquatic species and to the listing of the silvery minnow. Channel maintenance activities now focus on restoration of the river to a condition that will support viable silvery minnow populations while meeting traditional goals of effective water and sediment transport and protection of important riverside facilities.

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USING A WATERSHED-BASED NATIONWIDE PERMIT #27 TO ACHIEVE STREAM RESTORATION GOALS

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Abstract: Steamboat Creek (SBC) is considered the major contributing tributary of non-point source (NPS) pollution to the Truckee River. Steamboat Creek runs through the Truckee Meadows, a growing urbanized area. Historic alterations to grade level in the Truckee River resulted in severe downcutting and bank erosion along a substantial portion of the creek. NPS pollution in the forms of excess sediment, nitrogen, phosphorus and trace metals has resulted in the tributary being listed as "target impaired waters" by the Nevada Division of Environmental Protection. In a cooperative effort to reduce bank erosion and non-point source pollution to the Truckee River, a Steamboat Creek Restoration Plan was finalized in 1998.

The Plan is based on an analysis of the fluvial geomorphology of the stream system. It was developed in a collaborative process by compiling existing land-use plans and information about proposed developments adjacent to the Creek; identifying opportunities and constraints along the Creek based on existing and proposed land use conditions; and coordinating development activities between Washoe County and the City of Reno. The Plan provides reach-by-reach recommendations for the implementation of restoration projects, and serves as a guide for policy makers, landowners, developers, and citizens with interest in improving water quality and conserving riparian zones.

The success of the Plan enabled Washoe-Storey Conservation District (WSCD) to obtain a Nationwide Permit #27 through the U.S. Army Corps of Engineers to be used within the Steamboat Creek watershed. This allows all projects planned within the watershed to be brought to the Restoration Steering Committee for review of compliance with the Plan and the potential for application of the District's permit, providing a substantial savings in permit application time for the developer.

To further achieve our goals, WSCD and the Steamboat Creek Steering Committee have identified three categories of activities to be implemented.

The ***Public Outreach Component*** will provide community outreach needed to engage and maintain both public and agency involvement in the Steamboat Creek Restoration Plan. The Steering Committee continues to invite public participation in the planning process. We propose to launch a coordinated public outreach effort using presentations to diverse interest groups, vision development, community action days, media releases, information on the Restoration web page, and printed materials to increase public participation and knowledge of the restoration effort.

The ***Monitoring and Evaluation Activities Component*** will better share information and coordinate efforts of many people, businesses, organizations and agencies, by developing a

Geographic Information System (GIS) for the Steamboat Creek Watershed. The GIS is important for time-sensitive planning by cooperators so that the vision of a restored stream can become reality in the fast-paced environment of ongoing suburban development.

The ***Implementation Component*** will identify areas in need of restoration. These activities will be prioritized by the Watershed Coordinator, the Steering Committee and WSCD. The Coordinator and support staff will assist in the coordination of projects, including design, implementation oversight, monitoring and assessment. Highlights of several proposed projects are presented.

INTRODUCTION

History of the Steamboat Creek Watershed: The 240-square-mile Steamboat Creek (SBC) watershed bisects the valley referred to as the Truckee Meadows that lies between the Carson and Virginia Mountain ranges near Reno, Nevada. SBC flows from Little Washoe Lake 17.5 miles north to intersect the Truckee River near Vista (see Figure 1). Geothermal areas located adjacent to the creek are the source of elevated levels of arsenic, boron, chloride, and total dissolved solids.

Prior to the arrival of European settlers, the Washoe Indian tribe occupied the Truckee River Basin. When early European settlers arrived in western Nevada, they soon began to dry out wetlands and marshy areas at the mouth of SBC, and then to divert its waters for use in irrigation. Today, SBC is a highly regulated system used for transporting water for irrigation and agricultural uses. Many ditches withdraw and return water to the Creek. Entire sections of the Creek have been straightened and realigned in order to achieve a faster and more continual flow of water that could be used to flood fields. As a result of the agricultural water diversions, flows in SBC vary from the historic regime, being both higher and lower in various reaches during peak irrigation periods.

The most significant alteration to SBC was the artificial lowering of the water surface in the Truckee River by the Army Corps of Engineers in the early 1960s at Vista Reef east of Reno as part of a flood control project. The result was accelerated erosion, downcutting, and deeply incised, 8 feet to 12 feet deep banks. Downcutting was halted only by the construction of a grade control structure several miles from the confluence with the Truckee River.

Today, much of the land use is in transition from agricultural to urban. Water impoundments trap sediments and alter the sediment supply along the creek system. Portions of the creek have been channelized into a trapezoidal section to create a large flood channel.

Water Quality Impairment: With the advent of concerns about nonpoint source pollution and increasing urbanization of the SBC watershed, interest arose in determining the magnitude and importance of inputs of pollutants from the watershed. Analysis of bimonthly data collected by the Nevada Division of Environmental Protection from 1987 to 1991 revealed high total nitrogen loading, particularly from urban/ag drains (Reuter and Goldman, 1993). The mean annual load of total nitrogen at the farthest downstream sampling site, immediately upstream of inputs from

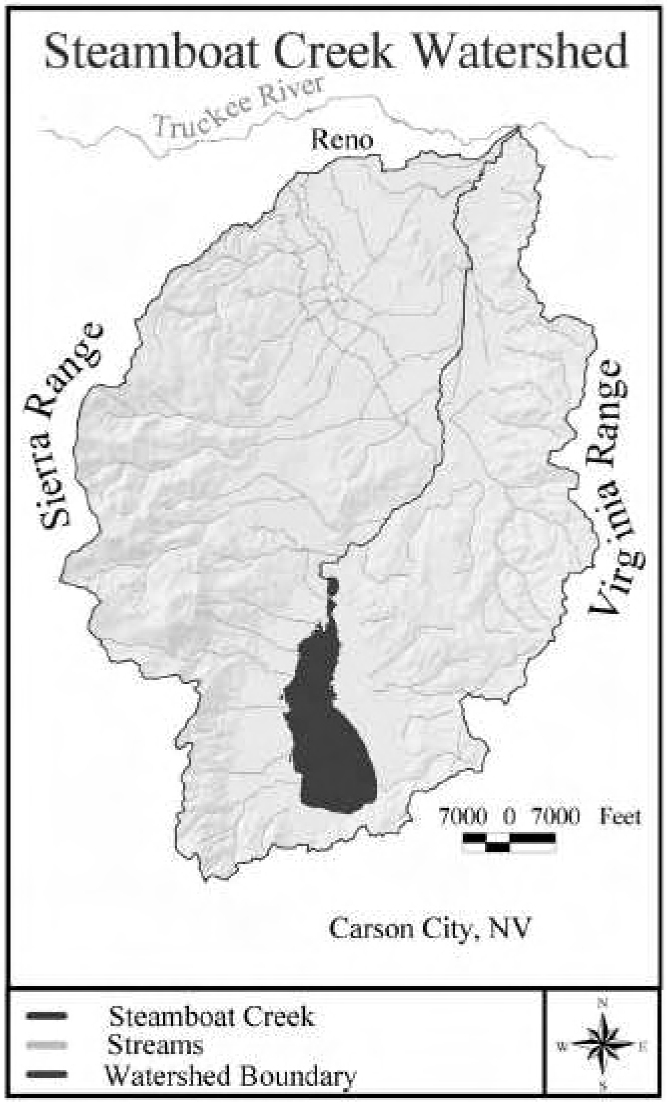

Figure 1: Steamboat Creek Watershed

the Truckee Meadows Water Reclamation Facility (TMWRF, or the regional wastewater treatment plant), approached 250 pounds/day in 1988, or about 50% to 55% of the total nitrogen load from TMWRF. Mean annual loads of total phosphorus averaged about 40 pounds/day over the study period. Arsenic and boron routinely exceeded water quality standards.

By 1994, SBC had been listed as "target impaired waters" by the Nevada Division of Environmental Protection (NDEP, 1994). Preliminary analysis of more recent data reveals several flushes of total nitrogen in excess of 600 pounds/day in 1995 to 1998, likely due to high flow events following a prolonged drought period from 1986 to 1994. The mean annual daily load of total nitrogen (based on monthly sampling) rose from 386 pounds/day in 1995 to 634 pounds/day in 1996 (unpublished data). The current total maximum daily load in the Truckee River downstream of the confluence with SBC at Lockwood is 1000 pounds/day, with a waste load allocation of 500 pounds/day annual average or 30-day average (May to October) for TMWRF (NDEP, 1994). Mean annual daily phosphorus loads show a similar trend, increasing from 67 pounds/day in 1995 to 132 pounds/day in 1996. The TMDL at Lockwood for total phosphorus is 214 pounds/day, with a load allocation of 80 pounds/day from NPS and background inputs.

As the Truckee Meadows area continues to grow and urbanize, TMWRF is reaching its regulatory limits. Interest is focusing on potential pollutant trading that could occur if loads in SBC were reliably decreased as a result of restoration activities involving wetland and riparian area construction coupled with streambank erosion control.

The Challenges of Urban Stream Restoration in Reno: Nevada has experienced unparalleled growth in the past 30 years. The population of the Truckee Meadows (mean statistical area) has increased from 121,068 in 1970 to 308,700 in 1997 (EDAWN, 2000). This rapid growth has fueled the transformation of agricultural lands into suburban and urban developments, resulting in water quality impairment related to septic systems, stormwater runoff, increases in impervious surface area, and home landscape management. Within the SBC drainage, 98 percent of the land is privately owned, with the majority either developed or approved for development. While it was clear that SBC represented a significant source of pollution to the Truckee River, the difficulty of working with the many private landowners and developers led to a cooperative effort to develop a restoration plan, vision, and goals for SBC.

DEVELOPMENT OF THE STEAMBOAT CREEK RESTORATION PLAN

Plan Initiation: By 1996, the Washoe-Storey Conservation District (WSCD) and University of Nevada Cooperative Extension had initiated development of the Steamboat Creek Restoration Plan to address concerns about nonpoint source pollution loads to the Truckee River (WSCD, 1998). The plan was funded by the Nevada Division of Environmental Protection (NDEP) through a Clean Water Act 319(h) grant and a Regional Water Planning Commission grant to promote voluntary efforts by the community to improve water quality. The plan builds upon information developed by WESTEC, Inc. in their Steamboat Creek Fluvial Geomorphology Study that discussed flow and sediment transport conditions and the potential for physical changes including aggradation, degradation, lateral channel migration, and the potential for excessive bank erosion (Myers, 1994).

In January of 1996, WSCD hired the firms of Jeff Codega Planning/Design Inc. and WESTEC to prepare a Stream Restoration Plan for Steamboat Creek. The Plan was developed under the guidance of the Steamboat Creek Steering Committee and WSCD to meet the District’s water quality improvement goals.

The approach was to compile existing land-use plans and information about proposed developments adjacent to the creek, identify opportunities and constraints along the creek based on existing and proposed land use conditions, coordinate between Washoe County and the City of Reno, recommend best management practices for specific reaches of the creek, provide design recommendations to establish continuity between restoration projects, increase public awareness, and provide recommendations for public policies and implementation strategies that target implementation by developers and voluntary participation by private property owners.

Building a Steering Committee to Develop Vision and Goals: Early in the planning process, a Steering Committee was formed to include as many public and private agencies and citizens as possible. This included regulatory and non-regulatory agencies, Citizen Advisory Board members, and private landowners within the watershed. WSCD and University of Nevada Cooperative Extension believed the best way to encourage participation was to educate the public about the plan and technical resources available to them with a voluntary non-regulatory approach. The steering committee identified the following vision statement:

"The Steamboat Creek Restoration Project is a community-wide, cooperative effort to restore, enhance, and preserve the Steamboat Creek Watershed."

The Steamboat Creek Restoration Plan seeks to develop Steamboat Creek and its tributaries into a multifaceted corridor (WSCD, 1998). Included in the vision is the desire to restore the creek as an amenity containing recreational trails and open space within future urbanized areas, as a stable, non-polluting stream channel, as a wildlife corridor and viewing area, and as a respected natural feature through the numerous small ranches and parcels that exist in close proximity to the creek today. Land within the creek should be showcased for the aesthetic and recreational enjoyment of future generations.

Goals: In order to assess opportunities and constraints for stream restoration and determine a plan of action, the following project goals were identified and prioritized by the Steering Committee:

- Improve the water quality of Steamboat Creek.
- Restore Steamboat Creek to a sustainable condition.
- Re-establish wildlife habitat appropriate for individual stream reaches.
- Re-establish vegetation appropriate for individual stream reaches.
- Combine stream restoration with recreation in areas appropriate for public access.

IMPLEMENTING THE PLAN

With completion of the plan, it soon became clear that a mechanism was needed by which the Conservation District would be able to provide input on conformance of proposed projects within the watershed to the restoration effort. At the request of the Army Corps of Engineers, WSCD applied for a Nationwide Permit #27 to permit restoration activities within the entire watershed area. Grant funds were obtained to hire a restoration coordinator to oversee use of the permit and implementation of the plan.

Provisions of a Nationwide Permit #27: The Nationwide Permit #27, stream and wetland restoration activities, applies to restoration projects that serve the purpose of restoring "natural" wetland hydrology, vegetation, and function to altered and degraded non-tidal wetlands and "natural" functions of riparian areas (U.S. ACE, 1997). On granting the use of the permit to WSCD, in addition to standard requirements, the ACE placed the following restrictions: 1) Projects must include on- and off-stream best management practices; 2) Copies of all projects under consideration for use of the permit must be provided to ACE and other appropriate resource agencies for review and concurrence; 3) No construction will be allowed during the active nesting season (to comply with the Migratory Bird Act); and 4) Notification must be provided to Nevada Division of Environmental Protection prior to construction.

All projects that fall within the SBC watershed that may qualify for use of the permit are then referred to WSCD and the Steering Committee for review and consideration. It quickly became necessary to develop a process by which projects could be evaluated for their conformance with both the restoration plan and the allowable activities under the Nationwide Permit #27. The process involves submission of a preliminary application that describes the proposed project and its purpose as well as activities that may affect wetlands, waters of the United States, or existing plant and animal communities. In the event that the project fits appropriately under the jurisdiction of the permit, the time required for the permitting process is greatly shortened (to the benefit of the developer), and WSCD is able to provide input on the relationship of the project to the restoration plan vision and goals, and assist the developer in conforming to the plan.

Clean Water Action Plan: With the restoration plan and the Nationwide Permit #27 in hand, the Steering Committee was ready to engage the general public in the effort to restore the creek. SBC was prioritized for funding under the Clean Water Action Plan, allowing the development of three concurrent components. The *Public Outreach Component* provides community outreach needed to engage and maintain both public and agency involvement in the Steamboat Creek Restoration Plan. This involves the development of a creek logo, community action days, media releases, information on the Restoration web page, and printed materials to increase public participation and knowledge of the restoration effort. A first-ever public conference is planned for February, 2001 to involve the public in the project.

The *Monitoring and Evaluation Activities Component* allows rapid access to existing information on land use, natural resources inventories, water quality data, and more via a Geographic Information System (GIS) for the Steamboat Creek watershed. The GIS is an important tool for time-sensitive planning so that the vision of a restored stream can become reality in the fast-paced environment of ongoing suburban development. The GIS was developed with the assistance of the Washoe County Water Resources and Community Development Departments and the UNR Department of Environmental and Resource Sciences.

Implementing Projects: The *Implementation Component* identifies areas in need of restoration activities. Projects are prioritized by the watershed coordinator, the Steering Committee, and WSCD. The Coordinator and support staff assist in project planning and completion including design, implementation oversight, monitoring and assessment. Highlights of three projects that make use of the District Nationwide Permit #27 are presented below.

Jumbo Creek: Jumbo Creek originates on the rural western slopes of the Virginia Range and flows westerly to Washoe Lake. The drainage area passes through the residential community of New Washoe City where it also receives stormwater runoff. The channel was seriously degraded due to erosion and downcutting associated with a major 1997 rain-on-snow event. The channel was incised for approximately 1700 feet with steep vertical walls over 8 feet deep, preventing access to its floodplain.

The original design proposed by Washoe County Public Works Department and FEMA called for hardening the channel using concrete lining. WSCD proposed to modify the design by integrating the conventional engineering techniques with bioengineering techniques to restore and enhance the riparian values that had been lost while also controlling downstream sedimentation. Because the area where the incision took place is an urban setting and space was limited, using strictly vegetative techniques and restoring the channel to a natural channel geometry was not possible. In 1999, a series of grade control structures (gabions) were used and between these structures a hydroseeded turf reinforced mat (TRM) geo-textile, backfilled with topsoil, was used for bed and bank protection. Different native vegetation mixtures were used to promote rapid regrowth as well as control and trap sediment movement. Monitoring is ongoing.

Anderson/Bartley Ranch Park: The stream classification method is proposed as an assessment and design tool for the realignment of the Evans Creek channel (tributary to SBC) through Bartley Ranch Park in Reno, Nevada. The current channel conditions can be classified as braided as a result of a 1997 rain-on-snow event and associated avulsion. The project proposes to relocate the braided channel through the current meadow system and alleviate the associated headcuts by developing a Rosgen E-type channel that reduces the slope and controls the velocities in this area by providing more frequent access to the floodplain. Addressing the current condition and the associated headcuts is critical to the project and will improve water quality downstream.

Currently there is active grazing on this rangeland and the natural ability of the vegetation to reduce velocities and control sediment delivery from the rapidly urbanizing upper watershed is compromised. A management plan for future livestock access will be addressed as part of this restoration process. A Proper Functioning Condition (PFC) assessment has been conducted for the site and the District recently hired a local engineering firm to conduct a more thorough and quantifiably based assessment as part of the project design.

Steamboat Creek at Andrew Lane: Steamboat Creek at Andrew Lane is high on the District's priority list. A number of years ago the floodplain for this section of the channel was filled with concrete rubble material in an effort to stabilize the channel and reduce flooding. As a result of these activities, the overall geometry of the channel in this section has changed. The protection that the concrete provides is inadequate and the associated higher velocities have resulted in localized areas of erosion. Material has migrated into the channel where it is unstable, restricts many of the natural functions of the stream, and is a potential problem to downstream landowners.

The conceptual proposal for this site is to remove approximately 250 cubic yards of concrete material that was used as fill material on the floodplain and for bank protection and stabilize areas of the eroding bank. Appropriate vegetation will be reestablished using a combination of willow pole plantings, wattles and coir fiber fascines to serve as temporary toe protection until the vegetation becomes established.

SUMMARY

A unique and rapidly vanishing opportunity to restore Steamboat Creek and meet multiple objectives led to the development of a restoration plan. To allow optimal coordination of development plans with the restoration plan goals and decrease lag time for permit application, a watershed-based Nationwide Permit #27 for riparian restoration was obtained by the Washoe-Storey Conservation District. In addition to project implementation under the Nationwide permit, a watershed GIS has been developed and an outreach program is underway to gain citizen support for the effort.

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A HIERARCHICAL RIVER STABILITY/WATERSHED-BASED SEDIMENT ASSESSMENT METHODOLOGY

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ABSTRACT: A hierarchical framework utilizing a prediction methodology is presented that provides a basis for the assessment of both suspended and bedload sediment. The objective of this watershed scale approach is to identify; natural variability in sediment; geologic versus anthropogenic sediment sources, erosional and depositional process sources of sediment; streamflow changes, and stream channel stability conditions.

Various levels of assessment are intended to identify watershed and river effects based on land use activities that: 1) Indicate potential change in streamflow/sediment relations, channel stability and associated evolutionary states at a broad level of investigation; 2) Provide a prediction methodology used to quantify streamflow, erosion and sediment relations at an intermediate level of investigation; and 3) Establish validation monitoring methods at the most detailed level of determination. The various levels from broad, intermediate and detailed are selected from an initial determination of the nature, magnitude, severity, and consequence of potential change in sediment relations. Examples of the three levels are presented.

The application of "reference reach" data by stream type and corresponding development of regionalized, dimensionless ratio sediment rating curves for both suspended and bedload sediment is used to determine potential sediment departure. The methodology assists in the determination of the direction and magnitude of departure from a reference condition, and in this manner, mitigation measures such as process-specific best management practices, stabilization, restoration, and riparian management can be appropriately recommended.

INTRODUCTION

The general purposes of watershed management assessments are to determine the effects of various land use activities on the quality and quantity of water produced and the physical and biological function of the drainage network. This particular watershed assessment procedure is focused primarily on the changes in erosional processes, streamflow changes due to vegetation alterations and roads, stream channel stability relations and the integration of these processes that potentially change suspended and bedload sediment supply. Increases in sediment supply or availability to the drainage network often results in adverse on-site and downstream consequences. The supportive evidence in scientific literature indicates the consequence of excess sediment load and/or size of sediment often results in stream channel instability (dis-equilibrium). The adjustments due to sediment change can alter the dimension, pattern, profile and materials of the stream channel. The consequences of stream channel adjustments often lead to additional sediment supply due to accelerated stream bed and bank erosion. The cumulative effects of stream channel dis-equilibrium often results in aggradation, degradation, accelerated lateral accretion, increased flood stage for the same magnitude flood, loss of fish habitat, increased land loss, downstream impacts due to sediment supply, change in morphological stream types and adverse short and long-term loss of physical and biological function. Sediment impacts cannot be isolated from streamflow changes. The consequence of increased magnitude, duration, and timing of streamflows can also lead to channel instability and associated adverse impacts.

The author measured bedload and suspended sediment concurrently above and below a braided reach of the East Fork of the San Juan River in 1986 to determine consequence of willow removal conducted in the early 1930's. The results produced an increase in total sediment by 49 % due to streambank erosion on 4.8 km of a pristine 135 km^2 watershed. The willow clearing induced instability and subsequent change in morphological stream type that converted a meandering, 18m wide, gravel-bed stream (C4 stream type), to a braided channel (D4 stream type) and increased the bankfull channel width to 259m (Rosgen, 2001, In Press Interagency Sediment Conf.). Stream channel adjustments of instability and corresponding stream type changes due to anthropogenic influences can be responsible for significant sediment increases beyond geologic rates. Effective restoration of these disturbed systems have been conducted, reducing streambank erosion to negligible rates (Rosgen, 1996 and 2001, In Press), returning sediment levels and channel stability to pre-disturbance conditions.

Improper grazing practices that change riparian vegetation types from woody to grass/forb community types have been responsible for accelerated streambank erosion, channel instability and increased sediment supply (Rosgen, 2001, In Press, Interagency Sediment Conf.). It is essential that any watershed management assessment method ascertain the cause, magnitude, and consequence of change from a stable, reference condition.

Assessment Methodologies. USEPA efforts in developing a watershed-based assessment (WRENNS) were conducted by the USDA Forest Service and EPA (1980). This effort was validated nationwide with extensive peer review. The objectives of this watershed-based approach was to determine potential changes in streamflow, surface erosion, mass wasting, channel stability and sediment yields associated withsilvicultural activities such as timber harvest, vegetation alteration, and road construction. Continued data collection since this effort has shown the value of measured sediment relations coupled with channel stability indicies and morphological stream types (Rosgen, 2001 In Press, and Troendle, et al, 2001, In Press, Interagency Sediment Conf.). Imposed change of streamflow, sediment and/or direct disturbance can cause sediment rating curve shifts associated with stream channel instability that can lead to a changes in stream type, and channel evolution stage shifts. These channel adjustments often lead to both short and long-term accelerated sediment supply changes (Rosgen, 2001, In Press, Interagency Sediment Conf.).

Recent efforts by the USDA Forest Service and USDI Bureau of Land Management (McCammon, et al, 1998 developed a framework for characterizing the hydrologic condition of watersheds. This broad level assessment was developed to ascertain the generalized effects of land uses on water quantity, timing and quality. Similar efforts by States and other agencies are being developed and implemented in order to determine the "cumulative effects" of surface disturbance activities and vegetative changes on water resources, including sediment. Several individual states are currently developing clean sediment, total maximum daily loads (TMDL's) due to recent requirements imposed from the implementation of the Clean Water Act. The TMDL's should recognize natural spatial and temporal variability in sediment, geologic erosion, rather than to set a "fixed" number. It is critical, however, to recognize potential instability and erosion/deposition processes that can cause accelerated, unacceptable sediment loads due to poor land use practices. The ideal direction is prevention, however, the extent and severity of change often requires stabilization and/or restoration measures. An assessment methodology should address the physical processes responsible for potential sediment acceleration in order to more specifically and practically provide a solution to the problem.

METHODS

The methodology presented is designed to provide assessment protocols at three distinct inventory levels: 1**)** ***Broad watershed assessment,*** primarily identifies locations and potential changes in streamflow, erosional processes related to specific land use activities, and altered stream types that are used to set direction for mitigation and/or priorities for a more detailed assessment, depending on potential magnitude and consequence of change; 2) ***Intermediate quantitative prediction***, potential changes in streamflow, erosion, and sediment supply are predicted relating to specific land use activities, spatially located within the watershed. The majority of mitigation, process-specific "Best Management Practices" and recommendations for watershed master plans, and restoration can be accomplished at this level; and 3) ***Detailed process-specific validation/monitoring*** is accomplished including measurements of streamflow, suspended and bedload sediment, channel stability verification, and other sediment related sources. The objective at this level is to provide data for prediction model validation, measurement of the extent, magnitude and consequence of sediment change, and, effectiveness monitoring to document the resultant sediment reductions due to implementation of revised management efforts, stabilization and/or restoration.

Broad Level Watershed Assessment: The objectives for this level of assessment are to identify land use activities by type and extent that may effect sediment yields, streamflow and channels instability. This level of assessment should be the first priority of determination, as it may avoid unnecessary, time-consuming and expensive studies. It will also identify specific potential problem areas that may identify successful management practices and obvious mitigation and/or justify additional, more detailed investigation and prediction of impacts. The overall direction for this level is presented in the flow chart in Figure 1. Much of this analysis can be obtained from existing inventory, time-trend studies from aerial photographs, previous maps indicating location and extent of certain land use activities including roads, timber harvest, land development and similar surface disturbance and vegetative change. Map overlays of the land use activities by existing data indicating geologic hazards (landslides, and faults), soils inventory indicating erodibility categories (surface erosion and mass-wasting potential), and riparian inventory, provides an indication of potential erosion sources. Valley and stream classification is obtained as described in Rosgen (1994, 1996), at level I (Geomorphic characterization level).

The specific objectives at this level are to; 1) Identify location and magnitude (concentration of use) by particularly sensitive geology, soils and/or stream types that potentially may increase sediment levels by various erosional processes such as surface erosion, mass wasting, stream bed erosion (aggradation/degradation), and streambank erosion, 2) Inventory areas impacted by relatively recent vegetation alteration within the watershed exceeding a specified percentage of watershed acreage that would show potential increases in streamflow magnitude and timing change, 3) Identify riparian vegetation composition and density changes and broad level stream type mapping and, 4) Inventory historical direct disturbance to stream channels such as channelization, levee construction, post-flood alterations, floodplain encroachment and abandonment, and other direct disturbances to stream channels that alter dimension, pattern, profile and/or channel materials. The Inventory summaries of the various assessments are shown in Table 1. The land use activities that have the largest extent of area of impact on the most unstable lands and stream types associated with poor land use practices will be "flagged" as potential high to very high sediment sources (Figure 2). Conversely, good land use practices on stable lands and river systems can withstand larger areas of similar activities without significant sediment increases.

The identification of land use activities by specific locations in sub-watersheds that indicate high to very high potential change in erosional processes and corresponding increases in sediment yields are candidates for mitigation and/or changes in management practices which will off-set the potential adverse impacts. The value of the resource, consequence of instability and potential magnitude of sediment increases may justify advancement to the prediction level of assessment. If the general risk relations shown in Figure 2 are questioned, then advancement to the prediction level may also be appropriate to properly evaluate individual stream or land systems identified as high or very high potential. The broad nature of this level of assessment and general relations as shown in Figure 2 may not justify expensive, detailed mitigation including restoration without the benefit of a more detailed prediction level. This broad level assessment initiates the first step to establish a basis for prioritizing potential problem areas that may require a more detailed level of assessment.

Intermediate level - Quantitative Prediction: The procedures documented in Rosgen, (1978) and in USEPA (1980) are appropriate to this level of investigation. The revised flow chart (Figure 3) as presented by Rosgen (1999) depicts the general steps associated with predicting potential sediment increases associated with changes in flow and erosional process changes. Some of the procedures in this method have been recently updated and improved (Troendle, et al 2001, In Press). A computerized version of WRENNS, USEPA (1980) was developed by Rosgen and Silvey (1981) and applied on over one hundred, 4th order watersheds as part of the Arapaho and Roosevelt National Forest Plan (1984). This level of assessment requires prediction of streamflow change, sediment supply associated with streambank erosion, surface erosion and mass wasting. Site-specific field inventory for land and stream channel systems are required at this level of assessment. Stream stability prediction and departure analysis is conducted by stream type for both the reference reach and potentially impaired stream reaches. Procedures for this analysis are summarized in the above mentioned sources with portions presented in Rosgen (1996), associated with "Level II and III" geomorphic description and condition analysis (Chapters 5 and 6). To separate geologic rates from anthropogenic source sediment, the use of reference reach (stable) stream systems of the same morphological types are compared to altered or impaired reaches. The stability analysis and evolution of stream types is used at this level to indicate shifts in sediment rating curves that reflect changes in sediment supply.

Changes in streamflow from timber harvest caused a significant increase in sediment yield due to channel source sediment (Troendle and Olsen, 1993). If channel stability is altered with a corresponding upward shift in the sediment rating curves, then streamflow increases can produce an exponential increase in sediment yield. Measured streambank erosion rates/year conducted by the author on the impaired, unstable reaches of the Lamar River (Montana), Weminuche River, Wolf Creek and the East Fork of the San Juan River (Colorado), have been increased by three orders of magnitude above the reference, stable condition (Rosgen, 2001, In Press). If streamflows are increased due to upstream changes, then sediment yields can be increased significantly over geologic rates.

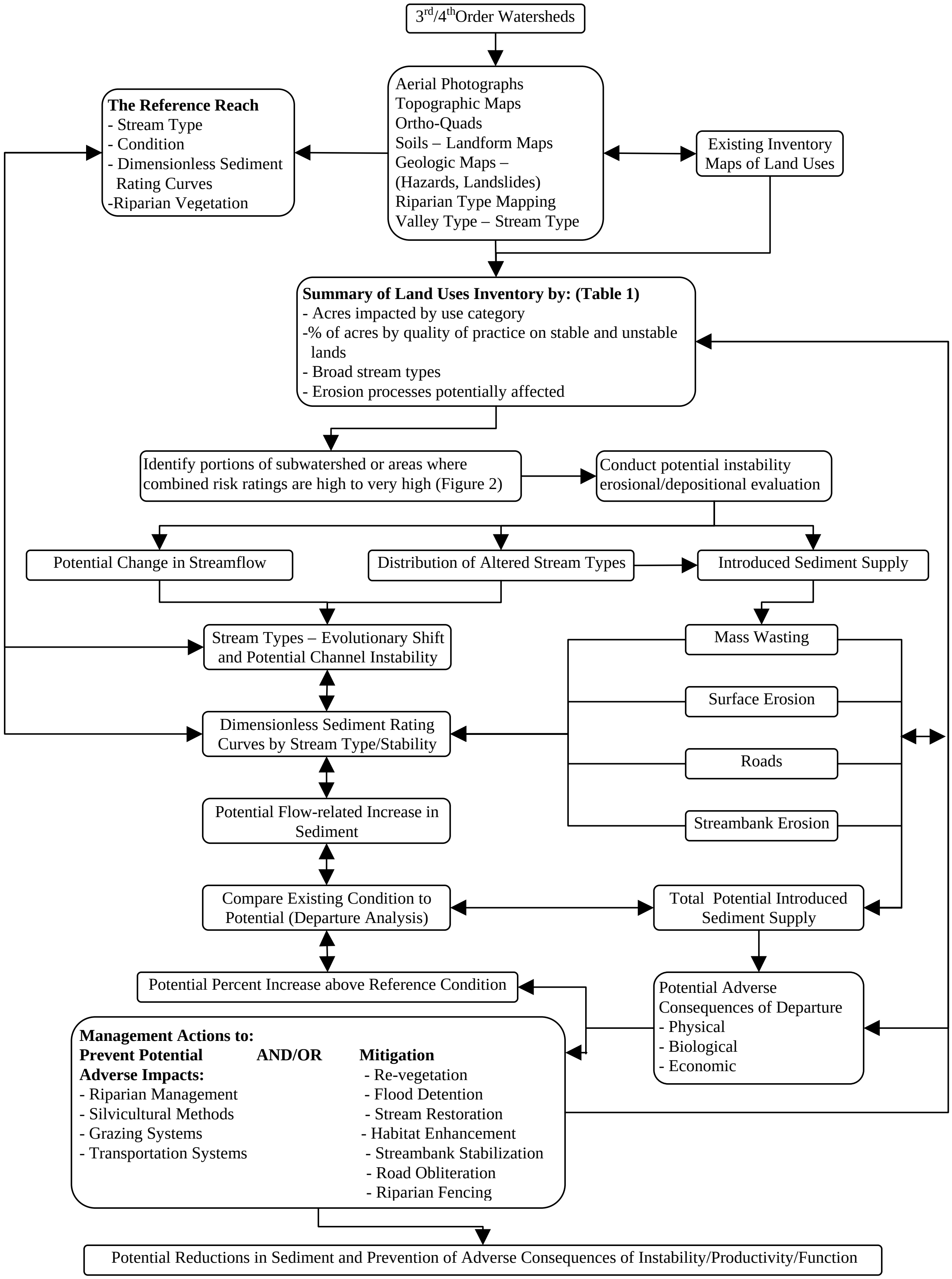

Figure 1. The Broad and/or Initial Level of Assessment

Table 1. Summary of Land Use Inventory by area, location within stable and/or unstable lands stream types, quality of practice, and potential erosion process type

Land Use Activity (Type) General Categories	Total Acres Affected	Stable Lands Quality of Practice (acres)			Unstable Lands Quality of Practice (acres)			Stream Types Affected (%) (Broad Level Classification)								Potential Erosion Process Affected (%)		
		Good	Fair	Poor	Good	Fair	Poor	A	B	C	D	DA	E	F	G	Channel Erosion (bed and bank)	Surface Erosion	Mass Wasting
Mining																		
Hillslope																		
In-channel																		
Silvicultural																		
Partial/Selection harvest (30yrs)																		
Clear cut harvest (last 30 yrs)																		
Roads																		
Road Density																		
# Stream crossings																		
Position of road in drainage																		
Agricultural																		
Riparian vegetation conversion																		
Cropland																		
Hay production																		
Fallow ground																		
Feed lots/concentrated zones																		
Livestock grazing																		
Urban Development																		
Percent Impervious																		
Direct Disturbance to Stream Channels																		
Channelization																		
Levees																		
Lining (concrete, gabions)																		
Degradation/incision																		
Aggradation (excess sediment deposition)																		
Confinement (lateral containment)																		
Summary																		
Total Acres																		
Percent of Acres																		

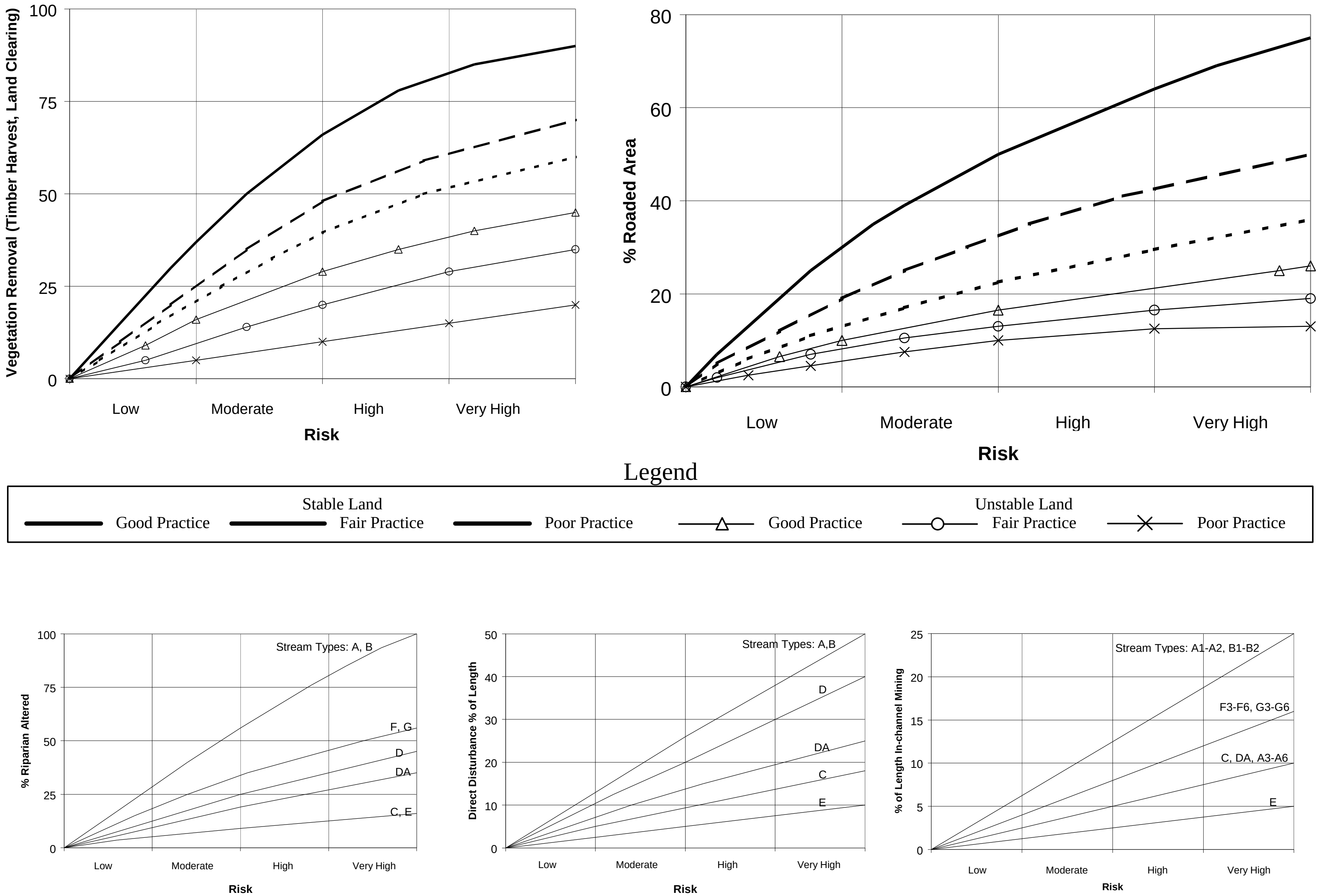

Figure 2: Examples of watershed assessments to determine potential increases in sediment due to the extent of various land use activities, stability of the landscape and rivers, and the quality of land use practices.

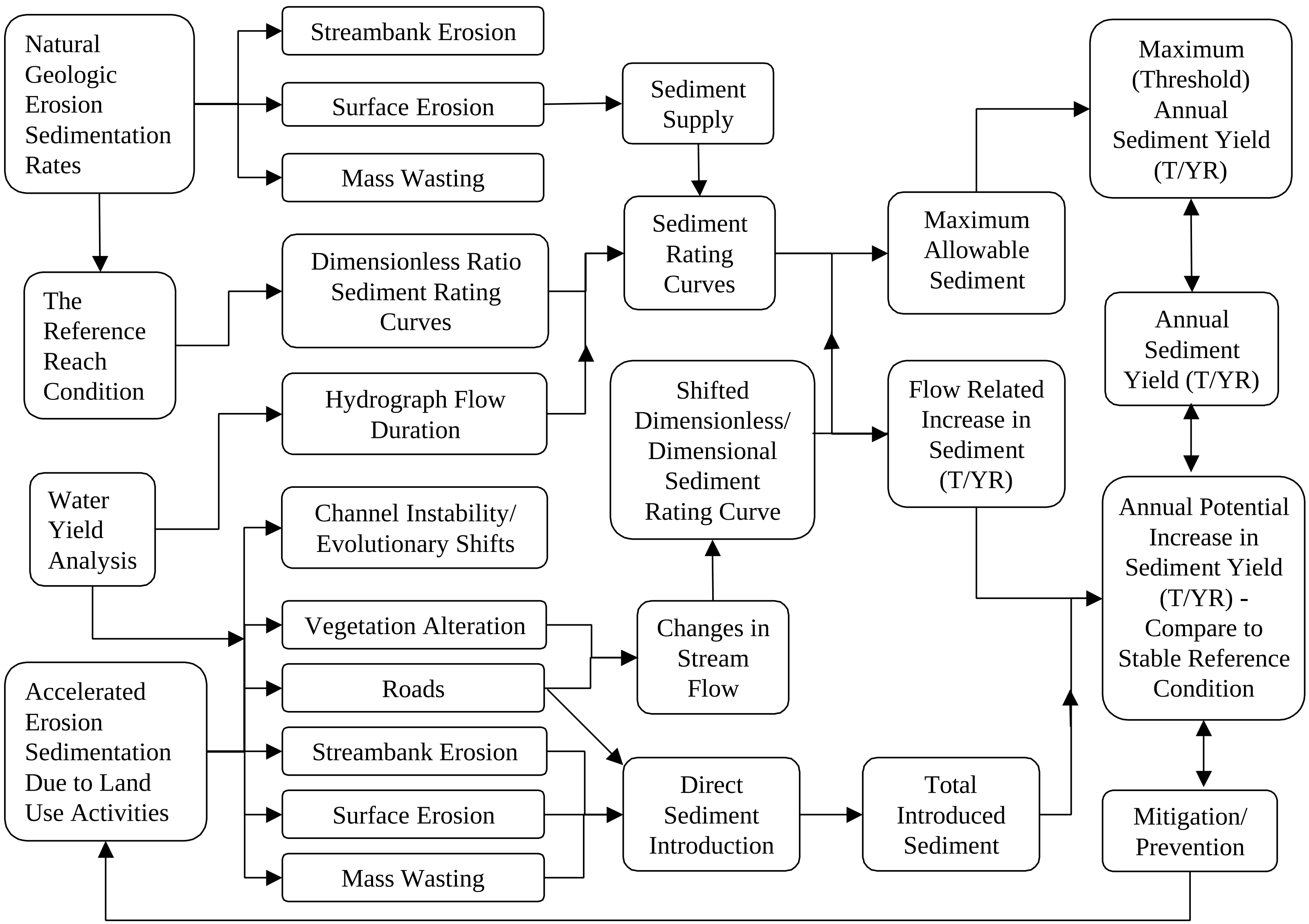

Figure 3. Flow Chart of the Intermediate Level of Assessment, Showing Predicted Comparison of Natural Rates of Sediment Compared to Potential Increase Due to Land Use Activities (from: Rosgen, 1999)

A flow-chart that depicts the list of assessment items for both the reference reach and potentially impacted reaches is shown in Figure 4. The advantage of the quantitative prediction level of assessment is to identify specific process, land uses and specific stream reaches for mitigation, "Best Management Practices", and priorities for restoration. A consistent, quantitative, comparative procedure has merit for prediction at this assessment level, however, as normal for any prediction methodology, the error term associated with prediction can be quite large. If, due to very high values and/or potential serious adverse consequence of change, a validation level may be justified for specific areas, processes and/or stream reaches.

Detailed Process-Specific Validation/Monitoring Level: This assessment level is implemented to; 1) Validate prediction methodologies, 2) Measure the extent of existing flow, sediment and stability problems, 3) Compare measured departure to a reference condition, 4) Provide site-specific mitigation, enhancement and restoration requirements, and 5) Evaluate effectiveness of implemented management practices, mitigation and restoration designed to restore natural channel stability and reduce sediment yields. The same measurements and analysis conducted on impacted reaches are also conduced on reference reaches in order to identify the extent, nature and consequence of departure conditions (Figure 4). The re-survey of permanent, monumented cross-sections indicate the accuracy of the prediction of streambed stability. Validation monitoring includes installation of bank toe pins to profile streambanks annually to measure erosion rates, scour chains are installed vertically in the streambed to verify entrainment sizes and extent of channel scour, and pebble counts taken under permanent cross-sections determine change in particle size distribution of channel materials (Figure 4). Measured suspended and bedload sediment, concurrent with streamflow, allows an analysis of sediment rating curves and continued establishment of dimensionless rating curves to be extrapolated at broader levels of assessment.

Simon (1989) reported significant shifts in the slope of measured suspended sediment rating curves following channelization. The resultant channel instability induced a series of channel evolution sequences resulting in increased sediment supply from both bed degradation and streambank erosion processes. In this study sedimentation rates of 57 metric tons/yr./km^2 (163 tons/year/mi.^{2}) were measured on the Hatchie River, a stage I evolution. The channelized reach of the South Fork Forked Deer River was converted from a previous stable reach associated with an evolution stage I to an evolution stage IV condition (severely degraded and widening). The consequence of this conversion produced 872 metric tons/km^2 (2490 tons/yr. mi.^{2}). This author visited these sites and classified the Hatchie River as an E6 stream type. From field inspection by the author, the channelized South Fork Forked Deer was previously an E6 but was changed to an F6 stream type. The instability caused by channelization and resultant increase in sediment supply and the corresponding shift in the sediment rating curve was also associated with a change in morphological stream type. These stability assessments infer a potential shift in sediment rating curves, which can be measured to verify actual change in sediment/discharge relations.

SUMMARY

Historical watershed management has been primarily associated with planning efforts involving the next development project. Very few measurements or quantitative assessments have been undertaken to determine the effects of past planning implementation on changes in streamflow, sediment and stream channel stability. Research conducted on these subjects has been on-going, however, the state-of-the science is far more advanced than the state-of-the art of implementation of scientific discovery. The cumulative effects of continued watershed demands places a great challenge on technical assessments that can properly evaluate the nature, extent and consequence of change.

Appropriate watershed and river assessments are not simple and are not quick. A greater effort must be expended in order to answer the difficult questions posed by today's society. Hopefully, these assessment levels can assist managers in being more effective in the prevention of adverse change and making appropriate recommendations for process-specific mitigation allowing for attainment of sustained resource benefits. What we have learned in the past must be used to prevent similar potential problems in the future and help direct management in a positive direction. A proper watershed assessment can direct stabilization and/or restoration efforts to maintain the natural stability and the physical and biological function of rivers.

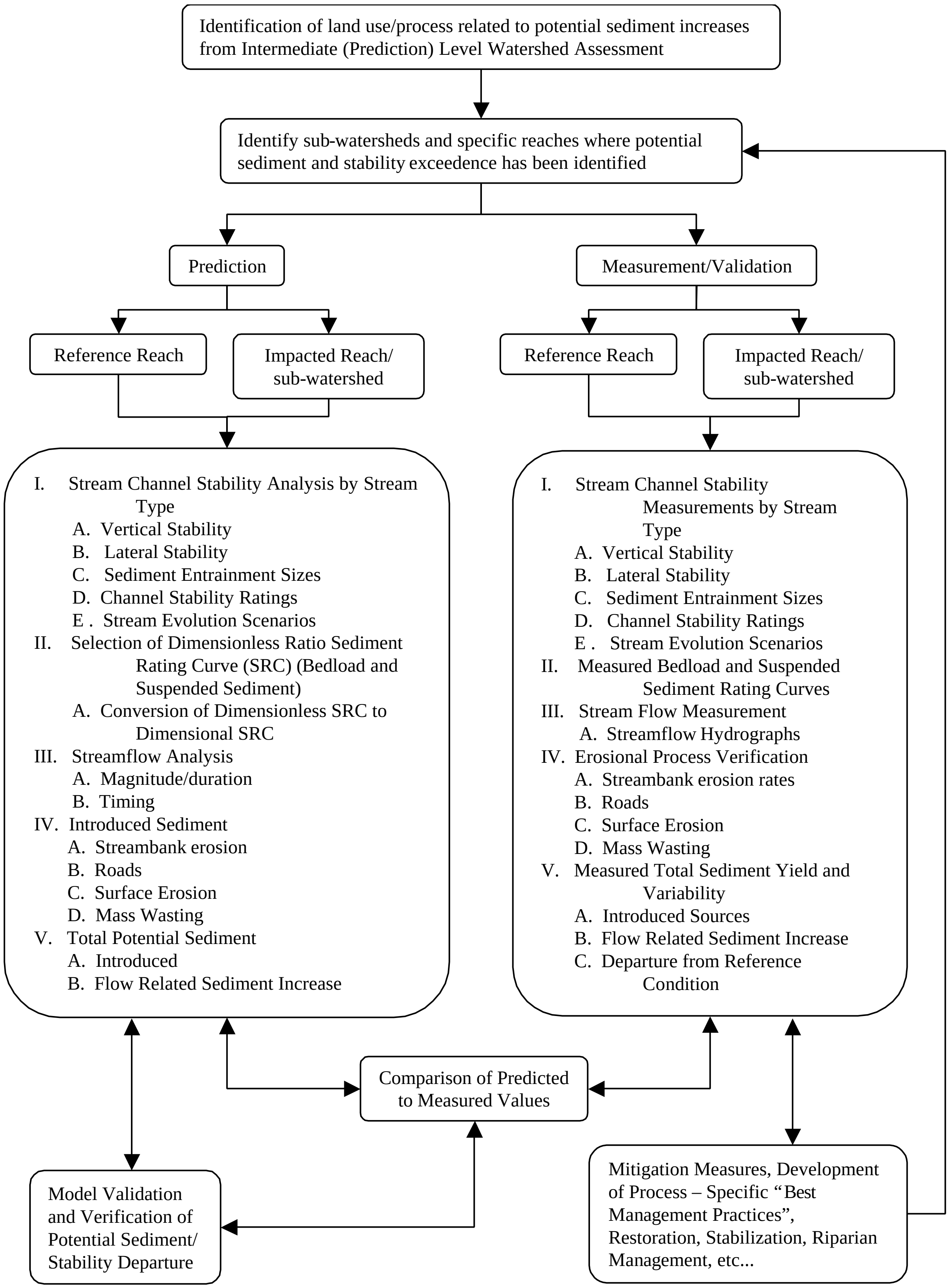

Figure 4. Prediction and Validation Assessment Levels to Determine Magnitude and Extent of Sediment and Stability Departure from a Reference Condition

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NO ACTION ALTERNATIVE FUTURE SCENARIOS FOR THE ELEPHANT BUTTE, NM HEADWATER AREA

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Abstract: The Rio Grande near San Marcial was and continues to be a mainly aggrading system. Aggradation causes a number of problems in the area above Elephant Butte Reservoir. The levee system and the San Marcial Railroad Bridge decrease the capacity causing flows out of Cochiti Dam to be reduced. In addition, the river can become disconnected as the headwaters of Elephant Butte Reservoir recede.

There exist several options for managing the aggradation and the potential disconnection. One option is to discontinue all maintenance in the reach near Elephant Butte Reservoir. However, the sediment deposition coupled with a restricted floodplain and no maintenance activities could cause the occurrence of several situations. The potential scenarios include the breaching of the levee, the formation of a sediment plug, and the minimization of capacity at the San Marcial Railroad Bridge. Geomorphic and modeling analyses were conducted under these scenarios, along with the current conditions and the impacts from construction of a temporary channel, to ascertain the effects on the river system.

Results of the analyses are presented for both the reach wide perspective and the local perspective. Reach averaged hydraulic properties are reported for the local downstream subreaches for each of the scenarios. Channel width, velocity, hydraulic depth, and wetted acreage decreased with decreasing flow limits of the bridge capacity. In addition, despite the increase in bridge capacity due to the temporary channel construction, the channel velocity and hydraulic depth did not experience great changes primarily due to the large increase in channel width. Reach wide, channel widening and/or narrowing is projected in the Middle Rio Grande between Cochiti Dam and Elephant Butte Reservoir parallel with the change in scenario flow limits. Changes in the other hydraulic variables over the entire reach, similar to those of the local downstream subreaches, are expected.

INTRODUCTION

The reach of the Rio Grande above Elephant Butte Reservoir (figure 1) continues to undergo longterm aggradation due to sediment loads that are higher than the transport capacity of the valley slope. The riverbed at the San Marcial Railroad Bridge rose 24 feet between 1895 and 1989. In the late 1950s the US Bureau of Reclamation constructed a channel from San Acacia through the delta downstream about 25 miles into the reservoir. The river was altered to essentially be a canal (Low Flow Conveyance Channel or LFCC) and a flood overflow channel (Floodway). The Floodway was confined to the eastern edge of the valley from San Acacia to the Narrows of Elephant Butte Reservoir by the spoil levee created by excavating the LFCC.

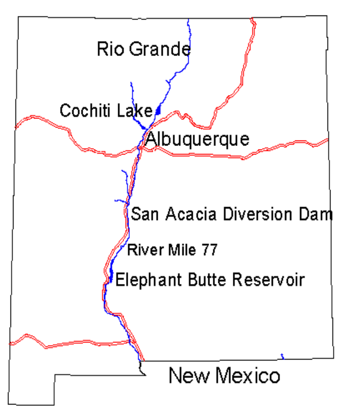
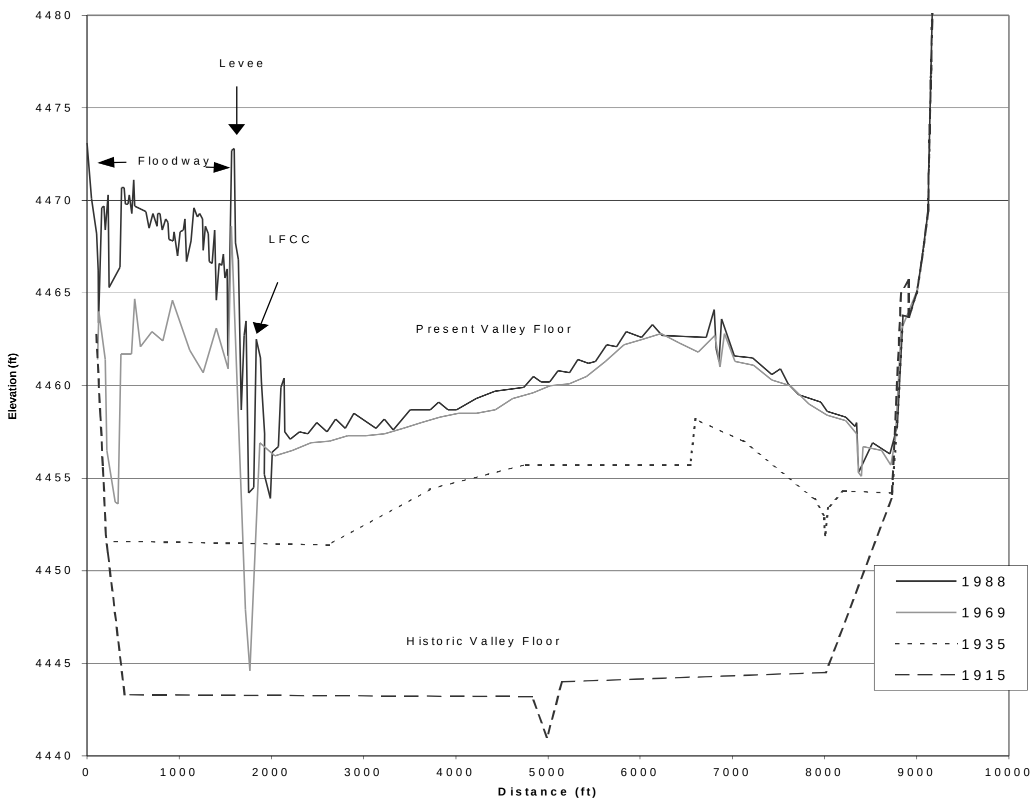


Figure 1. Location map

Continued aggradation and levee raising has decreased capacity and perched the Floodway channel up to 15 feet above the rest of the valley for several miles upstream from the reservoir pool as shown in figure 2. Until very recently, Elephant Butte Reservoir has been relatively full since 1985 and the outfall from the LFCC has become buried in sediment. Maintenance of this outfall was increasingly difficult and diversions to the LFCC (with minor exceptions) were suspended in the spring of 1985.

Figure 2. Historic Cross Section – EB Reservoir Rangeline 12



One option considered in this study was to discontinue maintenance in the headwaters of Elephant Butte Reservoir. Several future situations could arise, and all would have negative impacts on not only the terrestrial and aquatic habitats but also on the quantity and efficiency of water and sediment transport. Some impacts are the result of the reduced outflow from Cochiti Dam so the Middle Rio Grande between Cochiti Dam and Elephant Butte Reservoir would be affected. Cochiti Dam is operated such that outflows are equal to the inflows until the outflow rate is great enough to threaten downstream facilities, i.e. the levee system and the San Marcial Railroad Bridge.

SCENARIO ANALYSIS

Three potential scenarios were analyzed for the no maintenance plan. The first scenario is the anticipated continued reduction in capacity at the San Marcial Railroad Bridge. The second scenario is the breaching of the levee system and the third scenario is the formation of a sediment plug. (Both the second and the third scenarios are based on historical events.) A fourth scenario was also analyzed, constructing a temporary channel through the exposed delta of Elephant Butte Reservoir.

The analyses were completed from two perspectives, the reach wide perspective and the local perspective. The reach wide analysis examines the effects on channel width from outflow restrictions at Cochiti Dam due to the scenarios and reports subreach widths from Cochiti Dam to Elephant Butte Reservoir. The local perspective examines the scenario effects in greater detail from San Acacia downstream. Locally, results are given in twosubreaches; San

Acacia Diversion Dam to river mile 77 (RM 77) (about four miles north of the Bosque Del Apache National Wildlife Refuge's, BDANWR, south boundary or Socorro, SO, rangeline 1596.6) and RM77 to Elephant Butte Reservoir. Variables reported are average channel width, average channel depth, width/depth ratio, average channel velocity, wetted channel acreage, wetted overbank acreage, and total wetted acreage.

<u>Existing Conditions</u>: Reach wide existing conditions are based on the 1992 photodigitized geometry. Modeling of the scenarios for the local analysis used the 1992 river cross sections from San Acacia Diversion Dam to RM77 and the 1998 river cross section surveys from RM77 downstream to define existing conditions. The 1998 main channel cross sections were inserted into the 1988 valley cross sections to model the overbank areas. The 1988 overbank areas were adjusted for the deposition that occurred over the 11 years in the subreach from RM77 to Elephant Butte Reservoir. The aggrading nature of the river from Highway 380 south to Elephant Butte results in areas of the floodplain and valley that are lower than the main channel. Therefore, HEC-RAS designations of levee or ineffective flow areas were applied to elevations in the floodplain to more accurately portray the flow of water through the floodplain. Scour was included in all of the scenarios for the local perspective. With the contraction scour at the bridge, the maximum permissible discharge is 3,800 cfs (1998 data).

<u>Length of Effect</u>: Each scenario will affect the nearby river in the short term. The reach of the river that is immediately affected by a particular scenario, the length of effect, was determined prior to changing the channel widths. The length of effect was established by comparing the hydraulic variables (i.e. channel top width, channel depth, channel velocity) calculated for the modified geometry for each scenario to the hydraulic variables calculated for the existing conditions. Both the scenario and the existing condition models were executed at the same discharge, which was the lower of the two. (For example, in the case of the sediment plug the discharge value used was that of the scenario, 1,400 cfs, but in the case of the temporary channel the discharge was that associated with the existing conditions, 3,800 cfs.) The length of effects for all of the various scenarios ranged from roughly four miles north of the BDANWR's southern boundary (RM77 or SO 1596.6) downstream to the low water crossing (near Elephant Butte at EB rangeline 26).

<u>Width Changes</u>: Width changes were a long term effect projected from width-discharge relationships – assuming the discharge limit flow occurred for several years. Discharge data is available for many gages in or near the reach of interest. Data up through 1992 was used in the regression study. The gages and year data begins are Otowi – 1899, Cochiti –1927, Albuquerque – 1943, Bernardo - 1937, San Acacia - 1937, and San Marcial - 1899. Otowi is included to extend the upstream record, but both Otowi and San Marcial gage records have gaps in the early data.

A long-term record of active channel widths exists from the early 1900s to 1992. The 1908/1918 data are from surveys performed in 1908 and in 1915 to 1918. The surveyors noted vegetation types, land use, sand bed, and islands on the contour maps they produced. The other active channel data is from aerial photographs taken in 1935, 1949, 1962, 1972, 1985, and 1992. The distortion of the photo mosaics varies, generally reducing with later dates.

When the measured subreach averaged active channel widths are plotted against the annual maximum mean daily peak discharge, a strong correlation is observed. The average of the previous five years annual maximum mean daily discharges of nearby gages has been identified, in an on-going study, as the strongest relationship between average width and discharge. This 5 year average discharge approximates the channel forming flow and will be referred to as the calculated channel forming flow. Good correspondence between the calculated channel forming flow and the average subreach width was found with the San Marcial discharge. Results deteriorated when shorter gage records available from gages closer to individual subreaches were used.

A detailed water budget for the Rio Grande on a daily basis has not been completed at this time. In general, the Rio Grande is a losing stream (discharge decreases as flow moves downstream) in the reach considered. To allow estimates of discharge at upstream gages based on discharge at San Marcial Bridge, linear regression discharge equations were developed to estimate losses between gages.

Nonlinear regression of power relationships between active channel width and calculated channel forming flow resulted in width equations in the form of $\mathbf{w = aQ^b}$ where:

w = reach average active channel width and
Q = calculated channel forming flow.

The robustness of the relationship $w = aQ^b$ (R^2 varied from 0.80 to 0.98) is striking when all the variability in the data is considered. The active channel widths are derived from data sets with differing degrees of accuracy and the channel has experienced changes in hydrology, the sediment-discharge relationship, mainstem dam construction, bank stabilization, and sediment management. The strength of the correlation is due in part to the averaging of the individual cross section active channel widths.

Modeling of Local Perspective: Cross section geometry data was available from San Acacia to EB 24. Initial analysis showed that the immediate effects of channel geometry changes anticipated for the different scenarios extended upstream to around RM 77. This location was defined as the division point for the two downstream most subreaches. Long term effects on channel width further upstream are also expected to result from the discharge limits. San Acacia to RM 77 (SA-RM77) was modeled with only width changes applied to 1992 geometry. Width and scour adjustments were made to SA-RM77 reach for each scenario based on the corresponding discharge limit. Scour was calibrated to match photographed top widths for a gauged flow of about 5000 cfs (USBR 2000). Depth of scour was found to be inversely proportional to active channel width, and so the amount of scour was not a constant within a subreach. Cross sections, within a subreach, were grouped based on width and slope trends and relationships between discharge, width, and scour were described for each cross section group. For each discharge limit, these same scour relationships were used to compute the depth of scour. Scour was assumed to occur uniformly in the main channel. The width of the main channel was increased or decreased by a percentage based on the change between the 1992 reach averaged width and the width calculated from the regression equations for each discharge limit. The geometric configuration of the main channel was preserved by applying the width adjustments to every pair of channel coordinate points (i.e., the channel was compressed or expanded). The valley width was not changed.

The cross section geometry used from RM 77 to EB 24 (RM77-RS) was developed based on assumptions related to each specific scenario as discussed herein. Width changes were applied after bed elevation changes. Subreach averages of channel top width, hydraulic depth, width/depth ratio, velocity, and inundated area, and inundated overbank area are reported for the modeled reaches.

Scenarios:

Future San Marcial Railroad Bridge Capacity: The San Marcial Railroad Bridge has been, and continues to be, the predominant flow limiting facility between Cochiti and Elephant Butte. In the mid-1980's Burlington Northern and Santa Fe (BNSF) railroad officials stated that there was roughly eight to ten feet of clearance beneath the railroad bridge and by 1989 the clearance had reduced to about two feet (USACE). Presently, there is about four to five feet of clearance which only permits a flow rate of roughly 3,800 cfs to pass beneath the low chord.Lower capacities at the Railroad Bridge translate to lower release flows from Cochiti Reservoir. Previous analysis has determined a no action minimum bridge capacity of roughly 2,000 cfs (USBR 2000).

The Railroad Bridge cross sections and the cross section immediately downstream of the Bridge had to be modified in order to model the minimum capacity. The cross section width was adjusted to reflect the future width based on a 2,000 cfs peak discharge. An iterative process of aggrading the bed elevation and then scouring the bed resulted in an elevation increase of about 0.25 feet prior to scouring.

Levee Breach: The occurrence of a levee breach has the potential to be quite devastating to both the structures and the environment in the area. The waters of the Rio Grande Floodway would pass through the breach seeking a lower elevation, typically along the invert of the LFCC. The elevation difference between theriver bed and the LFCC would result in a significant and abrupt change in slope causing a massive headcut. At the breach location, the bed could degrade as much as ten feet (USBR 2000). To the west of the levee, the breach would result in extensive local sedimentation and loss of effective drainage.

The levee analysis involved modeling two different breach locations. The first breach was located about four miles downstream of the San Marcial Railroad Bridge (EB 17) where the river bed is roughly 15 ft. higher than the rest of the valley. The second levee breach was modeled upstream of the railroad bridge near Tiffany Junction (SO 1641). The second breach was analyzed because of the close proximity of the Elmendorf drain outfall, the drainage for BDANWR. If a breach occurs at the Tiffany Junction location, the river could expel into the LFCC, filling the LFCC and causing the Elmendorf drain to become blocked.

Both of the hypothetical levee breaches were modeled at two discharge levels. An earlier study (USBR 2000) established the bridge capacity over time in the event of a levee breach using generated potential future hydrographs. The maximum bridge capacity of 5,600 cfs is the first discharge modeled and occurs about two years after the breach. The second discharge is the long-term capacity after the area below the breach has filled in with sediment. The discharge for the long-term capacity is 2,000 cfs and occurs roughly thirteen years after the breach event.

For the high bridge capacity case, the levee breach was modeled with a top width of roughly 150 ft and a depth of about 15 ft. The river cross section at the breach location was lowered by 10 ft and the resultant head cut was modeled by adjusting the bed elevations to reflect the equilibrium slope (0.0009 ft./ft.). When the equilibrium bed elevation exceeded the existing bed elevation, degradation was considered to have ceased and the upstream cross sections remained as the existing cross sections.

The model was split into two models to establish what percentage of the flows would pass through the breach and what percentage of the flows would pass down the existing channel. For the case of the maximum flow, an iterative process was conducted to match water surface elevations at EB 17 for both models. When no convergence was established, the assumption was made that all the water flowed out through the levee breach.

The second long-term bridge capacity discharge case was not as complex as the maximum discharge case, primarily because of several assumptions. The first assumption was that the scour hole filled in completely. The second assumption was that the flows at the breach location were split 50-50; fifty percent passed through the levee breach and the other fifty percent passed down the existing channel.

Sediment Plug: The Tiffany and San Marcial areas have, in about the past fifteen years, experienced the formation of three sediment plugs. Once a plug starts to form it can grow quite rapidly. With the initial dam of sediment, the water backs up behind the plug thus slowing the water velocities. The slower velocities allow the sand particles to deposit causing the sediment plug to lengthen in the upstream direction. The silt and clay particles are carried into the overbank areas where some are deposited and the rest return to the main channel downstream of the plug.

Flows up against the levee for prolonged periods of time result in a seepage condition and reduced capacity that the levee is unable to withstand. (Such was the case in 1991.) It is important to note that the reduction in levee capacity is not a temporary situation. With the re-establishment of a main channel and a channel system to Elephant Butte Reservoir, degradation may be experienced in the main channel. However, the overbank areas usually do not encounter degradation and therefore the reduction in overbank capacity is permanent.

The sediment plug modeled is based on Bureau of Reclamation records for a plug that occurred above the Railroad Bridge in 1995. The sediment plug was modeled in HEC-RAS by placing blocked obstructions in the main channel for each of the cross sections affected by the plug, thus forcing all of the water into the overbank areas. For the case of the sediment plug, the levee capacity was the discharge limitation rather than the Railroad Bridgecapcity. The maximum discharge, 1,400 cfs, was determined to be the discharge that would overtop the levee.

Temporary Channel: A final scenario was also examined, construction of a temporary channel from the end of the existing river channel to the reservoir pool. The temporary channel, with an average width of 250 ft. and an average depth of 2 ft., was added to the reservoir cross sections and then the degradation was incorporated into the upstream cross sections. The degradation was a pro-rated value based on the degradation quantities observed after the construction of the 1997 temporary channel. For the cross sections with unknown degradation amounts, an estimation was made based on an interpolation between known degradation values. The resultant flow rate passable by the Railroad Bridge was approximately 5,300 cfs.

RESULTS

The study presented here describes reach averaged behavior to discharge constraints and does not account for specific local issues involved on the Rio Grande or morphologic differences within a reach. Averaging values over long distances can overshadow the morphologic variation within a reach. Results are discussed separately for each modeled subreach. Degree of entrenchment and its effect on width changes is not specifically considered, but is lumped into the regression analysis of width and discharge. Entrenchment would make width change more difficult, particularly widening of the channel, and cause greater scour depths for a channel forming flow. The subreaches immediately below Cochiti Dam and San Acacia Dam show significant degrees of entrenchment since the 1970s. The widths generated at high discharge limits for these two reaches may be unduly wide. This in turn would affect the other hydraulic variables.

At the two lowest discharges, the narrowing generated by the width-discharge regression relationships is probably excessive based on trends noted for the hydraulic variables. For example, the calculated channel width, at the low discharge limits, were so narrow that very shallow flooding occurred on the overbanks for large areas. The historic minimum width for each subreach provides measured data on how the Rio Grande has responded to extended periods of low flow. Based on the information available, the true river response to the very low discharges would likely fall somewhere between the results of the regression equation and historic minimum change in width. Width change values between the historic minimum and the results of the regression equations were applied to the 1992 geometry and modeled with HEC-RAS to determine the width change that best reflected the use of the discharge limit as a bankfull flow. These are the widths reported in table 1.

The maximum discharge limits at San Marcial Bridge for the scenarios examined are:

1400 cfs - sediment plug in river upstream of bridge
2000 cfs - long term no action bridge capacity or capacity after breach fills with sediment
3800 cfs - current conditions
4200 cfs - 1992 calculated channel forming flow
5300 cfs - benefit with temporary channel in operation
5600 cfs – maximum capacity after a levee breach

For purposes of comparison, the regulated 2-year peak flow at San Marcial is 8,500 cfs and the regulated 5-year peak flow is 13,600 cfs. Applying the width regression equations to these discharge limits produced the average channel widths shown in table 1. Actual values for 1992 and historic minimum widths are also shown. Multiple sources of error are associated with these width calculations including error from discharge regressions and width regressions. These errors can be additive.

Table 1. Average Subreach Channel Width in feet by Discharge Limit.[1]

Subreach	1,400 cfs	2,000 cfs	3,800 cfs	1992 Measured	4,200 cfs	5,300 cfs	Historic Minimum
Cochiti to Bernalillo	275	300	425	300	450	550	300
Bernalillo to Isleta	525	550	650	600	675	825	600
Isleta to Mouth of Rio Puerco	450	475	575	550	625	725	500
Mouth of Rio Puerco to San Acacia	375	400	550	475	625	850	475
San Acacia to San Marcial	300	300	425	450	475	650	350
San Marcial to Reservoir (EB 24)	70	80	110	150	130	180	90

[1]Widths are reported to the nearest 25 feet except San Marcial to Reservoir values are rounded to the nearest 10 feet.

Four of the five annual peaks averaged to compute the 1992 calculated channel forming flow are thunderstorm related. In most cases, thunderstorm flows are not of long enough duration to be channel forming flows. The 1992 calculated channel forming flow is, therefore, likely higher than the actual channel forming flow. This would have two consequences: 1) the measured 1992 widths would be narrower than those generated from the width regression equations and 2) discharges generated using the discharge equations for the upstream reaches would exceed the 1992 calculated channel forming flows. As the generated width is directly dependent on discharge, generated widths for the 3,800 and 4,200 cfs limits are greater than the 1992 measured widths in the four upstream subreaches presented in table 2.

Table 2 reports the hydraulic variables computed for each of the discharge limits used in the corresponding modified geometry and for a 4,200 cfs flow in the 1992 geometry. The values in the ensuing tables are length-weighted averages for the existing river channel. In the case of the levee breach, the downstream hydraulic variables were zero or very small as a result of the river completely or partially passing through the breach. The result is low averages, especially for the upstream breach. The reach averaged widths shown in tables 1 and 2 do not directly correspond because the modeled reaches are divided differently from the width regression reaches downstream of San Acacia. In table 3 hydraulic properties are presented for a median historic winter flow of 500 cfs and in table 4 are those for a median historic summer flow of 200 cfs. These two tables show the hydraulic variables resulting from a common flow run through each discharge limit modified geometry.

Table 2. Hydraulic Variable of Discharge Limited Channels at Discharge Limit

Discharge Limit (cfs)	Channel Top Width (ft)	Channel Hydraulic Depth (ft)	Width/Depth Ratio	Channel Velocity (ft/s)	Channel Inundated Area (ac)	Inundated Overbank Area (ac)	Total Inundated Area (ac)
San Acacia to RM 77							
1400	325	2.1	150	3.1	1400	30	1430
2000	375	2.4	160	3.4	1650	50	1700
3800	525	2.9	180	3.7	2300	80	2380
1992 Geometry 4200	550	2.9	190	3.9	2450	60	2510
4200	575	2.9	200	3.8	2550	50	2600
5300	775	2.7	290	3.7	3450	30	3480
RM 77 to EB 24							
1400	100	3.9	20	3.5	220	1450	1670
2000	120	3.8	30	3.4	300	1490	1790
3800	140	5.1	30	4.4	350	1590	1940
5300	230	5.3	40	4.1	560	1660	2220
5600[1]	190	4.2	30	2.8	460	830	1290
2000[1]	120	3.5	40	3.6	290	1520	1810
5600[2]	60	1.5	10	0.9	140	60	200
2000[2]	120	2.9	40	3.1	280	60	340

[1]downstream levee breach; [2]upstream levee breach

Table 3. Hydraulic Variables of Discharge Limited Channels at Winter Median Low Discharge of 500 ft^3/s

Channel Geometry Discharge Limit	Channel Top Width (ft)	Channel Hydraulic Depth (ft)	Width/Depth Ratio	Channel Velocity (ft/s)	Channel Inundated Area (ac)	Inundated Overbank Area (ac)	Total Inundated Area (ac)
San Acacia to RM 77							
1400	175	1.4	120	2.5	800	0	800
2000	175	1.4	120	2.5	800	0	800
3800	225	1.2	190	2.4	950	0	950
1992 Geometry 4200	225	1.2	190	2.3	1000	0	1000
4200	250	1.2	210	2.3	1050	0	1050
5300	325	1.0	325	2.0	1400	0	1400
RM 77 to EB 24							
1400	90	2.1	40	2.0	210	0	210
2000	110	2.1	50	2.6	260	160	420
3800	120	2.2	60	2.5	300	30	330
5300	190	1.8	110	2.3	460	10	470
5600[1]	160	1.6	70	1.4	390	0	390
2000[1]	100	1.9	60	2.5	260	0	260
5600[2]	50	0.8	10	0.3	130	0	130
2000[2]	100	1.8	60	2.0	250	0	250

[1]downstream levee breach; [2]upstream levee breach

Table 4. Hydraulic Variables of Discharge Limited Channels at Summer Median Low Discharge of 200 ft^3/s

Channel Geometry Discharge Limit	Channel Top Width (ft)	Channel Hydraulic Depth (ft)	Width/Depth Ratio	Channel Velocity (ft/s)	Channel Inundated Area (ac)	Inundated Overbank Area (ac)	Total Inundated Area (ac)
San Acacia to RM 77							
1400	150	0.9	170	1.9	700	0	700
2000	175	0.9	190	1.9	700	0	700
3800	200	0.7	290	1.7	900	0	900
1992 Geometry 4200	225	0.7	320	1.7	1000	0	1000
4200	225	0.7	320	1.7	1000	0	1000
5300	325	0.6	540	1.5	1400	0	1400
RM 77 to EB 24							
1400	70	1.5	50	1.7	180	0	180
2000	90	1.5	60	2.3	230	0	230
3800	100	1.5	70	2.2	250	0	250
5300	160	1.3	130	1.9	390	0	390
5600[1]	130	1.2	80	1.1	320	0	320
2000[1]	100	1.5	70	2.1	230	0	230
5600[2]	40	0.2	40	0.4	100	0	100
2000[2]	80	1.2	70	2.1	200	0	200

[1]downstream levee breach; [2]upstream levee breach

San Acacia to river mile 77: In table 2 for the SA-RM77 subreach, channel width, channel inundated area, and width depth ratio all increase with increasing discharge limit. Wider channels are generally expected because the discharge limits are assumed to be the channel forming flow. Channel velocity and hydraulic depth results are also generally as expected, with slightly lower values for the low discharge limits and similar values at higher discharge limits. For some of the variables, the results for the 1992 geometry case are mildly inconsistent with the results from the modified geometries; these inconsistencies are within the precision of the study.

At the two median low flows, channel top width widens, channel velocity decreases, channel hydraulic depth decreases, and width depth ratio increases as the discharge limit at San Marcial Bridge increases. At the discharge limit flow, substantial increases in channel top width and total inundated area result from increasing discharge limit. All of these factors point to better habitat conditions for the Rio Grande silvery minnow at the higher discharge limits.

River mile 77 to the reservoir: Similar results were noted in the RM77 to EB 24 reach. At the discharge limits, the lower discharges resulted in lower hydraulic variables. There was little difference between the existing conditions and the higher discharges for the hydraulic depth and the velocity. For the winter and summer low flows all the scenarios experienced a drop in values, especially overbank inundation, which is to be expected.

In conclusion, the consequences of lower discharge limits at San Marcial bridge would be a generalized narrowing of the active river channel with increases in channel velocity and depth for all discharges in the new geometry. If the capacity at the bridge is increased, the above trends are reversed.

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CHANGES IN VEGETATION AND GEOMORPHIC PROCESSES IN CENTRAL NEVADA WATERSHEDS OVER THE PAST 5,000 YEARS: IMPLICATIONS FOR RIPARIAN ECOSYSTEMS

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The USDA Forest Service, Great Basin Ecosystem Management Project is examining the effects of climate change and human disturbance on vegetation and landform processes in central Nevada watersheds to understand current stream dynamics and to develop methods for maintaining and restoring riparian ecosystem integrity. The watersheds are in theToiyabe Mountains and are characterized by high-gradient, low-flow streams. Pack rat midden data, geomorphic and stratigraphic maps, stream gaging station data, and dendrochronology are being used to examine vegetation change, landform evolution, and stream dynamics. Vegetation patterns and watershed processes have tracked temperature changes and rainfall patterns. A dry and cool perod following the Neoglacial (1,300 and 2,000 YBP) exhibited low species numbers and, during its onset, significant hillslope erosion, side-valley alluvial fan building, and valley floors aggradation. More recent warmer and wetter periods have resulted in higher species numbers, a decline in hillslope erosion and sediment supply to the stream channel, and a tendency toward stream incision. Recent downcutting occurred after about 300 yrs ago and has been exacerbated by human activities. Currently, downcutting is related to major floods that move channel bed sediment and is controlling riparian ecosystem dynamics. This has important implications for riparian corridors and riparian ecosystem restoration.

IMPACT ASSESSMENT MODEL FOR CLEAR WATER FISHES EXPOSED TO CONDITIONS OF REDUCED VISUAL CLARITY

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Abstract

The new empirical model, described here, enables real-time assessment of impacts caused by excess water cloudiness as a function of (a) reduced visual clarity [cloudiness], and, (b) duration of exposure to cloudy conditions) in fisheries adapted to life in clear water ecosystems. This model, based on peer consultation and limited *meta* analysis of peer-reviewed reports does the following things: it (a) identifies the threshold of the onset of ill effects among clear water fishes, (b) postulates the rate at which serious ill effects are likely to escalate as a function of reduced visual clarity and persistence, (c) provides a context (the "visual clarity" matrix, with its cell coordinates and diagonal row labels) to share and compare information about impacts as a function of visual clarity "climate," (d) demonstrates changes in predator-prey interactions at exposures greater than, *and* less than, the threshold of direct ill effects, (e) identifies Black Disk sighting range [m] as a preferred monitoring variable, and (f) provides three additional optical quality variables -- Secchi Disk, Beam Attenuation, and turbidity -- which, suitably calibrated (as they have been in this study), expand the range of monitoring options in situations where Black Disk sighting equipment is not available or is impracticable to use. This new model demonstrates the efficacy of peer collaboration, and defines research horizons for refinement of the model.

INTRODUCTION

Although existing models for the assessment of impacts caused by excess suspended sediment provide useful guidance in settings where short time delays are not critically important, and where multiple modes of ill effect operate simultaneously (Newcombe and Jensen 1996, Newcombe and MacDonald 1991, Eifac 1965), field practitioners have identified the need for a model that would enable real-time assessment in settings where a single mode of impact -- reduced visual clarity -- is the predominant factor. Timeliness and expense of the data stream, and modes of impact, are introduced separately, below, to illustrate the benefits of a new type of model, and the limits of the available technology and data. These limits define the strategy for creating a useful solution.

With respect to timeliness and expense of data, existing sediment-"dose" models are optimal for controlled studies, for simulations, and for pollution events that are planned or unavoidable, or for chronic non-point pollution problems that recur on a predictable pattern or are linked to annual hydrological cycles. These are the kind of situations where an operational budget is in place, and where a real-time data stream is not critical to management decisions. Sediment-dose models depend on water quality data that are expensive and time-consuming to obtain. Suspended sediment samples must be sent to a lab for analysis; or a statistical correlation between water cloudiness (which can be measured in the field) and suspended sediment mass must be developed beforehand. In either case, reliance on suspended sediment data involves the delay and expense of lab services, or adequate lead-time for the development of a statistical correlation. And, with respect to multiple modes of ill effect, the existing (sediment-dose) models are optimized for sediment pollution episodes that generate multiple modes of ill effect. Most pollution events fall into this category in their initial phase, and close to the source of the impact.

Existing models succeed to the extent they do because the foundational database (Newcombe and Jensen 1996; and Newcombe and MacDonald 1991) integrates an entire suite of ill effects. Harmful impacts caused by excess suspended sediment are varied, complex, and often multi-modal. Thus, harm caused by one mode of ill effect -- loss of visual clarity for example -- could be masked by other unrelated effects. Excess suspended sediment can harm fisheries ecosystems in any of six different ways: (1) by acting directly on free-swimming fish; (2) by preventing the successful development of fish eggs and larvae; (3) by modifying the natural movements and migrations of fish; (4) by reducing the abundance of food available to the fish; (5) by altering habitat; and (6) by affecting the efficiency of methods for catching fish (Eifac 1965). Although loss of visual clarity is a common theme throughout this suite of harmful effects, it also can occur in isolation as a singular, persistent, harmful effect. None of the existing models contemplates this scenario. This persistence, coupled with the very small particle sizes involved, creates opportunities for the design and implementation of a new model.

Such a model would provide useful insights only to the extent that it is optimized for settings where impacts caused by coarser size-fractions (silts and sands) can be ruled out. Since this condition is frequently met during or after a sediment pollution episode, areas of potential application of an optical quality model are numerous and varied.

Available studies offer a glimpse of the possibilities for a visual clarity model, but there are relatively few that could contribute directly to its creation. Some published studies add to our understanding of the impacts of reduced visual clarity by revealing the correlation between it and reduced weight gain of lake-dwelling fishes; but the data in the classic study (Buck 1956) are reported as particle mass. These data would need to be converted to an optical quality metric before they could be used. Similar limitations in many of the more recent studies render them of limited use in the development of a new model.

Some studies identify changes in animal behavior linked to water cloudiness, but they tend to ignore other potential impacts in the fisheries ecosystem (Gregory 1993; Gregory, Servizi, and Martens 1993; Gregory and Northcote 1993). Other studies report the likelihood of harmful effects of sediment plumes (e.g., channel dredging of navigable-rivers) but, for logistical reasons, do not quantify the impacts in ways that would aid the development of a visual clarity model (Blouin, Kostich, Todd, and Savino 1998).

Because there are relatively few studies that make direct linkages between optical water quality and fisheries success, a database of optical quality impacts would be a challenge to create. Since the demand for a real-time model already exists today, other strategies ought to be developed to satisfy the need. The size of this data-gap and the effort required to close it dictate a novel solution.

A promising, alternative approach -- specifically, peer consultation and collaboration -- offers the fastest way to create a useable draft model for decision-making in the context of reduced optical water quality. A draft model based on the consensus of field practitioners, once developed to the *beta* stage, could be refined at leisure. Such a model offers near-term solutions, and long-term benefits to field practitioners.

This paper proposes a new model designed for impact assessment in settings where loss of visual clarity is the predominant mode of ill effect. Three design criteria guided the development process. First among these was that the model ought to encompass the probable range of pollution events of interest, and define this range as a function of (a) diminished optical water quality and (b) the persistence of potentially adverse (optical) conditions. Second was for the model to include various measures of water clarity, thereby to preserve or enhance the investigative options of field staff (whose equipment-on-hand might preclude collection of a preferred type of data). And a third consideration was that the model ought to facilitate discussion and ease the process by which new data might be incorporated. These criteria help to ensure that the new model builds on the strengths of the existing ones, while enhancing the timeliness of impact assessments of pollution events where a singular phenomenon -- reduced optical quality -- is the predominant mode of ill effect.

METHODS

The new model began as a prototype for data collection and discussion. Refinement of the prototype involved review of recent literature (a source of relevant data) and consultation with practitioners in the field of sediment impacts (a source of authoritative comment). Details of the developmental process follow, below.

The structure of the new model is patterned on that of existing models (Newcombe and Jensen 1996). Specifically, the new model is a matrix consisting of (a) a time scale that ranges from 0 hours to 48 months in 11 columns (10 exponentially-spaced intervals). Mid-points of these columns are 1, 3, and 7 hours, 1, 2, and 6 days, 2, and 7 weeks, and 4, 11, and 30 months; and (b) a water clarity scale based on nephelometric units (NTU). This scale is divided into 8 calibrated rows (1, 3, 7, 55, 148, 403, and 1097 NTU), and 7 rows where an intermediate calibration is implied but not stated. Columns and row values increase exponentially. This progression is useful because it compresses the arithmetic range of the two key variables -- duration (h) and optical water quality (initially, NTU) -- such that all potential pollution episodes of interest can be represented on a single sheet of paper. This satisfies one of the initial design criteria.

Compensation depth (after Lloyd and others 1987) was presented in the null hypothesis as an alternate calibration. The logic for including compensation depth as a proposed alternate calibration was that it ought to vary as an inverse function of increased water cloudiness: compensation depth should approach zero as water cloudiness approaches maximum values.

Cells in the matrix have letter (alpha) coordinates, and the diagonal rows tangent to the origin have number coordinates. Column labels start at the origin and run from "a" to "k" (lower case), and row labels, which also start at the origin, run from "A" to "O" (upper case). Column values (duration) increase by a factor of about 2.7183 ($e^1 = 2.718$) with each incremental step away from the origin. Row values (calibrated in NTU, and other variables added during the study) increase by a factor of about 1.6487 ($e^{0.5} = 1.6487$) which is half the exponential rate for columns: added rows double the number of rows, but halve the interval between them.

The family of diagonal rows tangent to the origin of the matrix represents bands of equivalent *impact*. This equivalency is such that brief exposure to conditions of severely reduced optical quality is regarded as comparable to long duration exposure to conditions of slightly reduced optical quality. These tangential diagonals represent an array of 24 exponential increments of impact. Diagonal rows have the symbol Δ (delta) and are numbered from "Δ_1" to "Δ_{25}" -- collectively, "Delta Values". These rows (and the clarity-duration combination each represents) are referred to as Delta-1 and Delta-25, respectively. Increments between adjacent rows (e.g., Δ_n and Δ_{n+1}) are a factor of approximately 2.24, [e.g., (e^1 x $e^{0.5}$) ÷ 2 = 2.24, where e^1 represents the increments between adjacent columns (h) and $e^{0.5}$ represents the progression between adjacent rows. The interval between adjacent rows is not affected, in this model, by the choice of variable (BD, SD, BA, and NTU) as each has been calibrated to correspond to the exponential progression ($\log_e$ NTU) established in the null hypothesis, and preserved in the emerging model].

Severity of Ill Effect Scores. The study began with a null hypothesis designed to elicit responses from experienced field researchers. In this hypothesis cells of the matrix were populated by postulated severity-of-ill-effect (SEV) scores not intended to represent true impacts. These scores, arranged in order from the origin of the matrix, were as follows: at Δ_1 (SEV = 1), Δ_2 (SEV = 2) … Δ_n (SEV = n) … and at Δ_{14} (SEV = 14). Fields represented by Δ_{15} through to Δ_{25} were left blank in the null hypothesis, though in later versions the symbol $ was introduced in some of these bands to indicate conditions believed to harm obligate clear-water fishes, or to degrade their ecosystem.

A useful visual clarity model must contain data only for fishes (or life stages) known to be intolerant of (or harmed by) reduced visual clarity. The notion of "obligate clear-water fishes" was introduced (Dr. Dave Rowe, personal communication), and the concept provided a useful screen of potential data sources. Data for silt-tolerant fishes would confound the visual clarity model for clear-water fishes and obligate clear-water fishes. So, the fledgling database contained five (potentially more than five) data categories: (a) obligate clear water fishes: one or more life stages are intolerant of cloudy conditions; (b) fishes are found in clear water systems; but are tolerant of (and perhaps benefit from) seasonal changes (increases) in water clarity; (c) fishes occupy clear water systems *and* cloudy water systems; they tolerate conditions of reduced visual clarity resulting from un-seasonal events or non-point source 'clay and colloidal suspended sediment' pollution; (d) fishes are found in cloudy-water systems but tolerate seasonal changes (decreases) in water clarity; (e) obligate cloudy-water fishes (or life-history phase); not found, or rarely found in clear water systems. These data classes served to screen information sources to isolate the data for clear-water fisheries.

The severity-of-ill-effect scale used in this study provides 15 intervals (from 0, nil effect; to 14, most severe effect) on the continuum of escalating harm (see Figure 1; see also, Newcombe and MacDonald 1991; and, Newcombe and Jensen 1996). Field practitioners suggested changes in the calibration of the null hypothesis (research-based severity-of-ill-effect, SEV, for any cell or group of cells), and they contributed information or data in support of their suggestions. Correspondence was carried out by e-mail, and an informal discussion group — the Turbidity - Visual Clarity Discussion Group, with an eventual membership of about 10 or 12 practitioners in various parts of the world — formed over the course of the model's development.

Predator-prey interactions were frequently raised in discussion and in published research results; and a system of notation had to be developed to track the reported patterns (see Key, Figure 1). Relevant data are linked to this notation by the alpha-coordinates of the cell most likely to represent the conditions (cloudiness, and duration) correlated with this datum.

RESULTS AND DISCUSSION

The Visual Clarity Model: The model that emerged from the Discussion Group represents a consensus of several researchers (Figure 1; see Acknowledgements). The position of the severity-of-ill-effect (SEV) scores in the null hypothesis was shifted to coincide with conditions known to affect the life of obligate clear-water fishes. This shift is based largely on suggestions by Drs. Jacques Boubée, and David Rowe, NIWA, NZ (personal communications) as follows: (a) the best available information suggests that harmful effects caused by cloudiness in the water do not occur near the origin of the matrix (lower left corner); and (b) the threshold of ill effects as a function of 'exposure' (cloudinesss and duration) is somewhere between the Δ_8 and Δ_9 levels. The new SEV pattern is as follows: Δ_1 to Δ_7 (SEV = 0), Δ_8 to Δ_9 (SEV 0 to 1), Δ_9 (SEV 1), Δ_{10} (SEV 2), Δ_{11} (SEV 3) … Δ_n (SEV n - 8) … Δ_{22} (SEV = 14).

Key Threshold: Delta-eight (Δ_8) is a key threshold in this matrix: It marks the boundary beyond which conditions of reduced optical water quality -- excess water cloudiness -- can be directly harmful to clear water fishes. This threshold is one of the most certain results of this study. Relative certainty (about the rate at which harmful conditions deteriorate in cloudy water at Delta Values equal to, or greater than, Δ_9) diminishes as a function of distance from the matrix origin: confidence is greatest at the origin (Δ_1) and least in the regions furthest from the origin (Δ_{25}). Impacts such as inhibition of upstream migration among sensitive fishes can occur at relatively low turbidity levels (e.g., *Galaxias fasciatus*, 20 NTU; Boubée and others 1997). This avoidance response could occur at turbidities as low as 15 NTU when the turbidity is caused by extremely fine particle sizes (Boubée, personal communication).

This observation -- impacts of excess turbidity on sensitive fishes -- supports the proposed position of the Delta-eight threshold. A momentary interruption of migration (on the order of 3 hours or so) might have no harmful effect, but a sustained interruption (on the order of a day) could put the fish at increased risk of harm (SEV = 1). And a prolonged interruption (on the order of multiple days, or weeks) would further increase the risk harmful impacts (SEV >>1). For a representation of the impact of 15 and 20 excess NTU in clear water (0 NTU) as a function of time, see cells F(c) to F(h), and G(b) to G(g), respectively, Figure 1.

Notwithstanding the high level of confidence about this threshold, the zone from Δ_1 to Δ_8 (inclusive) represents conditions that are potentially harmful to managed fisheries: predator prey interactions are known to be influenced at most (probably all) Delta Values equal to or greater than Δ_1. And, effects such as alarm reactions (which could affect a fish's diurnal patterns for part of a day, though perhaps for less than 6 hours) might also operate in the exposure range from Δ_1 to Δ_8. Opportunities to refine these findings are limited only by the rate at which new, suitable, data are created. Over time, data from a rigorous *meta* analysis of peer reviewed studies will be needed to improve the level of confidence in these draft (postulated) values.

Utility of Data Categories: Findings for some (NZ) fishes indicate that they are neither *clear water fishes*, nor *clear water fishes that are merely tolerant of seasonal episodes of reduced visual clarity* (Dave Rowe, personal communication). Rather, this kind of fish (*able to forage in complete darkness*) ought to be classed as "tolerant" of

Duration of exposure to conditions of reduced VISUAL CLARITY ($\log_e$ hours)

Visual clarity of water (BD), and comparable measures

Severity-of-ill-effect Scores (SEV) -- Potential

(Postulated Scores Underlined)

NTU	BA	SD	BD		0	1	2	3	4	5	6	7	8	9	10	$\log_e$ [NTU]	
	<c>	<c>	<b, c>	D_{15}	D_{16}	D_{17}	D_{18}	D_{19}	D_{20}	D_{21}	D_{22}	D_{23}	D_{24}	D_{25}			<a>
1100	500	0.01	**0.01**	D_{14}	7	8	9	10	11	12	13	14	$	$	D_{25}	7	*O*
				D_{13}	6	7	8	9	10	11	12	13	14	$	$		*N*
400	225	0.03	**0.02**	D_{12}	P5p	6	7	8	9	10	11	12	13	14	$	6	*M*
				D_{11}	4	5	6	7	8	9	10	11	12	13	D_{22}		*L*
150	100	0.07	**0.05**	D_{10}	3	P4p	P5p	6	7	8	9	10	11	12	13	5	*K*
				D_9	2	3	4	5	6	7	8	9	10	11	12		*J*
55	45	0.15	**0.11**	D_8	P1p	2	3	4	5	6	7	8	9	10	11	4	*I*
⇳ NTU +25 (Σ)				D_7	0	1	2	3	4	5	6	7	8	9	10		*H*
20	20	0.34	**0.24**	D_6	0	P0p	P1p	2	3	4	5	6	7	8	9	3	*G*
				D_5	0	0	0	1	2	3	4	5	6	7	D_{16}		*F*
7	9	0.77	**0.55**	D_4	0	P0p	0	0	1	2	3	4	5	6	7	2	*E*
⇳ NTU +5 (Λ)				D_3	0	P0p	P0p	0	0	1	2	3	4	5	6		*D*
3	4	1.53	**1.09**	D_2	0	P0p	0	0	0	0	1	2	3	4	0	1	*C*
				D_1	0	0	0	0	0	0	0	0	0	0	0		*B*
1	2	3.68	**2.63**		P0p	P0p	P0p	0	0	0	0	D_8	0	0	D_{11}	0	*A*
					1	**3**	**7**	**1**	**2**	**6**	**2**	**7**	**4**	**11**	**30**		
					Hours			**Days**			**Weeks**		**Months**				
					a	*b*	*c*	*d*	*e*	*f*	*g*	*h*	*i*	*j*	*k*		

Figure 1. Impact assessment model for clear water fishes exposed to conditions of reduced water clarity: a model to estimate severity of impact on rearing success of clear water fishes as a function of reduced visual clarity of water (m) and duration of exposure (h), for juvenile and adult life-history phases.

Key:

BD Black Disk sighting range, m. BD is inversely proportional to Beam Attenuation (BA).

NTU Nephelometric Turbidity Units: "cloudiness," approximate alternative to Beam Attenuation, BA.

BA Beam Attenuation: An inverse function of Black Disk sighting range (BD).

SD Secchi Disk sighting range, m. SD is roughly proportional to Black Disk sighting range, m. (zSD ≡ 1.4yBD; conversely, yBD ≡ 0.74zSD).

SEV Severity-of-ill-effect scale: indicates potential impact of reduced visual clarity. 0-3 low; 4-8 moderate, *sub*lethal; and, 9-14 lethal and *para*lethal. SEV 9 represents reduced growth rate; SEV 10, 0-20% mortality; SEV 11, >20-40% mortality; SEV 12, >40-60% mortality; SEV 13, >60-80% mortality; and SEV 14, >80-100% mortality.

[Δ_{1-25}] Delta Values: Incremental deterioration of rearing conditions as a function of diminishing visual clarity and increasing duration of exposure to conditions of reduced visual clarity; [Δ_n x 2.24 = Δ_{n+1}].

⇳Λ Alaska's water quality standard for lakes: increases not to exceed 5 NTU (Lloyd & others 1987).

⇳Σ Alaska's water quality standard for streams: increases not to exceed 25 NTU (Lloyd & others 1987).

n Underlined numbers in this matrix are postulated. They are for discussion only and are to be amended based on real data for severity of ill effect (SEV) of excess turbidity as relevant data or estimates become available. Ultimately, postulated scores should be replaced by pooled data and statistical correlations.

n Numbers in the matrix from which the underlining has been removed, have (a) been suggested by knowledgeable researchers, based on their experience; or (b) been suggested by data in peer-reviewed papers; or (c) both a, and b. See discussion below (Reference Co-ordinates). These numbers are open for further discussion as new information becomes available.

$ *Supra*lethal effects (greater than SEV = 14) -- potentially outside the intended scope of the model. This is a zone where serious habitat damage seems probable, and where aquatic community structure would likely be altered as a result. Such impacts would likely include a sustained loss of clear-water fisheries, and a sustained shift toward turbidity-tolerant fisheries communities.

<a> Aa … Jf … Pk -- Reference co-ordinates for data and personal communications.

<b> Conversion from turbidity to visual clarity (dimensionless) was done by Dr. Rob Davies-Colley, NIWA, NZ, personal communication, 2000 June 11.

<c>Power law assumed (e.g., $yBD = 2.64T^{-0.807}$ where T represents turbidity expressed as NTU). Dr. Rob Davies-Colley (NZ, NIWA), pers. comm.. 2000 June 11; Smith and others 1997; and, Smith and Davies-Colley. 2000 AWRA, in review.

Medium Grey. Areas in the matrix believed to be outside the expected data-envelope.

Light Grey. Areas outside the matrix used to indicate (a) Delta Values (for the array of diagonal rows tangent to the origin of the matrix), and (b) row and column co-ordinates.

P Represents "predator" in predator-prey interactions. Superscripted P (e.g., ^{P}) indicates that excess cloudiness is known (in one species at least) to offer a competitive advantage to a predator (P) at the expense of a prey item (fish or invertebrate). Increased growth rate of the predator is a qualified advantage. Subscripted P (e.g., $_{P}$) indicates that excess cloudiness is known (in one study at least) to create a competitive disadvantage for a predator (fish) which benefits a prey species (fish). Reduced growth rate of the predator is a qualified disadvantage, offset or partially offset by improved rate of survival of the prey species (fish).

π Represents "prey" in predator-prey interactions. Superscripted p (e.g., $^{\pi}$) indicates that excess cloudiness is known (in one species at least) to offer a competitive advantage to a prey species (fish) at the expense of a predator species (fish). Improved growth rate or improved rate of survival is a qualified advantage, offset or partially offset by reduced rate of survival of the predator species. Subscripted p (e.g., $_{\pi}$) indicates that excess turbidity is known (in one study at least) to create a competitive disadvantage for a prey species (fish) which operates to the benefit of a predator species (fish). Reduced rate of growth or survival of a prey species (fish) is a qualified disadvantage, offset or partially offset by improved rate of survival of the predator species (fish).

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reduced visual clarity, or "adapted" to cloudy water ecosystems. These are important distinctions in the context of an impact assessment model for clear-water fisheries. Such findings illustrate the need to classify "tolerance" data according to the fish's preferred life strategies. Data for *clear water fishes* have potential utility in the development of a visual clarity model where the issue is increased cloudiness. By contrast, data *for cloudy water fishes* might have utility in understanding the impacts of *decreased* water clarity. Pooling of disparate data types would mask these distinctions, and hinder the development of a useful model. This finding, though it is obvious upon reflection, is an important matter for the design of a suitable database.

Predator-prey Interactions: This study confirms that predator prey (P-π) interactions (a) occur at Delta Values not known to be directly harmful to obligate clear-water fishes, (b) occur at Delta Values believed harmful to clear-water fishes, and (c) are altered by amount of cloudiness in the water. Seven examples -- six from the literature, and one from discussion -- illustrate these patterns (see also Figure 1):

(1) Among the (NZ) fishes that demonstrated reductions in feeding rate at elevated turbidity levels, this pattern was the result of a reduced inability to feed, rather than to stress, as exposure to the highest turbidity level did not reduce feeding motivation or appetite. (Rowe and Dean 1998). "But, [*in the words of the author*] some fishes showed no change in feeding success at various turbidity levels, and are believed to have other sensory systems that guarantee success even in complete darkness" (Rowe, personal communication).

(1) *Razorback sucker larvae as prey*: Predator-prey (P-π) dynamics are reported for Δ_1 levels of 'excess' water cloudiness (Johnson and Hines 1999). In one of the three data points generated by their study, the predator gained a benefit from ambient water cloudiness -- cell A(a); and, in two other instances the prey gained the benefit -- see cell coordinates I(a), and M(a). Although larval razorback suckers (*Xyrauchen texanus*) demonstrate a strong preference for clear water and avoid turbid water when tested in a suspended-sediment gradient chamber, and although these larvae choose to live in the clear water stratum of an otherwise turbid littoral zone, they are highly vulnerable to predation (a) in a modified ecosystem where turbidity has been reduced, and (b) in an ecosystem (Lake Mohave, Arizona) where there are fish species -- native and introduced -- that eat fish larvae. In 10-minute trials at zero turbidity (0 ppm SS) green sunfish (introduced species) and Colorado squawfish (native) consumed all, or nearly all of the (razorback) larvae. Turbidity associated with sediment concentrations of 250 mg SS/L and 2000 mg SS/L reduced the rate of predation substantially. Turbidity values (expressed in nephelometric turbidity units, NTU) can be calculated from the equation NTU = 0.91 + 0.27*b*, where *b* is the suspended-solid concentration in mg SS/L (Johnson and Hines 1999). According to this correlation, 250 mg SS/L corresponds to 68 NTU, and 2000 mg SS/L corresponds to 541 NTU. In trials where the turbidity was 68 NTU razorback sucker survival averaged about 45 % (pooled data). And in trials where the turbidity was 541 NTU, larval survival averaged 43.5 % (pooled data).

(3) *Lake trout as predator*: See cell co-ordinates C(b) to E(b) in which predator - prey data from Vogel and Beauchamp (1999) are presented. A unified model of lake trout (*Salvelinus namaycush*) reaction distances as a function of light (0.17-261 lx) and turbidity (0.09, 3.18, and 7.4 NTU) pooled over three prey sizes (rainbow trout and cutthroat trout 55, 75, and 139 mm long) shows that reaction distances (at light levels greater than or equal to 25 lx) were about 95 mm in clear water (0.09 NTU) and less in turbid water. Reaction distance was reduced to about 60 mm in turbid water (3.18 NTU); and about 50 mm at 7.40 NTU. Prey size was not a significant variable.

(4) *Brook trout as predator*: See cell co-ordinates H(f) in which predator - prey data for brook trout (*Salvelinus fontinalis*) from Sweka (1999) are presented. The rationale for the predator-prey dynamics is presented below, as is the deduced SEV value (SEV = 5) for this cell. The short-term reduction in growth rate reported in Sweka's study is roughly equivalent to other effects such as (a) short-term reduction in feeding rate or (b) short term reduction in feeding success. Effects of this kind, which take place on a relatively brief time-scale, would likely have been assigned a SEV score of 4, on a 15-step scale of ill effects (where 0 represents nil, and 15 represents 100 % mortality) resulting from exposure to *suspended sediment* (see Table 1, in Newcombe and Jensen 1996). This assignment of impact is potentially consistent with the report that the brook trout (*Salvelinus fontinalis*) were able to maintain a relatively constant rate of food intake in turbid water. But, the change in feeding strategy from "ambush" to foraging -- necessitated by reduced visual clarity of the water -- required increased expenditure of energy. This increased expenditure explains the significantly (>50%) reduced growth rate of these brook trout in cloudy water. Notwithstanding the reduction in growth rate caused by foraging, the alternative strategy -- continued reliance on the clear water ambush strategy -- would likely have resulted in a less favorable outcome for the brook trout, up to and including weight loss. Cell H[f] represents the conditions of reduced visual clarity and duration of exposure in Sweka's study. These results help to define an impact level for this cell. Although a *short-term* reduction (< 2 weeks) in growth rate might be assigned an SEV score of 4, a significant (short-term) reduction (50% or greater, as found in Sweka's study) would be better represented by an SEV score greater than 4, perhaps on the order of SEV = 5. Hence, in cell H(f) the SEV score suggested by this "data-triplet" (cloudiness, duration and ill effect) is SEV = 5; see Figure 1. Other studies will be needed before a statistical correlation can be developed for any of the cells in the matrix, but this datum represents a first step in the process. (In studies that last two weeks or more, the sustained reduction in growth rate would be regarded as a *para*lethal effect -- one that jeopardizes the successful completion of the life-history phase -- and be assigned an SEV of 9 units.)

(1) *Mudfish*. See Diagonal row Δ_{16}; and cell G(j). Mean turbidity was 11.5 nephelometric turbidity units (NTU) at sites with mudfish, but 21.3 NTU at sites without mudfish (*Neochanna diversus*, NZ, Hicks and Barrier 1996). The authors do not link these observations to any particular life-stage (e.g., larva) or activity. However, they seem to indicate thatmudfish, in spite of the evocative name, are adapted to life in a *relatively* clear-water ecosystem. Duration of exposure to conditions of reduced visual clarity is not specified, nor is it known for purposes of this study whethermudfish qualify as *obligate clear water fishes*, or *clear water fishes that tolerate seasonal episodes of reduced water clarity*. More likely, mudfish (in spite of any life strategy to which, perhaps, it owes its name) are moderately tolerant of chronic, low levels of reduced visual clarity. Tentatively, therefore, the data (averages of multiple observations) are presumed to (a) integrate effects of moderate increases in cloudiness of moderately clear-water ecosystems, and (b) reflect the annual visual clarity "climate." To the extent this supposition is true, these observations tend to confirm two patterns in the matrix: (a) the relatively high severity-of-ill-effect score (SEV = 8) for cellG(j) in diagonal row Δ_{16} , and (b) the relatively lower SEV scores at lesser Delta Values. Other explanations are possible, especially if brief exposure to excess cloudiness could be harmful to a sensitive (e.g., larval) life stage. In this alternate scenario, emphasis would shift diagonally along Row Δ_{16} from cell G(j) in the direction of cell O(a). But it is not possible to know from the available information where, precisely, on this continuum, the ill effect occurs. Indeed, harm to a species could occur anywhere in the "exposure" continuum. And, if the harmful effect has a measurable endpoint (that is, if it reduces the success of one or more life-stages, or activities) the harm can be quantified and mapped on the matrix. These challenges define future research goals. Notwithstanding future insights, this current research on mudfish puts the SEV value somewhere between 8 and 14 when turbidity =21.3 NTU. The matrix is calibrated to indicate severe *sub*lethal effects. Quite possibly a sediment climate that does not cause direct mortality, if repeated on an annual cycle over a sufficiently long period of time (years, decades or centuries), might be enough to produce this effect (absence of mudfish). For this reason, and in the absence of more compelling data to suggest greater levels of harm (e.g., SEV >8), the cell values should remain as postulated (that is, at Δ_{16} SEV is 8). A large database capable of revealing statistical correlations would, however, be an improvement over this single-study datum

(6) Bluegill. See Cell I(a). Clay turbidity (60, 120 and 190 NTU) reduced the feeding rate of Bluegill (*Lepomis macrochirus*), but not size selectivity, in feeding trials that lasted three minutes (Gardner 1981). These conditions -- relatively low turbidity, extremely short intervals -- are consistent with a low severity of impact. This suggests an SEV for cellI(a) of 1.

(7) Juvenile Pacific Salmon (Oncorhynchys spp.). See cells A(b,c) to E(b,c), clear water; and G(b,c) tp K(b,c), turbid water. Piscivorous fishes in the moderately turbid Fraser River (27-108 NTU) were less successful at capturing prey than piscivorous fishes in the relatively clear waters of near-by Harrison River and a clear-water side channel (1-6 NTU). Only 10% of theFraser R. piscivores (turbid water) had recently eaten fish prey, whereas about 30% of Harrison R. piscivores (clear water) had recently eaten fish prey. Loss of tethered prey was greatest at dusk (estimated here to be about 3 hours duration, or 6 hours duration over two successive days), and was greatest in the clear-water setting (Gregory andLevings 1998). The authors conclude that naturally elevated turbidity of the lower Fraser River protects Age-0 juvenile salmon.

Cell Notation for Predator-prey Dynamics: Differing patterns in multiple studies can not be represented in a single cell. Where two studies overlap (as is the case for Gregory and Levings 1998; and Vogel and Beauchamp 1999), the improvised conventions for notation in this study required revision. Text has been used to state the complete range of rows and columns to which an observation applies, and cell notation (Figure 1) is only a guide, with overlapping ranges not shown.

Black Disk Sighting Range: Black Disk sighting range (m) is a preferred metric for water cloudiness. Turbidity -- is not the most robust of the available techniques (Rob Davies-Colley, NIWA, NZ, personal communication, 2000). But it and other variables such as Secchi Disk, and Beam Attenuation emerged as acceptable, if flawed, alternatives (Rob Davies-Colley, personal communication, citing Kirk 1988).

Compensation depth (an alternate calibration for water cloudiness in the null hypothesis) was rejected because it is too easily affected by conditions not related to the amount of excess cloudiness caused by fine mineral solids in suspension (Rob Davies-Colley, personal communication; Davies-Colley and Vant 1988). Two arguments prevailed: (a) compensation depth depends (more strongly) on light absorption, which is not detected in turbidity measurement, and (b) in some humic-stained clear-water systems the expected patterns of light penetration are altered in unusual ways by turbidity (Davies-Colley and others 1992; and Davies-Colley, personal communication in reference to turbidity from gold-mining on NZ's West Coast, on streams where the water is stained by humic compounds).

CONCLUSION

A new model for assessment of visual clarity impacts among clear-water fisheries has emerged as a*beta* version, suitable for (a) determining the impact status of a sediment pollution event in relation to the threshold of ill effects Delta-eight (Δ_8), and (b) further testing and refinement of postulated scores, (aided by data, yet to be published, from peer reviewed sources). Future research, whether it is directed at improving this model or not, should contemplate the utility of the various measures of water cloudiness (Black Disk, Secchi Disk, Beam Attenuation, and turbidity), and the potential utility of additional data such as sediment particle size, and the concentration of suspended sediment (mg SS/L).

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RAPID ASSESSMENT PROCEDURE FOR AQUATIC HABITAT, RIPARIAN & STREAMBANKS (RAPFAHRS)

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INTRODUCTION

Since July 1998 periodic stream assessment work along private land has been ongoing in Walla Walla and Columbia Counties, Washington. This work has been done for the Walla Walla County and Columbia Conservation Districts. These conservation districts want to determine total watershed health, stream corridor condition, and the proper context for districts, in their partnerships with the Natural Resources Conservation Service and the Washington Department of Fish and Wildlife, to install projects that lead to stream restoration. The procedure developed to accomplish the stream assessment work has been named the Rapid Assessment Procedure For Aquatic Habitat, Riparian, and Streambanks (RAPFAHRS)

DATA COLLECTION

The field assessment work along streams is generally a continuous recording of data that includes: (1) reach designation, (2) riparian reach length and rating, (3) large woody debris or overhead cover in percent of pool surface, (4) pool substrate, (5) pool depth, (6) pool quality rating index (PQI), (7) stream bank erosion reach length, height, and rating, (8) stream percent slope, (9) stream d50 and bimodal condition if present, (10) Rosgen Stream Classification (Rosgen, 1996), (11) fencing, (12) stream bank geomorphic surface type (i.e. floodplain, terrace, steep hillside, side slope, etc.), (13) locations of water diversions and returns, and (14) potential conceptual treatments, and aquatic winter rearing habitat. Periodic measurements or recordings were also made for: (1) water temperature and diurnal variation during assessment, (2) riparian type (trees, brush or grass), (3) Global Positioning (GPS), (4) streambed d100, (5) stream bank stratigraphy, (6) d50 and d100 of streambank gravels, cobbles, and boulders, (7) percentage of substrate covered with algae, (8) livestock presence in the river, (9) Montgomery and Buffington Stream Classification (Montgomery and Buffington, 1997), (10) Stages of channel evolution (CEM) according to Schumn, Harvey, and Watson, and (11) sinuosity. Frequently the reach length, specific erosion, or habitat conditions were photographed. In general these photographs were taken in a downstream direction. Reaches were usually evaluated from upstream to downstream with right and left banks determined looking downstream. Whenever possible 1998 aerial photographs with a scale of about one-inch equals 500-ft. (1:6,000) were used for designating the reach breaks. New reaches were established based on changes in riparian vegetation, erosion, large woody debris, stream slope, width, substrate, or stream type. Access to stream reaches was provided by notification to landowners by the two conservation districts. The procedure used both written notification and follow-up phone calls. Denied access only occurred in about 5 to 10% of the reaches. Data was collected during low flow conditions in the late summer and early fall whenever possible. However, do to funding constraints, some of the data had to be collected during winter and spring flow conditions up to water depths of half bankfull condition. Follow-up visits were made to get low flow pool conditions. Measurements were made from the reach photos and from older photos and maps to determine stream sinuosity.

EVALUATION

The data was summarized for the districts in spreadsheet format. Graphical presentations and queries were also provided. Pool quality, riparian buffer quality, and erosion severity indexes were developed to provide visual aids to help identify priority areas for treatment and differences in stream reach watershed health. Criteria to establish a pool quality rating for a given pool are shown in Table 1. The Pool Quality Index (PQI) is modified from WRD, Idaho (1993). To form the index, the sum of all pool quality ratings in a reach were subtracted from 10. Therefore, the lowest rating of 0 or lower was considered to have the highest pool quality. A reach pool rating of 10 had no pools.

Criteria to establish a riparian quality rating are shown in Table 2. An area with a healthy riparian buffer received a rating of 1. Areas with little or no vegetation received a rating of 10. An erosion severity index was created in a similar fashion and is shown in Table 3. A rating of 0 depicts a reach with no erosion. A rating of 10 depicts a highly eroded reach.

Most potential erosion problems are a result of a combination of factors of the type shown in Table 3. A common condition encountered in the field was modified channels with push-up gravel and cobble side slopes. These were rated 6 or 7 depending on side slopes. It is assumed at bankfull and higher flows that the pushed-up or stacked stream cobble and gravel will slough. This is common for this type of stream material when used as a “sugar dike” (one that essentially melts away in high flows).

Table 1. **Pool Quality Rating Index**

1. Depth		
	<0.5 feet	= 0
	between 0.5 and 1.5 feet	= 1
	>1.5 feet	= 2
2. Substrate		
	gravel (<2.5 inches)	= 0
	cobble (2.5-10 inches)	= 1
	boulder (>10 inches)	= 2
3. Overhead cover		
	<10% of the pool surface	= 0
	10-25% surface area	= 1
	>25% surface area	= 2
4. Submerged cover: large organic debris, small woody debris, and other forms below or on the water surface.		
	<10% of the pool surface	= 0
	10-25% surface area	= 1
	>25% surface area	= 2

Table 2. **Riparian Rating Index**

Needs Trees		Needs enhancement		Good
10 9	8 7	6 5 4	3	2 1
-Farmed to edge	-Sparse buffer	-Narrow buffer with	-Trees in buffer	-Many lrg. tr.
-Overgrazed	-weedy	minimal older trees	-some shade	-good shade
-No strm. shade	-livestock dmg.	that provide shade	-No livestock	-veg. healthy
	-human disturb.			

Table 3. **Erosion Rating Index**

The number of items present that are listed below the rating are used to establish the rating.

10 (3 or more) 9 (at least 2) 8 (at least 1)
Unvegetated, with high stream bank overhang angle.
Unvegetated with high uncemented sandy stream bank.
Unvegetated with a stratigraphy of fines and sands over uncemented gravels and cobbles that occur within bankfull flow condition.
Unvegetated with uncemented gravels and cobbles that occur within bankfull flow condition.
High depth (> 3 ft.) of washed root zone.

7 (3 or more) 6 (at least 2) 5 (at least1)
Unvegetated with moderate stream bank height (1/3 of stream banks still above bankfull) with vertical to 1:1 sloped stream banks.
Unvegetated moderate height of stream bank with uncemented sands.
Unvegetated stratigraphy of fine sand over uncemented gravel or cobble, and contact is above bankfull depth (i. e. terrace).
Low percentage (<25%) of stream bank with roots.
Uncemented pushed-up gravels and cobbles against stream bank or as a "sugar dike".
Moderate depth (2ft. to 3ft.) of washed root zone.
Cultural evidence of erosion such as stream undercut fences, pipes, buildings, and roads.

4 (4 or more) 3 (at least 3) 2 (at least 2) 1 (at least 1) 0 (no items).
Low (1ft. to 2ft.) of washed root zone.
Low percentage (<25%) of stream bank with roots.
Evidence of recent stream bank sloughing.
Unvegetated stream bank with vertical to flatter slope, unless bedrock.
Unvegetated, uncompacted (i.e. loose) sands.
Unvegetated, uncompacted gravels or cobbles.
Unvegetated, uncompacted stratigraphy of fines over sands or gravel.

Unerosive, very stable stream banks, such as those along bedrock or ones with a good cover of grasses, shrubs, or trees, or those with a high percentage (>50%) of stream bank with roots would rate a zero. In addition stream banks with compacted fines, sands, or gravels that eroded at slow rates were rated between 1 and 3 depending on location (i.e. outside curve position rated higher). Previously treated areas with tree revetments (TrR) or rootwads and boulders (R&B) frequently still showed some low rate of erosion so they may have been rated to show erosion. The reason for this was that there may not have been any associated soil bioengineering treatment, or the treatment was not successful.

A list of potential conceptual treatments used in the assessment is shown in Table 4. The terms Vanes and Barbs are shown to be used interchangeably. However vanes are typically built with uniform rock; barbs are typically built with graded rock sizes.

In general no rock or rootwad treatment was shown for erosion rates of 4 or less. In these reaches only soil bioengineering treatment such as staking were recommended. The exception was when channel reconstruction (CR) or rock weirs (RW) are shown to narrow the channel width and to deepen the thalweg.

All rock structures such as rock weirs (RW) and vanes (V) should be assumed to be installed in Walla Walla and Columbia counties with associated rootwads unless there are special circumstances like a bridge constriction that would preclude their desirability. In addition, unless there are special circumstances, all structures are assumed to be installed with associated soil bioengineering treatment to restore the riparian area, as well as fencing and grazing management.

The final determination of conceptual treatment should be based on a follow-up field visit, discussion with the landowner, and NRCS Standards. The follow-up field visit will place the conceptual treatments into the specific context of bankfull depth, width, and slope. In addition the field visit will establish the associated treatments that may be located across the river or in another reach. For example, a winter rearing channel in one reach may need to be tied in with several R&B (rootwad and boulders) at an upstream location to keep flood flows and sediment from entering the winter rearing channels at the upstream end. A field visit is also essential to determine if there is sufficient channel width or the right stream bank height to install TrR (tree revetment of rootwads with boles into stream bank), verses R&B (rootwad and boulders parallel to stream bank) or OPL (overlapping, parallel logs with rootwads).

Table 4 **Conceptual Treatments**

Treatment	Abv.	Gen. Site Condition & Special Treatment
Staking	S	Floodplain (F. P.) & low terrace (T) scarps and top.
Staking and Geotextile	S&G	Same except add geotextile when gravel side slope.
Dormant Post Planting	DPP	Low and high F. P. and low T scarp and top.
Whole Plant Transplant	WPT	In areas behind TrR, R&B, OPL, J, V, & Other.
Joint Planting	JP	In existing riprap.
Facine	F	Above bankfull condit. on F. P. and T. scarps.
Facine & Geotextile	F&G	Same except add geotext. for gravel, cobble scarp.
Fence	Fn.	Along all stream banks with stock access.
Vegetated Geogrid	VG	Shaped F. P. and T. slopes and repairs.
Live Cribwall	LC	Base of F. P. and low terrace side slopes.
Tree Revetment	TrR	Base of F. P. or T side slopes (boles into bank).
Rootwad and Bldrs.	R&B	Base of F. P. or terrace scarps (parallel bank).
Overlap. Prl. Logs	OPL	Base of F. P. or terrace scarps slopes.
Instream R&B	IRB	Within channel rootwads and boulders primarily in C's, but also in B's and F's stream types.
Bank Shaping and	BS&	F. P. and some terrace side slopes, especially

Table 4 (cont'd)

Vegetation	V	low terrace scarps.
Rootwad Vanes/Barbs	RV	Rootwad installed as a vane at base of slope.
Vane/Barb	V	Rock vanes should be installed rather than barbs (up to bankfull and pointed upstream).
J Vanes (curved tip)	J	Base of F. P. and T. especially with stable far bank.
Vanes/Barbs With Rootwads	VR	Vanes/barbs with rootwads in pool.
Channel Reconstruction	CR	Blown out C, E., and D Stream Types.
Log Cover Structure	LCS	F. P and T. damaged and reshaped slopes.
Lunker Structures	LS	Base of F. P. or T for fish hab., and bank protect.
Rock Vortex Weirs	RVW	Across channel for aquatic hab. and to narrow.
Rock Vortex Weirs (with Rootwads)	RVW /R	Same with instream rootwads for aquatic habitat.
Rock Weirs	RW	Across channel to narrow, create pools for fish passage and to direct flow direction.
Rock Weirs and R	RWR	Same & inchannel rootwad, to create habitat
W Rock Weir	WRW	To direct flow into two channels.
Toe Rock	TR	Base of F. P. or T.
Blders. & Clust.	B & BC	In some B 2,3, & 4; F 2,3, & 4; and C 2,3, & 4 To create pools.
Boulder Clusters	BC	Same
Single or Double Wing Deflectors	SW/ DW	Base of F. P. or T slope.
Cabled Cross Logs	CCL	Cabled logs to streambed to trap sediment.
Saw-toothed Gabon	STG	Bank protection and edge complexity.

FIELD APPLICATION OF RAPFAHRS

This version of RAPFAHRS has been used for field assessment on about 90 miles of channel or 180 miles of stream banks in the Walla Walla River Basin in Washington. An analysis of factors limiting the abundance and distribution of salmonids within the basin is underway. The RAPFAHRS stream assessment data has been very beneficial to the limiting factors analysis. The amount of large woody debris, pool frequency, and pool quality have been shown in the RAPFAHRS assessment to generally be poor throughout the basin. Also within the basin there are extensive reaches which are at or shallow to bedrock. These areas have minimal spawning habitat as well as minimal pools for rearing habitat. Stream bank stability (erosion) and riparian zone quality are generally fair, however, there are many miles of stream corridors that are overgrazed or farmed very close to the stream bank. Locally their are extensive reaches of degraded C4 and D4 stream types. These areas have very wide bankfull width to depth ratios, are braided, have only a few inches of water at low flow conditions, and have poor shade conditions. The assessment sinuosity data shows extensive reduction in the meandering of the streams in the Walla Walla Basin. in historic time.

Some obvious problems with procedure measurements have been noted. For example, reach and erosion lengths may be underestimated because distance is measured with a Laser Distance Measurer, which may not always get an accurate measurement of distance around the stream curves. In addition, narrow stream widths (<51 ft.) are sometimes estimated because the lower limit for the laser is 17 yards. (i.e. 51 ft.). However, widths under 20 ft. were usually measured with an extended survey rod. Bankfull depth was determined with a survey rod. Adequate pool length, width, and depth can only be determined at low flow so a follow up visit is needed at selected sites to correct preliminary estimates. Temperature measurements only reflect the narrow time widow that the assessment team was in the river. The d50 and d100 particle size was occasionally measured with a tape, but data for most of the reaches are based on visual estimates because of the size consistency of particle sizes along most streams. In addition the difference between, for example, C3 and C4 is not relevant for most of the potential conceptual treatments. However, there are some stream reaches in the basin that appear to reflect a bimodal distribution of the gravel and cobble sizes. The coarser sizes in these areas were selectively measured.

The speed for doing the assessment varies by stream type, access, extent of fencing, amount of LWD in the river and on the stream banks, thickness of riparian vegetation, the number of times the stream is crossed, water depth, and landowner interest in discussion. Some of the LWD, water depth, and riparian vegetation delays are minimized by doing the work in chest waders, so the stream can be crossed to avoid difficult areas. For the overall inventory, the slowest inventory was about 0.75 mile in a day and the fastest about 3.75 miles per day. The average was about 2 miles per day by the two-person team. The data is presently being analyzed for trends of the quality of aquatic and riparian area and stream bank erosion needs.

CONCEPTUAL DRAWINGS AND INSTALLATIONS

Some conceptual drawings have been prepared after consultation with landowners. These are for treatments that benefit aquatic habitat and reduce land loss due to stream bank erosion. This results in a win-win situation for aquatic habitat and landowners. Conceptual drawings have so far been prepared for about 46 locations, most of which are along Coppei Cr. in Walla Walla Co., which was inventoried using a different procedure. Aquatic habitat and related stream bank protection work was installed on about a dozen site reaches in the late fall of 1998 along Coppei Cr. Redd surveys along Coppei Cr., in the spring of 1999, have found 47 steelhead redds* around the project work.

* Personal communication with Mike Pelissier, WWCD, Walla Walla, WA, June, 1999

ASSESSMENT USE

The assessment reflects that there are many miles of riparian area that need vegetative treatment and fencing in the Walla Walla Basin. The Walla Walla County Conservation District and Columbia Conservation District have already installed several miles of soil bioengineering treatment and fencing. The re-establishment of riparian corridors in the Walla Walla Basin will help to re-establish connectivity of the wildlife habitat.

The assessment is also being used to discuss with the landowners the various aquatic habitat, riparian area, and stream bank erosion problems that may exist along their specific stream reaches. The landowners decide if they want to participate in the restoration work. The districts, in consultation with their partners the Natural Resources Conservation Service (NRCS) and the Washington Department of Fish and Wildlife (WDFW), decide which projects have priority for conceptual design, analysis, permit, and treatment. The assessment provides some of the range of treatment strategies for a given reach. Not all treatments are applicable for every site and depend to a large degree on stream type and stream width considerations. At the conceptual treatment stage of project work, the range of treatments in the assessment may be expanded or reduced based on landowner, NRCS, or WDFW interest. Species listed as threatened under the Endangered Species Act are found within much of the Walla Walla Basin. Biological assessments are often needed prior to granting of permits. Both the stream assessment and conceptual drawings are being used as part of the biological assessment process. The stream assessment also gives a perspective of the watershed health of the various sub-watersheds, as well as the whole Walla Walla Basin.

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Riparian Capability for the Carson River

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Abstract: The capability of a riparian area is defined as the highest ecological status an area can attain given its political, social, or economic constraints. Defining this becomes essential for valleys where people crave the dynamic equilibrium of riparian proper functioning condition but cannot accommodate the great widths of the original natural floodplain nor remove other limiting factors such as water rights or diversion dams for consumptive use. After incision or channelization, narrow interests might strive for regaining stability through bank protecting revetment or alternatively, a hands-off approach. Neither achieves the optimum benefits of combining the riparian imperatives for proper functioning condition with the freedom to choose desired future communities. Defining riparian capability considers the physical drivers of the setting, hydrology, vegetation, and landform while evaluating trade offs or natural tensions between potential and social, economic or political constraints. The Carson River Coalition is actively confronting the natural tensions inherent in river and land use management as it charts a course based on eleven guiding principles.

INTRODUCTION

The Carson River has its headwaters in the Sierra, at Kit Carson Pass and from 11,000 foot peaks in California south of Lake Tahoe. Its two main forks (East and West) flow north and east across the Nevada state line and into the Great Basin, which has no drainage outlet to the ocean. During the California gold rush, the Carson River Watershed was traversed by "49ers" on the way to the "mother lode" on the west slope of the Sierra. The first settlement in Nevada, Mormon Station (now Genoa), was founded in 1851 next to the Carson River, as a way station for the 49ers. It wasn't until the great Comstock Lode was discovered in Virginia City, Nevada in 1856 that settlement in the Carson River watershed boomed. By the 1860s, the forests in the headwaters were being extensively logged, with the logs being rafted down the River. The floor of the Carson Valley was converted from naturally wet meadows to irrigated agricultural fields.

The Carson Valley, just south of the State Capital, Carson City, is still mostly agricultural, with its green pastures and alfalfa fields irrigated by ditches from diversion dams on the river channel. The valley is an attractive destination for thousands of retirees and new families, who have been moving there in increasing numbers since the 1970s. With its current population boom and its sprawling urbanization, the Carson Valley is like many other western communities, coming to terms with change. On New Years day of 1997, a huge rain-on-snow event in the Sierra caused a 75 to 100-year flood on the Carson River and other nearby watersheds. The flood did millions of dollars in damage to both urban and rural properties and infrastructure. This "wake-up call" caused many to pay attention to the river and its watershed.

Integrated Watershed Management: After the flood, a coalition of citizens, educators, agencies, and conservation districts formed to hold the first Carson River Watershed Conference. Over 250 people attended the two day conference, that stressed the importance of integrated watershed management, the process of addressing issues of water quality, quantity, flooding and habitat in a coordinated, holistic manner.

The Carson River Coalition (CRC), sponsored by a regional water subconservancy district, formed to build cooperative and collaborative relationships between diverse interest groups and to collaborate for watershed, river, and floodplain resource management. Within a year, the group hired a watershed coordinator and drafted a list of eleven "guiding principles" for integrated watershed management. (Table 1). The guiding principles were presented to numerous civic groups and elected bodies. Several local governments and agencies adopted them.

Table 1. Carson River Integrated Watershed Planning Process – Guiding Principles

1. Manage the water resources for economic sustainability, quality of life, and protection of private and public property rights.
2. Acknowledge and respect the watershed’ s natural processes in land use decisions.
3. Maintain or improve the quality of the water to support a variety of beneficial uses.
4. Protect the headwaters region as the system’ s principal water source.
5. Recognize and respect the interests of all stakeholders upstream and downstream by fostering collaborative and mutually respectful relationships.
6. Maintain the riverine and alluvial fan floodplains of the Carson River Watershed to accommodate flood events.
7. Protect and manage uplands, mountain ranges, wetlands, and riparian areas to enhance the quality of surface flow, groundwater recharge, and wildlife habitat.
8. Promote conservation of water from all sectors of the community’ s water users for the benefit of municipal, industrial, agricultural, domestic, recreational, and natural resources.
9. Encourage management of growth that considers water quality and quantity, open space preservation, and maintenance of agriculture in floodplains.
10. Protect and support opportunities for public recreational access to natural areas throughout the watershed—including the river corridor—where appropriate.
11. Promote understanding and awareness of watershed resources and issues through cooperative education efforts throughout the watershed.

Now, the CRC faces the next challenge--bringing these eleven principles to life through implementation. This will be difficult, because the principles encompass a variety of thorny issues, including water quality and quantity, floodplain management, habitat protection, open space preservation, private and public property rights and the need to respect the watershed's natural processes in long term land use decisions.

PUTTING THE GUIDING PRINCIPLES TO WORK

In a strategic planning session in May 2000, the CRC saw the need to begin making their "vision" for the river corridor and its tributaries more detailed and specific. How much should the community "acknowledge and respect the watershed's natural processes in land use decisions"? How specifically would the community "maintain its riverine floodplains to accommodate flood events" and “natural processes” in reaches where the channel has downcut 8 feet or more since settlement? Indeed, how will the people of the Valley build the guiding principles into a vision for the future that provides perpetual motivation for the present? For example, how will the CRC "promote the conservation of water from all sectors of the community's water users for the benefit of "recreational and natural resources"? Nevada water law grants water rights to those who appropriated the water first (the agricultural community) and proclaims that any water not used by permit holders would be reallocated. These questions and others stimulated much thought and discussion and spawned several proposals for research and construction of river restoration projects.

The CRC Water Quality Working Group searched for definitions of water quality that would fit the coalition's stated purpose of integrating management of water quality, quantity, flooding, and habitat. They realized that the river's numerical and beneficial use water quality standards were not met in several reaches of the river. In the Carson Valley, the beneficial use of a cold water fishery was no longer viable. They investigated the sources of pollution, interpreted a decade's worth of water sampling data, and went to the river to examine the channel, bed and banks.

Riparian Proper Functioning Condition: To get beyond point and nonpoint sources of pollution and values-based standards as the only focus for water quality, the Riparian Proper Functioning Condition (PFC) assessment protocol could broaden people's perspective. Riparian PFC was developed by an inter-agency, interdisciplinary team (Pritchard et al.1993 and 1998). This method rates any given river or stream on a reach-by-reach basis, focusing on the physics that are the foundation for biological and social values. PFC assessment focuses watershed management on those reaches that are "at risk" and those attributes that make them so.

Given the potential and capability of any particular setting, **a riparian area functions properly** when adequate vegetation, landform, or large woody debris are present to:

1) Dissipate stream energy associated with high water flows, thereby reducing erosion and improving water quality;
2) Filter sediment, capture bedload, and aid floodplain development;
3) Improve floodwater retention and groundwater recharge;
4) Develop root masses that stabilize streambanks against cutting action;
5) Develop diverse ponding and channel characteristics to provide the habitat and the water depth, duration, and temperature necessary for fish production, waterfowl breeding and other uses; and
6) Support greater biodiversity

Riparian PFC assessment does not imply the goal of pre-settlement or pristine conditions. Rather, the process recognizes that a channel or riparian area can function properly without achieving its full natural potential or capability. However, an understanding of potential or capability for each reach allows the interdisciplinary team of assessors to rate each reach without the need for "magic numbers" or other concepts that may not apply. "Potential" is "the highest ecological status an area can attain given no political, social, or economic constraints". The "Capability" is "the highest ecological status a riparian-wetland area can attain given political, social, or economical constraints or limiting factors". Understanding capability-limiting factors helps a team avoid pointless discussions about issues irrelevant to real management opportunities. For example, in many reaches below dams, the altered sediment supply has changed the very nature of the system. Fortunately, this change has not yet happened to the

Carson River. Capability-based discussions accept limiting factors of the current situation and focus on the requirements of the riparian area to meet the PFC definition as it can be met now. **Determining Capability:** In urbanizing valleys, defining the capability requires a fair number of assumptions or decisions that are ultimately up to the private landowners and the public. Defining capability is therefore one of the crux issues of any integrated watershed management process. Yet it is a discussion that may have never happened, a process of focusing on the connections between channel and floodplain form and the management of the water and riparian vegetation. To define a river's capability, all the diverse interest groups and specialists and governmental bodies in a watershed must take stock of their resources, their challenges and their opportunities. During this planning process, tensions and conflicts become evident. It is problems like these that the process of integrated watershed management is meant to resolve.

Defining capability becomes essential for valleys where people crave the dynamic equilibrium of riparian proper functioning condition but cannot accommodate the great widths of the original natural floodplain nor remove other limiting factors such as water rights or diversion dams for consumptive use. After incision or channelization, narrow interests might strive for regaining stability through bank-protecting revetment or alternatively, a hands-off approach. Neither achieves the optimum benefits of combining the riparian imperatives for "proper functioning condition" with the freedom to choose desired future communities, land uses, or land forms.

Natural Tensions: Defining riparian capability considers the physical drivers of the setting, hydrology, vegetation, and landform, while evaluating trade offs or natural tensions between potential and capability with its social, economic or political limiting factors. Such limitations come in the form of bridges (especially narrow ones fed by high floodplain-damming roads), Levees (especially levees close to the channel rather than set far back from the channel), buildings or subdivisions (especially ones close to the channel or in the floodplain or meander belt width), diversion dams, water use commitments, etc. These and other limiting factors come with trade offs that create natural tensions often unrecognized in land use planning (Table 2).

Table 2 Some natural tensions in defining riparian capability.

1. The less land available for frequent flooding, the more artificial a riparian system may have to become with hardened banks and energy dissipating structures that constrain opportunities for dynamism and for the slowing of nutrient spirals and that may require ongoing maintenance.
2. Where insufficient water can be made available for riparian vegetation during critical times less vegetation will be available for energy dissipation and less water will be stored in the alluvial aquifer.
3. The less land between levees or available for a meander-belt-width sized new low floodplain, the more land will be subjected to infrequent big floods large enough to access the old high floodplain that became a terrace through incision.
4. The more diversion dams or drop structures become necessary for preventing incision, the less opportunity for the river to smoothly maintain a sinuous gentle gradient by gradually moving its location through meandering.
5. The greater the width of the incision and the new low floodplain, the narrower and deeper the active channel within it can become.

Therefore, the less land and water devoted to floodplain wetlands, the more money will have to be invested elsewhere for almost any standard of water quality.

Restoration Paradigms: To test the margins of political, social and economic constraint, to define "capability" or to describe the optimum "vision", from the guiding principles, the combined perspective of all involved parties, needs to become integrated (Cobourn 1997) The primary alternative restoration paradigms need definition and analysis. Each restoration paradigm will involve benefits and costs in economic, social and ecological terms and their level of acceptance will ultimately derive from the combination of interacting effects. Different paradigms may be appropriate for different reaches of the River, and an optimum vision for any given reach could involve aspects of all three. Currently three alternative paradigms (Table 3) for the future river seem relevant for the part of the Valley below Minden. Above there, urban development and differing geomorphology constrains some options.

Table 3 Alternative Paradigms for River Restoration and Water Quality, a preliminary list.

Benefits, Costs, and Offsetting economic factors	**1. Stabilize the river banks in close to their current position while adding active channel structure and vegetation**	**2. Elevate, revegetate and reform the river so that its current agricultural terrace becomes its riparian floodplain again**	**3. Excavate a new wide accessible floodplain at the river's current elevation and reform and revegetate the river to fit the new geomorphic context**
Benefits or riparian functions			
Dissipate stream energy	Roughness added with rock structures.	Broad floodplain with riparian vegetation dissipates energy across an area as wide as current development allows.	Moderately broad floodplain with riparian vegetation dissipates energy across an area at least as wide as needed for a meander belt width
Reduce erosion	Current high and unstable banks become somewhat stabilized with bioengineering riprap, or groins etc.	Current high unstable banks become the banks of off-channel ponds. The banks of the newly created channel, on the terrace, remain stable because of low stress and vegetation.	The current high and unstable banks become stabilized with riparian vegetation and rock where needed after moving them beyond the meander belt width and laying them back.
Improves water quality	Erosion reduced so sediment input lower	Riparian vegetation stabilizing banks so erosion reduced, nutrient-laden floodwater and even base flow infiltrates into root zone for plant uptake before re-release, denitrification at the aerobic/anaerobic fringe, stabilized banks narrow channel for lower insolation and increased shade	
Filter sediment	Deposition beside barbs or groins and on low-lying floodable areas	Water slows on a wide vegetated floodplain where fine sediment deposits on point-bars and floodplains before meandering gradually	Water slows on a **moderately** wide vegetated floodplain where fine sediment deposits on point-bars and floodplains before meandering gradually

		releases it to erosion during high flows.	releases it to erosion during high flows.
Capture bedload	Deposition beside rock structures.	Bedload captured on point bars as initially wide channel narrows, then bedload moves through channel without excess aggradation.	
Aid floodplain development	NA	See "capture bedload" above	
Improve floodwater retention	Floodwater infiltrates into terrace (10-50 year floodplain) briefly during very high water	Floodwater infiltrates into floodplain in most years. In high water years this may cover extensive areas for long periods.	Floodwater infiltrates into floodplain in most years. In high water years this may cover the entire meander belt width for long periods.
Ground water recharge	Area for recharge limited to wetted channel	Area for recharge as wide as allowed by incompatible land uses	Area for recharge as wide as the meander belt width.
Develop root masses that stabilize streambanks against cutting action	Shear stress excessive for vegetation in many places, but woody vegetation near structures or as created by bioengineering techniques will provide additional stabilizing effects	High water table allows wetland and riparian vegetation with very high root density and depth for excellent bank stability **because** water for it remains consistently available	High water table allows wetland and riparian vegetation with very high root density and depth for excellent bank stability **if** water for it remains consistently available
Develop diverse ponding and channel characteristics to provide the habitat and water depth, duration, and temperature necessary for fish production, waterfowl breeding and other uses	Pools and riffles created by the structures and by woody riparian vegetation if it grows in other places	Pools and riffles or other habitat features created by channel meandering and riparian vegetation. Abandoned meanders (oxbow lakes) or the abandoned channel constructed into ponds create off channel pools	Pools and riffles or other habitat features created by channel meandering and riparian vegetation. Abandoned meanders (oxbow lakes) create off channel pools
Support greater biodiversity	Barbs or groins create pools for fish habitat and vegetation associated with them provides wildlife habitat	Most closely simulates the prehistoric mix of habitat conditions	Closely simulates the prehistoric mix of habitat conditions closest to the channel and includes other habitats on the terrace
Implementation costs			
Conservation easement for keeping	Moderate	Greatest as the floodway is the widest with this design	Beyond the meander belt width floodway, additional conservation easements are

development out of the floodway			for open space purposes
Disruption of traditional land uses	Allows continued modern traditional agricultural use of the open space floodway unless hindered by water quality restrictions.	Traditional irrigation less needed because high water provides lasting high water table with soil saturation in wet years. Care needed for livestock grazing management	Floodway flooded frequently by high water that provides lasting soil saturation in wet years and until irrigation diversion depletes river water within each river segment. Care needed for livestock grazing management
Meander easements	NA	May be necessary to allow the River to meander.	Necessary if river is allowed to build its own new floodplain. Also may be necessary to allow continued meandering if floodplain excavated.
Construction costs	Placement of large rock structures, riparian plantings, engineering etc.	Engineering, cutting and vegetationally armoring a new highly sinuous channel on the terrace and converting current incised channel into ponds (or NA if channel width and sediment supply allows sufficient deposition to refill incised channel).	NA if the river is allowed to created its own new floodplain. Unknown, but very large if the project is excavated and a new sinuous channel stabilized by vegetation.
Maintenance costs	Ongoing as the high shear stress of high water will alter the constructed channel and it will not repair itself.	The cost of conservative riparian vegetation management	
Offsetting economic factors			
Land use	Dual use of large floodway for agricultural open space	Dual use of large floodway for agricultural open space Enhanced wildlife habitat allows recreational (fee?) use	Conservative use of floodway for livestock grazing possible or useful for nutrient management. Enhanced wildlife habitat allows recreational (fee?) use
Wetland mitigation bank	NA or limited	Very large	Moderately large
Ease of irrigation	Greater river stability protects farm and irrigation infrastructure	High water table throughout valley extends the season of	Protects farm and irrigation infrastructure from flood damage and expands

		water availability to pastures.	riparian zone with extended water availability for forage.
Top soil sale	NA	NA	Very large supply probably overwhelms demand
Flood protection	No change	Current 100 year floodplain area enlarged	More land outside floodway is flood protected by large capacity of flood channel/floodplain
Water quality	Continued impairment risks legal action against polluters even if water cannot meet standards	Improved water quality may meet standards for beneficial uses	Improved water quality may meet standards for beneficial uses. However, low flow from diversions may preclude this.

CONCLUSION

In semiarid landscapes, cool, clear, well oxygenated and nontoxic water flows best from watersheds that capture and store seasonal or storm water for safe release during summer, fall or winter droughts. Flood water that spreads across a broad riparian floodplain soaks into a big sponge. Conversely, non-inundated lands capture little floodwater. Floodwater in downcut channels constrained by high banks expends much energy on bank erosion and quickly flushes the soil nutrient capitol of millenia downstream to where it eutrophies receiving waters. Against the high shear stress, emerging riparian vegetation struggles or fails, resulting in channels that grow wider quickly. Wide channels and wide water quickly becomes too cold in winter and too warm in summer. They also fill with sediment, eventually restricting the channel’ s flood capacity.

If the Carson River were allowed to absorb the impact of floods by building point bars and floodplains after eroding banks, riparian vegetation would grow to stabilize low banks and an expanding riparian surface. Floods would again dissipate their energy and flood water would recharge an alluvial aquifer. As the River meandered across and through that surface, the reinforced banks and a natural form that dissipates energy, would slow the dynamic movement. Slow movement through a meander corridor would maintain a persistent form. The persistent movement would keep excess deposited sediment from permanently altering the functional land form. The meandering channel and floodplain form would improve water quality and fish and wildlife habitat with the resilience of riparian vegetation. The strongly rooted vegetation would help keep banks close, narrowing a deep active channel that easily moves bedload sediment.

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THE HYDRAULIC ENGINEER'S ROLE IN STREAM RESTORATION PROJECTS

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Abstract: The hydraulic engineer's role on the interdisciplinary team for stream restoration projects is often unclear. This paper discusses some of the inherent difficulties in teamwork in this new mission area, how they can be resolved, and what specific contributions the hydraulic engineer can make.

INTRODUCTION

For many hydraulic engineers, working on stream restoration projects presents new challenges. One of these is the engineer's role on the interdisciplinary team that is a necessary part of any environmental restoration work (Federal Interagency Stream Restoration Working Group, 1998). This paper discusses some of the inherent difficulties in teamwork in this new mission area, how they can be resolved, and what specific contributions the hydraulic engineer can make. As a disclaimer, it must be noted that both stream restoration projects and "traditional" engineering projects vary widely in scope and scale, and that there are many projects and teams to which the following discussions will not apply. This paper addresses issues that seem to be common, but are fortunately not universal.

HOW ARE STREAM RESTORATION PROJECTS DIFFERENT?

The differences: Stream restoration projects are often different than traditional engineering projects in a number of ways:

- the goals and design criteria are often less well defined
- an interdisciplinary approach is necessary: engineers must work closely with biologists and ecologists
- the design criteria may be set by biologists or other disciplines
- the multiplicity of variables in a natural system may make the driving processes more difficult to understand, may cause the system's responses to be less predictable, and may make the evaluation of success more difficult
- it may be acceptable to take more risks (that is, a certain level of failure may be acceptable)
- the budget may be lower
- there is often less agreement on engineering methodology: what data should be collected, what survey data is necessary, what analyses should be performed
- there may be significant distrust, within the technical team itself, of the engineer's methods and motives

The similarities: Traditional hydraulic engineering projects often involve working with other disciplines or working on a small scale. Streambank protection projects, for example, are often small-scale projects where risk to life and limb is not a factor, so a higher level of risk in design may be acceptable. New techniques, low-cost methods, or demonstration projects may be more readily incorporated into the design. Hydraulic engineers have normally worked on interdisciplinary teams with economists, real estate specialists, cost estimators, and civil, geotechnical, and structural engineers. In almost any engineering projects, the wishes of local residents, local sponsors, and political representatives have to be understood and taken into consideration. The foregoing examples show that engineers have a track record of working on small-scale projects, of accepting higher risks where appropriate, and of working with other disciplines.

The defining differences: Although many differences between stream restoration projects and "traditional" hydraulic engineering projects have been listed above, there are three major differences which can diminish the effectiveness of the interdisciplinary team. These are:

- lack of consensus on analytical methods and restoration techniques
- lack of clearly defined roles within the interdisciplinary team
- distrust of the hydraulic engineer's methods and motives.

The first two problems stem from the fact that stream restoration is a new and rapidly developing mission area. Many practitioners from different technical backgrounds are using different analytical methods and restoration techniques, with varying degrees of success or failure. When a technique fails, it may not be immediately clear whether the failure was caused by the choice of technique, by the design details, by the construction methods, or by other unrelated factors. As a result, there is not yet general agreement among practitioners on techniques and analysis methods. This can be contrasted with flood control projects, where there is general agreement on the range of feasible project alternatives, required analyses, and necessary data collection. Using flood control projects as an example again, there is a general understanding among the team members as to what tasks are performed by each discipline. On stream restoration projects, there can be confusion about the responsibilities of each discipline. This is often accompanied by a lack of knowledge of what types of analyses will be performed by other team members, and what information they might require from one's own discipline. This problem results from the facts that biologists and engineers are often not used to working in close coordination, are often not well versed in each others' fields, and often use terminology that is unfamiliar to those in other disciplines. This lack of team experience should be resolved by the ongoing involvement of biologists and engineers on stream restoration projects. The third barrier to effective teamwork, a distrust of engineering methods and motives, is often directly related to the negative environmental impacts of flood control projects built prior to NEPA requirements. For a hydraulic engineer, the ability to discuss engineering alternatives in terms that the entire team can understand, a working knowledge of biological processes and analyses, and a willingness to consider alternatives brought up by other team members can be helpful in breaking down stereotypes of the "typical" engineer.

TEAMWORK

How the interdisciplinary team should work: An experienced and focused interdisciplinary team works like a well-oiled machine. The various team members know their roles and responsibilities, so time and energyare not wasted establishing duties. In a way, it is like a dance, where everyone knows the steps: for example, the team knows who is responsible for obtaining survey data, who must provide input, who will use the data, and so on. As in a dance, an experienced team can handle one or two inexperienced members (they will be reminded if they forget a task), but there is a general understanding of what tasks need to be performed, when they occur, and by whom they will be performed. Another analogy is that an effective interdisciplinary team works like a good volleyball team. In a good volleyball team, every player has to know what their position is and stick to it, as well as knowing what areas are being covered by the other team members. In an effective interdisciplinary team, each member must know what their own areas of responsibility are, should be careful not to take over other team members' areas, and must know enough about the responsibilities of the other team members to interact well with them.

How the interdisciplinary team often does (not) work: In practice, the interdisciplinary team on stream restoration projects may have some problems. If the analogy is a dance, sometimes it seems as though the team doesn't know whether the dance is a waltz or a polka. If the analogy is a volleyball team, sometimes the entire team seems to be in one corner looking at one aspect of the study, and other key study components are going unnoticed. There are often basic disagreements between team members as to what the final project will look like, what its components will be, and what the study and analysis process will involve. These differences in the vision of the project and the study can lead to major problems if not resolved.

TECHNICAL ROLE OF THE HYDRAULIC ENGINEER

General: As part of the study team, the hydraulic engineer brings an understanding of physical processes related to water and sediment. This includes the areas of hydrology, hydraulics, sedimentation, river mechanics, and water quality. An accurate understanding of the cause-and-effect linkages within the study area is essential to diagnosing the problem and prescribing a solution. The hydraulic engineer can compute (for example) the impact of various combinations of stormwater retrofits within a watershed on peak flows and water quality, or determine which alternatives within a watershed would be most cost-effective for reducing sediment loads. The hydraulic engineer can also assess the impacts of proposed alternatives on flooding, drainage, channel processes, and existing infrastructure.

Technical contributions: The following is a partial listing of some of the technical contributions which the hydraulic engineer can make to stream restoration projects:

- analysis of gage data
- estimates of flows of various frequencies: base flows, drought flows, flood flows
- fish passage flow estimates and structure design
- flow-duration curves
- frequency of overbank flooding or wetland inundation
- contribution of groundwater to streamflow
- hydrologic modeling of watershed: flows for existing, future, and pre-development conditions
- discharge hydrographs for various flood events
- cumulative impacts of alternative combinations of stormwater management ponds
- treatment of acid mine drainage
- water quality testing and modeling
- water surface profiles for various flows and alternatives
- impacts of proposed projects on flooding
- stage-discharge and stage-frequency curves at desired locations
- channel velocities and shear stresses
- hydraulic modeling of depths and velocities to match desired conditions
- two- and three-dimensional hydraulic, sediment, and water quality modeling
- unsteady flow modeling
- design of instream or floodplain structures: structure selection, rock sizing, scour depth computations
- design of streambank erosion protection
- siting and design of grade control structures
- design of stormwater management structures
- hydrologic and hydraulic design of wetlands and associated structures
- design of channel modifications and associated structures
- use of pilot channels to return streams to historic channels
- sediment transport modeling
- stability assessment at watershed, reach, and site scales
- assessment of channel stability for existing and future conditions
- determination of channel-forming discharge
- design of stable alluvial channel to transport inflowing sediment load
- design of threshold (armored) channel
- sediment budget for assessment of long-term stability
- sediment budget for assessment of stability during a flood event.

This list demonstrates the wide range of hydraulic engineering assistance available on stream restoration projects.

Corps of Engineers design procedures: The U.S. Army Corps of Engineers is currently preparing a design procedure for hydraulic engineers involved in stream restoration projects (Fripp, Copeland, and Jonas; 2001). The stream restoration design procedure presented in the referenced paper brings together geomorphic principles and traditional engineering methods. The combination of hydraulic geometry theory and analytical techniques can result in a reliable and cost-effective design, which is adapted to site-specific goals and conditions. This procedure builds on methods presented in earlier Corps publications on channel stability, sediment transport modeling, and hydraulics of flood control channels (Thomas et al, 2000; USACE 1982, 1994, and 1994).

Corps of Engineers contributions to stream restoration studies: The Corps of Engineers has some specific advantages to offer in stream restoration studies. As a federal agency, the Corps is often able to bring all the stakeholders (including other federal agencies) to the table. Since the Corps is organized by watershed boundaries, it can cut across political jurisdictions to address an entire watershed. And, last but not least, the Corps is a truly interdisciplinary agency, employing biologists and other scientists as well as engineers, and conducting a large research program in both the environmental and hydraulic engineering fields.

CONCLUSIONS

There are some legitimate difficulties in teamwork on stream restoration projects. Many of these are natural growing pains that come from working in a new and rapidly developing mission area, with newly formed teams that have to learn to work together. The good news is that conditions are improving, and that a general consensus among practitioners will probably occur at some point in the future. This process is aided by peer discussion, the exchange of ideas at conferences such as this one, and by the guidance provided by federal agencies and professional organizations, such as ASCE. As more studies and projects are completed, there seems to be more agreement within the field. It is hoped that the list of hydraulic engineering tasks presented may be useful to study teams working on stream restoration projects.

Acknowledgements

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DESIGN AND CONSTRUCTION OF A NATURAL STREAM CHANNEL WITH CONSIDERATION OF TEMPORAL VARIABILITY OF BOTH STREAM LOCATION AND BEDLOAD MOVEMENT

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Abstract: Floods in 1997 destroyed several homes along Big Canyon Creek near Peck in central Idaho. The management response was to move the gravel deposits from in the channel and create piled gravel levees along the stream. Similar gravel removal projects were completed after floods in 1956, 1965, 1974 and 1981. This repeated gravel removal and channel widening had left an incised, broad, 'U' shaped channel with two different bedload transport regimes. During a channel forming flow of approximately 900 $ft^3 s^{-1}$ (4.3 csm, approximately a 1.8 year return interval) the channel generated a relatively low tractive stress of approximately 1.5 $lb.ft^2$ and could not transport the bedload coming into the reach. An opposite response was displayed during less frequent events. Due to the levees along the channel, a flow of 3,200 $ft^3 s^{-1}$ (15 csm, approximately a ten year recurrence interval) generated shear stress in excess of 5.4 $lb.ft^2$, enough to mobilize a majority of particles in the bed and banks.

A design was completed to narrow and deepen the channel during frequently recurring floods, and to excavate a floodplain accessible to flows during larger floods. This design increased the reach-averaged tractive stress approximately 15% during the channel forming flow and decreased the reach-averaged tractive stress 18% during higher, less frequent flood flows. Integrated with the physical channel manipulation was the use of vegetative plantings to change the existing uniform flow patterns to a more typical and more variable pattern of pools (scour areas) and riffles (deposition areas). The confluence of a small tributary with a mass-failure dominated sediment regime as well as an undersized bridge provided challenges in the design. The design was installed on over 6,000 feet of the Big Canyon Creek channel in late 1999 at a total cost of approximately $200,000.

The expected impact on channel processes from implementation of the design are to move bedload through the channel system more consistently. This consistent movement will prevent large areas of aggradation during years of smaller floods, and oppositely, large scale channel adjustments during years with larger floods. The overall result will be a stream with more predictable characteristics upon which to base management strategies, and also a stream with more diverse habitat for fish and wildlife. Preliminary measurements and observations indicate the modifications to the channel are having the desired impacts.

INTRODUCTION

Big Canyon Creek, a tributary to the Clearwater River in Nez Perce County, Idaho, experienced several large flow events in 1996 and 1997. The channel had extensive erosion and deposition, and several homes were destroyed. The channel was cleared of gravel with a bulldozer, and an uncompacted gravel levee was constructed on the west side of the channel. Similar actions have been taken in the stream in 1955, 1965, 1974, and 1985 (personal communication, local residents). A grant was obtained by Nez Perce County from the Federal Emergency Management Agency to purchase properties in the floodplain and to improve the flooding characteristics of the stream. The county contracted with the Nez Perce Soil and Water Conservation District (SWCD) to be the local contracting authority for the flood improvement work. The SWCD in turn requested NRCS to design modifications to the channel that would improve the flooding characteristics.

Big Canyon Creek enters the Clearwater River at approximately river mile 35 on the left side. The project is located on the lower two miles of the creek, starting at the confluence of Bear Creek with Big Canyon Creek just below the town of Peck (Figure 1).

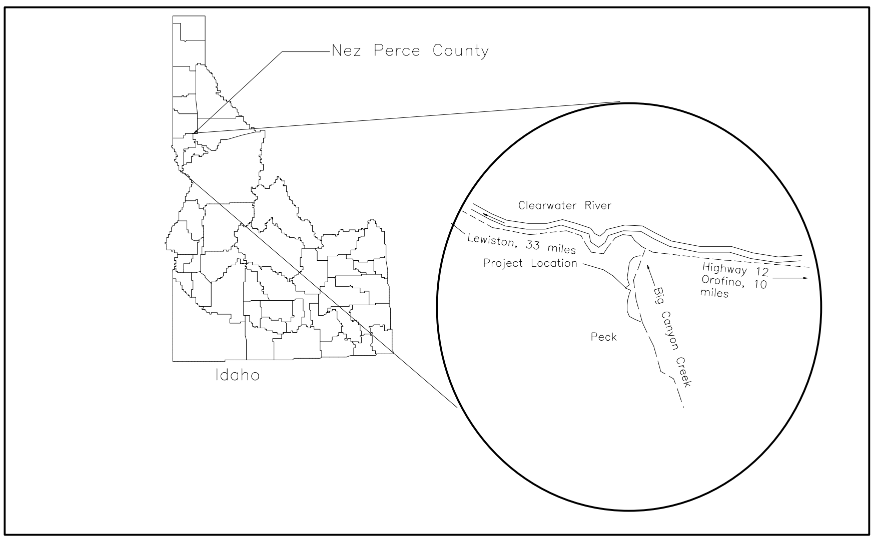

Figure 1. Project Location

ANALYSIS AND DESIGN

<u>Geology and Geomorphology</u>: Big Canyon Creek is in the Columbia Plateau Geomorphic Province. The creek flows through a canyon developed in deeply dissected Precambrian, Jurassic and Cretaceous Age bedrock mantled by Tertiary Age basalt (Rember and Bennett, 1973). The Precambrian and Jurassic Age bedrock consists of gneiss, schist and metadiabase, predominantly from the Belt Series Wallace Formation. Cretaceous Age quartz diorite of the Idaho Batholith intruded and contributed to the metamorphism of the Precambrian and Jurassic Formations. The Tertiary Age Columbia River basalt consists of numerous flows that filled Tertiary valleys and canyons mantling the entire area except for a few mountain peaks in the headwaters area. The basalt is being dissected along pre-flow valleys and canyons and in response to post-tertiary faulting activity. Big Canyon Creek has exhumed the pre-Tertiary bedrock throughout the project area.

Big Canyon Creek is a relatively steep stream with an average gradient of 1.2% over the project reach, with several sites having gradients in excess of 2%. Bed material is coarse gravel to large cobbles with a D_{50} of 3" and a D_{84} and D_{16} of 7" and 1", respectively. The stream currently has minimal sinuosity with a stream length to valley length ratio of 1.05. Big Canyon Creek is classified as a plane bed stream (Montgomery and Buffington, 1993). A plane bed stream lacks the rhythmic bedforms characteristic of either the steeper step-pool or the flatter pool-riffle channel morphology. Plane bed streams are often sediment supply limited during low flows and transport limited during large events, and this is the case with Big Canyon Creek. There are several occurrences of bedrock base elevation control in the channel bottom. Big Canyon Creek is entrenched, and laterally confined. Pre-project width at channel forming flow was 60 to 70 feet with an average depth of 2 to 2.4 feet.

Bedload is subrounded to rounded basalt with some subangular blocky material and a small percentage of granite and gneiss. Deposits are highly imbricated with very low embeddedness in channel. Before the project bedload was being transported in only the highest peak flows. This is due in part to channel widening because of gravel excavation after major flood events. Due to confinement by the levees and the entrenchment of the existing channel, average flow depth during large storms is very high, ranging between five and six feet. This deep flow caused by lack of access to a flood plain generates large tractive stresses, which can transport the bed and bank material of the channel. These large slugs of bedload excavated from the bed and banks of the channel are transported short distances downstream and deposited, disrupting the channel cross section.

Historically, the creek was less entrenched, perhaps with a channel depth four to five feet below the valley floor, with access to part of the valley as a flood plain. Historically the channel may have had co-dominant overflow channels that would capture the main channel every 10 to 100 years (one to 10 disturbance events, see Knighton, 1984, figure 4.1). This type of channel change implies sporadic movement rather than gradual erosion as a channel shaping force.

Bedload movement was less sporadic historically compared to the current situation. Flows occurring every one to two years distributed bedload evenly in the channel, depositing fine veneers of sediment on floodplain surfaces. More extreme flows transported these frequently deposited materials out of the creek to the river.

Hydrology: A USGS streamflow gage on Lapwai Creek at Lapwai, (gage number 13342450) has a record from 1975 to the present. Lapwai Creek is located 20 miles west of Big Canyon Creek, and the watershed has a similar shape, size, aspect, and is in similar geology. Flows from Lapwai Creek were used directly for the Big Canyon Creek design. Channel forming flow was estimated from the Lapwai flood frequency curve, and then checked against depositional features in the field. A single flow estimate of 8,360 cfs on 1/29/65 exists for Big Canyon Creek (IDWR, 1970). This flow is approximately an 80 to 100 year recurrence interval on the Lapwai Creek flood frequency curve, and this is consistent with the recurrence intervals typically assigned to the 1964-65 floods.

The flood frequency curve for Lapwai Creek is distinctly bent, with a flat section in the more frequently recurring storms and a much steeper section describing the more extreme events. The shape is indicative of a rain on snow flooding mechanism (Macdonald, et al., 1997). This high ratio of extreme flows to more frequent flows points to a channel that is prone to sporadic movement. The lower, more frequent flows do not condition the channel for flows that are eight to ten times greater, and so large scale channel change can occur.

Hydraulics: All flow capacity and velocity was estimated using Manning's *n* as an estimate of roughness. Barnes (1967) computed *n* on the South Fork of the Clearwater to 0.051. A procedure developed by Burkham and Dawdy (1976) yields an estimated *n* of 0.055. In most of the flow rating curves, *n* was allowed to vary linearly from 0.05 at low flows to 0.04 at high flows when examining the existing cross section. The rational for the lower *n* value is the lack of obstructions or cross sectional variation in the existing channel. When examining the proposed channel, which has access to a flood plain, *n* was allowed to vary from 0.05 to 0.045.

Proposed cross sectional changes were examined using WinXSPRO (USFS 1998). Existing cross sections were input, the proper slope and roughness assigned, and flow capacity, velocity and depth were examined at a station. Flows examined were 200 cfs, an estimate of flow on the day of the field surveys; 700 and 900 cfs as a low and high estimate of channel forming flow; 3,200 cfs to estimate the 10-year recurrence interval and 5000 cfs to estimate the 25-year flow.

To estimate water surface profiles, the farthest downstream cross section was used to set the elevation for each flow, and water surface elevations were calculated going back upstream using a Standard Step procedure (Chow, 1959). A velocity coefficient of α=1.15 and an eddy loss coefficient of 15% of the velocity head were used. These calculations were performed for Site B, Site C, and Site D and E, in both the existing and planned condition.

SELECTED ALTERNATIVE AND CONSTRUCTION

Selected Treatment: The selected treatment on Big Canyon Creek was to excavate a floodplain along the west side of the channel at four sites totaling 6000 feet (Figure 2). These sites are bounded by bedrock channel control or a confluence at their upstream and downstream limits. The primary goals of the cross section modifications are two fold. In order to move small amounts of bedload through the channel system more regularly, the channel forming flow needs to be deeper, thus the low flow channel needs to be narrower. Secondly, the higher flows need to be shallower to keep the channel from attacking its own bed and banks. With these goals in mind, two floodplain levels were constructed. The lower level coincides with the channel forming flow of 700 to 900 cfs, and the upper floodplain level coincides with the elevation of the 10- to 25-year flow of 3,200 to 5,000 cfs (Figure 3).

To estimate the appropriate constructed channel width and depth, the regression equations of Williams (1986) were examined. None of these relations related well to any of the channel features observed in the field. Secondly, the relationships presented in Knighton (1984) and Emmett (1975) were used, and these relationships provided answers much closer to those observed in the field. Using these relationships, the constructed cross section has an average channel width at channel forming flow of 48 feet, an average depth of 3 feet, and a maximum depth of 5 feet.

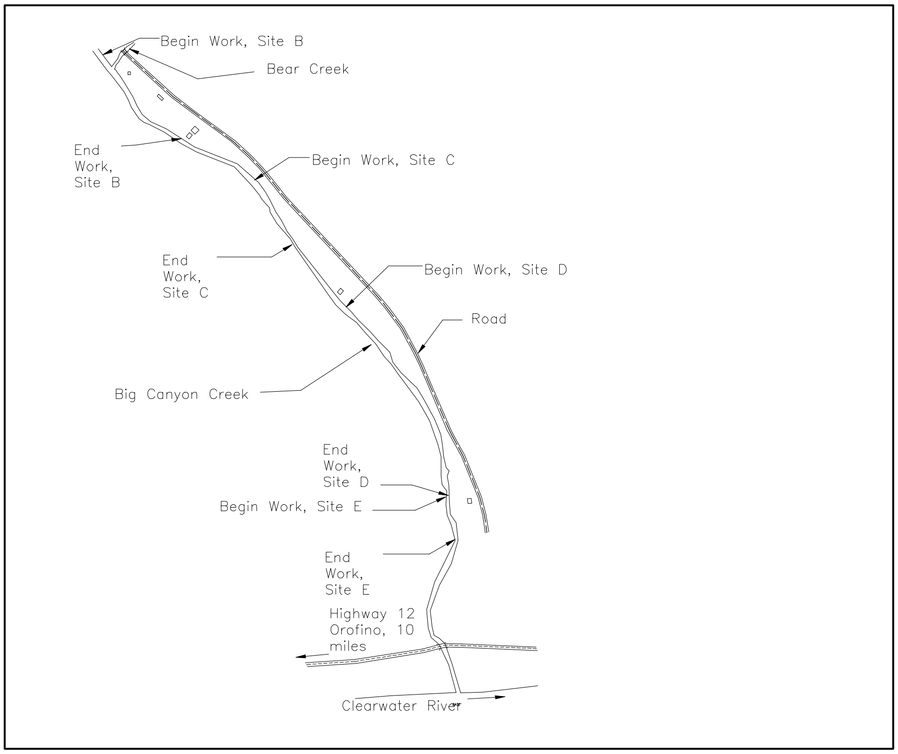

Figure 2. Location of construction sites B, C, D, and E.

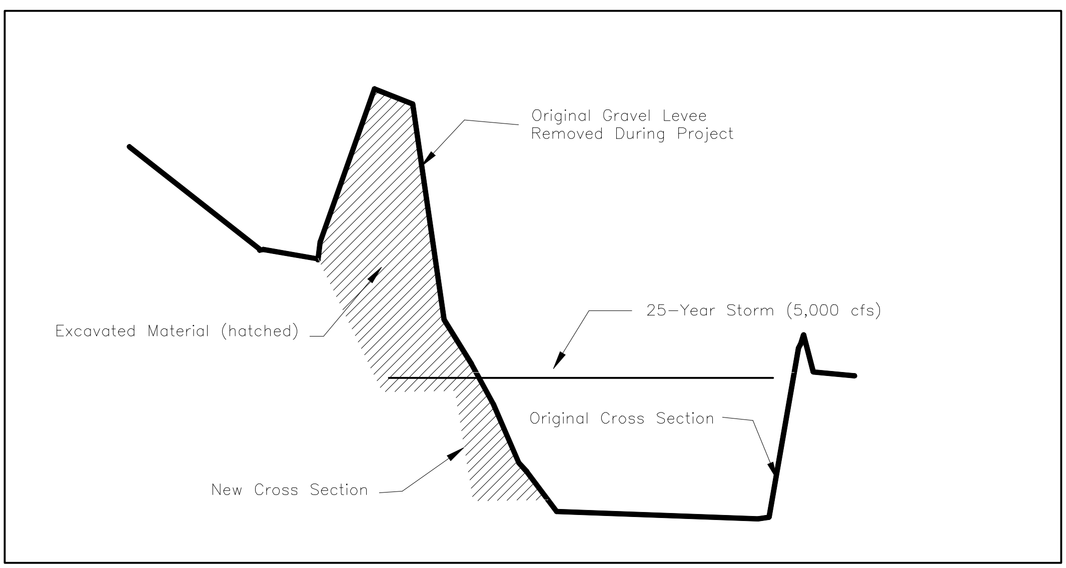

Figure 3. Constructed channel cross section compared to original channel cross section

Although local variations of pools and riffles are not hydraulically dominant in these stream types, they do exist, and account for most of the cross sectional variation. Pools are erosional features that form at areas of locally steep slope during channel forming flows, and these are at narrow stream cross sections (Leopold, 1994, McLean, et al., 1999). Riffles are depositional features occurring at flatter water surface slopes and wider channel cross sections. Knighton (1984) reports that local width variations of 52 feet for riffles and 45 feet for pools are indicated.

To mimic the historic cross sectional variability, pools and riffles were built into the constructed cross section. A pool-to-pool spacing of 7 to 11 channel widths or about 300 to 500 feet is indicated by Williams (1986) and was selected for Big Canyon Creek. Pools and riffles were located in the constructed cross sections by examining the existing channel slope and cross section. Also, known 'anchor' points such as confluences (riffle) and bedrock constraints (pools) were used to locate constructed features. At locations where a pool was desired, the cross section was kept narrow, and the excavated flood plain was kept slightly higher than the channel forming flow elevation. Where a riffle was desired, the cross section was allowed to widen, and the excavated flood plain elevation was placed slightly lower than the channel forming flow elevation.

Four types of structures are used to support the cross sectional modifications, structurally as well as hydraulically. Bank barbs are low rock weirs, oriented upstream, that force a hydraulic jump. They extend through the thalweg of the channel, and the weir section is set at an elevation that maximizes the strength of the jump at the desired flow. Barbs have the effect of hydraulically narrowing the channel, both by the scour caused from the jump and by forcing flow to the center of the channel. Barbs are employed at the head of pools, and where the channel is wider than desired.

Rock riprap was installed at the base of the excavated slope on the floodplain to minimize erosion at transitions. This rock will also prevent channel migration if a co-dominant channel forms in the flood plain at the base of the excavated slope.

Floodplain sills are rock riprap excavated into the constructed floodplain, oriented perpendicular to the flow. They are used to assure the elevation of the floodplain, and to prevent cutoff chutes from forming in the floodplain while it is being vegetated.

Whole tree revetments are used to roughen the banks of the channel where a deposition zone or riffle is desired. Whole tree revetments are placed up on the lowest excavated floodplain with the root mass extending into the flood flows. Maximum depth of water on this floodplain surface during a 25-year storm will be 3.5 feet. Trees are specified to have at least an 8-foot diameter root mass to keep the center of gravity of the tree from being submerged to assure stability.

The confluence of Bear Creek and Big Canyon Creek at Site B is being treated for similar problems as Big Canyon Creek. Bear Creek also has been widened and diked after floods. In order to assure a deep, narrow channel cross section, opposing bank barbs, known as V weirs, are placed at three sites in the channel. In addition, extensive work was done to reconstruct and widen the confluence between the two creeks.

<u>Construction</u>: Construction of the project was completed in October and November of 1999. Willow plantings on the constructed floodplain were completed in December 1999. Over 21,000 cubic yards of gravel were excavated, creating or providing access to over 27 acres of new floodplain. Over 3,700 cubic yards of rock were installed as bank barbs, rock toe protection for the floodplain, floodplain sills and ballast for the tree revetments. 120 whole trees were installed as tree revetments. Over 12,000 stems of willow plantings were completed.

The constructed channel cross section changed the physical parameters in Big Canyon Creek. For example, the tractive stress distributed along the channel section at site B increased 7% at channel forming flow and decreased 17% at a 25-year flow (Figure 4). The changes, along with increased flow disturbance from the installed bank barbs and vegetation will improve the sediment transport and flow characteristics at all flow levels in the creek. It should be noted that these works of improvement will not change flood elevations substantially, lowering most water surfaces by no more than six inches. Most of the benefits from the project are expected to be derived from more consistent bedload transport rates, and as a result, less severe adjustment of channel plan form.

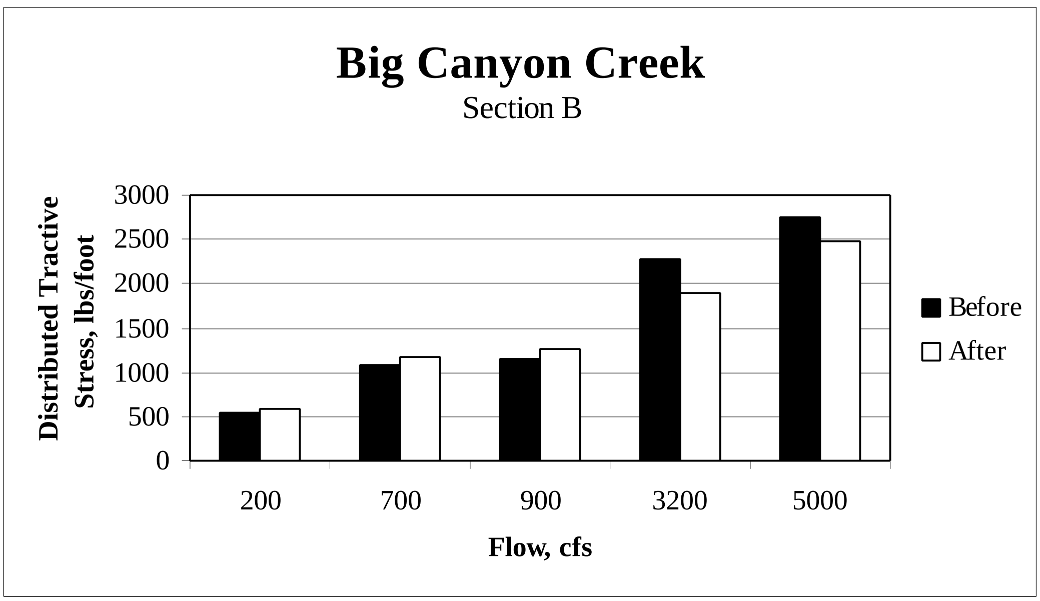

Figure 4. Change in reach-averaged tractive stress after completion of construction on Big Canyon Creek.

Detailed topographic surveys were completed before and after construction. Subsequent surveys will be used to determine if the assumptions used in the design were correct. Although high flow runoff in the spring of 2000 was not a sediment transporting event, indications are that hydraulics have been substantially changed, and that the selected elevations of the floodplains will allow flooding at the appropriate flow rates.

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CHANNEL RESTORATION DESIGN FOR MEANDERING RIVERS

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Abstract: This paper presents a framework for channel restoration design that bridges the divide between reconnaissance level geomorphological designs at one extreme and numerical modelling of hydrodynamics, sediment transport and morphological change at the other. The method nests within the broader approach to assessment of the watershed and river presented by Fripp *et al.* in the companion paper (Fripp *et al*., 2001). Re-establishing equilibrium between the sediment supply and available transport capacity in the restored reach is the primary objective of the design framework. A geomorphic-engineering approach is employed, which recognises that the river is ultimately the best restorer of its natural morphology and uses prompted recovery as an element in the restoration design. This is accomplished through designing a general channel mold to generate the broad dimensions of the restored channel, and then allowing the river itself to develop the intricate cross-sectional detail and intra-reach morphological features necessary to complete the restoration process. The geomorphic-engineering approach provides a practical solution by striking a balance between empirical/statistical and analytical (process-based) methods. The range of techniques that comprise the approach facilitate a realistic solution to the indeterminacy problem and confidence bands applied to 'typed' morphological equations provide a mechanism through which natural rivers can be used as realistic analogues for channel restoration design. By accounting for natural system variability, the design framework is an appropriate platform for generating restoration designs that mimic the natural channel morphologies and environmental attributes in undisturbed systems, while meeting multi-functional goals for channel stability and low maintenance requirement. Constructing physical habitats in restoration schemes is unsustainable because these features have geomorphic form but do not fulfil a geomorphological function. Conversely, in a geomorphic-engineering design, the types and levels of physical habitat diversity that are produced are sustainable in the restored reach because they are appropriate to the type of river, adjust to the flow regime, respond to the dynamics of sediment movement and rest conformably within the watershed context. The approach presented is not a 'cookbook' procedure for river restoration but presents a framework within which the sound judgement of practitioners with experience in applied river science may be applied.

THE DESIGN PROCEDURE

On the basis of a four-year co-operative study at between the University of Nottingham and the U.S. Army Engineer Research and Development Center an enhanced design procedure for restoring stable channel dimensions has been identified (Figure 1). The procedure requires a range of different techniques including: field reconnaissance; detailed site survey; magnitude-frequency analysis; analytical solution of non-linear equations, and; hydraulic geometry analysis Until three-dimensional numerical modeling can accurately replicate the intricate form of natural river channels, the procedure provides an appropriate solution to bridge the divide between reconnaissance level geomorphological designs at one extreme and numerical modeling of hydrodynamics, sediment transport and morphological change at the other.

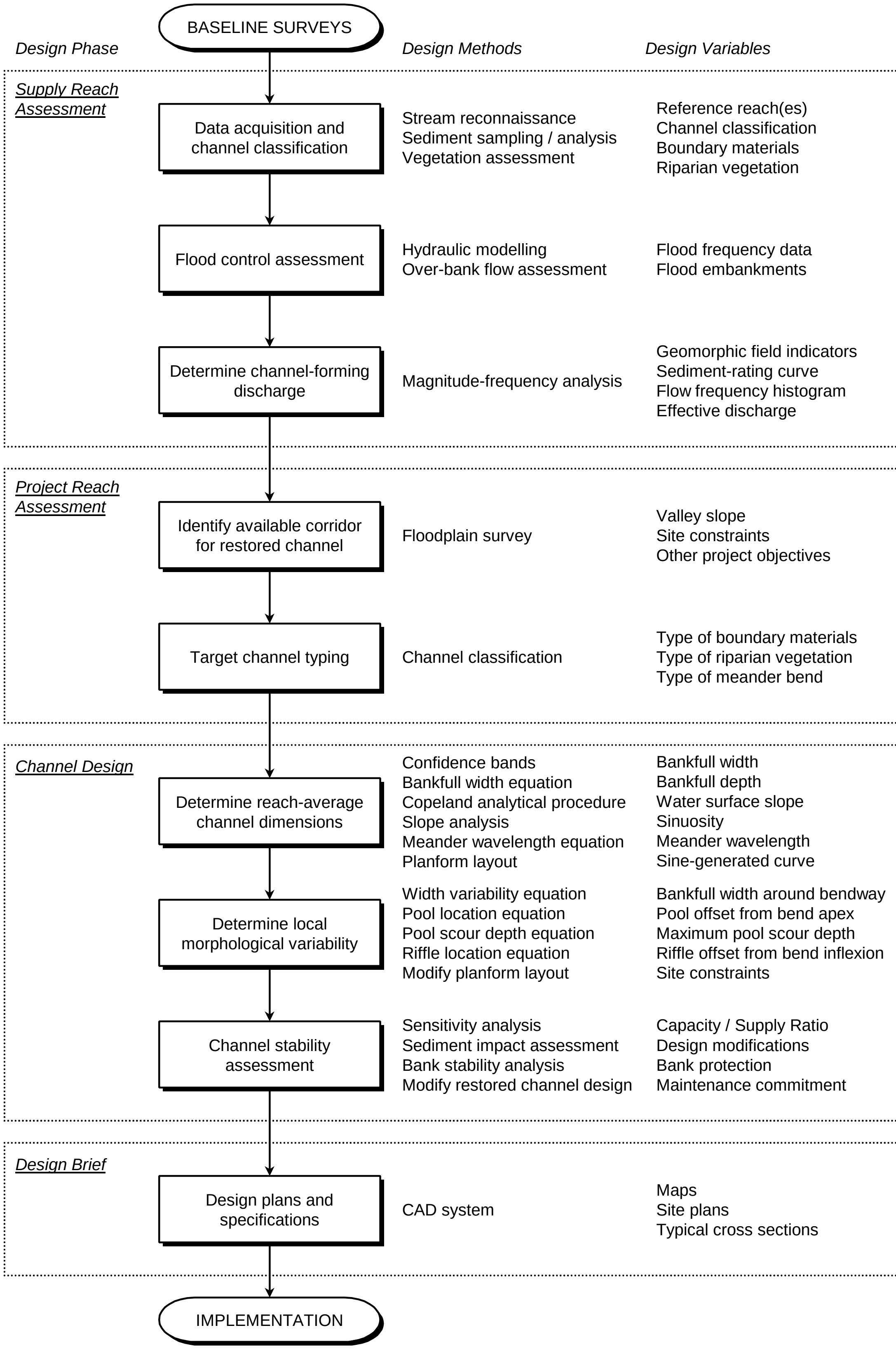


Figure 1 Best practice channel restoration design procedure.

A stable design solution and effective river management post-implementation require knowledge of both flow and sediment routing in the upstream supply reach, through the restored channel and in the existing channel downstream. This requires an approach based on sediment continuity and necessitates a holistic geomorphic appraisal that extends beyond the project reach scale (Figure 2). The procedure requires that a stable reference reach (or preferably reaches) be identified and used to supply essential baseline information, such as the magnitude and frequency of sediment-transporting flow events and a channel-forming discharge suitable for use in determining the preliminary main channel dimensions. If a Catchment Baseline Survey (Thorne *et al*., 1998) reveals that the majority of the watershed is highly unstable, then restoration of an individual reach to a stable (natural) configuration will require a commitment to long-term maintenance. However, in many disturbed watersheds, especially lowland streams that have been enlarged, sediment continuity will recover faster than the channel dimensions, so that aggradation and degradation are insignificant over periods of decades. In these cases, restoration to a more natural channel configuration using a design procedure based on sediment continuity may still present a viable management solution with a low maintenance commitment. The channel design procedure presented here must be supported by the results of earlier strategic studies that set the watershed context for the restoration scheme (Kondolf and Downs, 1996). The design procedure is structured as four sequential stages: i) a Supply Reach Assessment; ii) a Project Reach Assessment; iii) Channel Design, and iv) the final Design Brief:

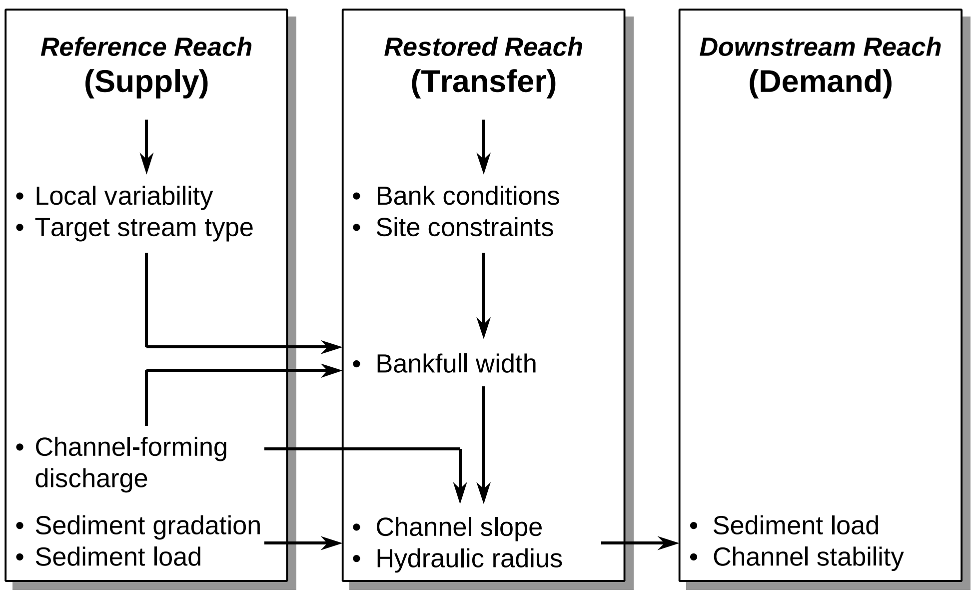


Figure 2 The fluvial system in terms of sediment supply, transfer and demand.

SUPPLY REACH ASSESSMENT

This assessment should follow from a strategic baseline survey of the watershed system, which has already identified reference reaches and classified the entire system in terms of potential destabilising phenomena and geomorphic conservation value. Stream reconnaissance methods should have been used to provide classificatory data on boundary materials and riparian vegetation and semi-quantitative information on the characteristic channel morphologies found throughout the watershed. Further details of these strategic studies are not given here as they are well documented elsewhere (e.g. Simon and Downs, 1995; Thorne, 1998; FISRWG, 1998). Bed material samples should be collected at reference reaches to provide particle size gradations of the supply sediment.

In many cases the preferred channel design solution will involve a two stage channel (or a channel with set-back flood embankments) with a main channel designed to convey the natural, bankfull discharge and an over-bank zone designed to convey additional water during floods up to a specified return period (based on partial duration or annual maximum series). The specification of two-stage channels for restoration design was beyond the scope of this study, but has been addressed subsequently (Thorne and Soar, 2000). It is recognised that in cases where the project stream is also a flood control channel it may not be feasible to construct the ideal main channel based on the channel-forming discharge.

The channel-forming discharge is the main driving variable for channel restoration design, however identification of bankfull stage from field indicators has been shown to be problematic and subjective (Williams, 1978). On this basis, the effective discharge, determined from magnitude-frequency analysis, may provide a more objective measure of the channel-forming flow. The effective discharge is the flow which transports the most sediment over a period of years (Andrews, 1980) based on flow frequency data and a sediment-rating curve. In stable, natural channels, bankfull, effective and channel-forming discharges have often been found to be roughly equivalent, although recent field research may cast doubt on this finding for sand-bed rivers (Soar, 2000). Research (Biedenharn *et al.*, 2000) has indicated that the best approach is to use several techniques to approximate the channel forming discharge.

PROJECT REACH ASSESSMENT

This assessment requires a survey of the available right-of-way (land take) for laying out the restored meandering channel. The available right-of-way is influenced by site constraints, such as floodplain constrictions and existing structures, and other project objectives that involve utilising the riparian and floodplain areas.

Data obtained during the supply reach assessment should be used to determine the target channel type, in terms of boundary materials, 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 imperative that target vegetation is identified prior to channel design as it influences flow resistance. If this is omitted, the stability of the channel could be affected. The target meandering type may be identified as: i) equiwidth meandering; ii) meandering with point bars; iii) meandering with point bars and chute channels.

CHANNEL DESIGN

There are three stages in designing the stable geometry for the restored, meandering channel:

i) Determine reach average dimensions and layout (bankfull width, bankfull depth, bed slope, sinuosity, wavelength and regular meander path);
ii) Design local morphological variability around meander bendways (including variable

width, location of pools and riffles, maximum scour depth in pools and adjustments to the layout to account for natural variability and site constraints);

iii) Fine-tune the initial design, based a channel stability assessment that matches reach sediment transport capacity to the supply from upstream (Soar, 2000).

In general, hydraulic geometry relationships between bankfull width and channel-forming discharge have been found to have the strongest correlation. On this basis, it is recommended that the relationship between channel-forming discharge and width be used as the starting point in determining the reach average channel dimensions. However, in specific cases other hydraulic geometry relationships such as channel slope to drainage area have been used with confidence. Rather than using a single regression equation to determine the stable or regime width, it is recommended that a range of widths within confidence limits be calculated from a hydraulic geometry equation appropriate to the type of target channel based on the nature of bed sediments and bank characteristics (Soar, 2000). Next, the remaining two design variables, depth and slope, should be determined from process-based equations that account for sediment transport and bed and bank resistance. The preferred technique is the Copeland analytical regime method (Copeland, 1994) that involves the simultaneous solution of flow resistance and sediment transport equations for the range of stable width, to yield a range of stable depth and slope. This method is a component of the hydraulic design package, SAM (Thomas *et al.*, 2000) for both sand-bed and gravel-bed rivers.

Once a stable bed slope has been determined, sinuosity is defined as the ratio of valley gradient to channel bed slope. Hey (1976) showed that for a given sinuosity an infinite number of meander patterns are possible and a determinate solution requires estimation of the meander wavelength. As wavelength is closely associated with width and only indirectly associated with discharge (Leopold and Wolman, 1957), it is recommended that a morphological relationship expressing wavelength as a function of width should be used in restoration design. A range of stable wavelengths should be determined within confidence limits to account for natural variability. Although there are a wealth of equations in the literature (e.g. Williams 1986) which predict other planform variables (arc length, radius of curvature, arc angle and meander belt width, etc.) they are redundant once wavelength and sinuosity have been determined as only these two parameters are necessary to layout a regular meander pattern.

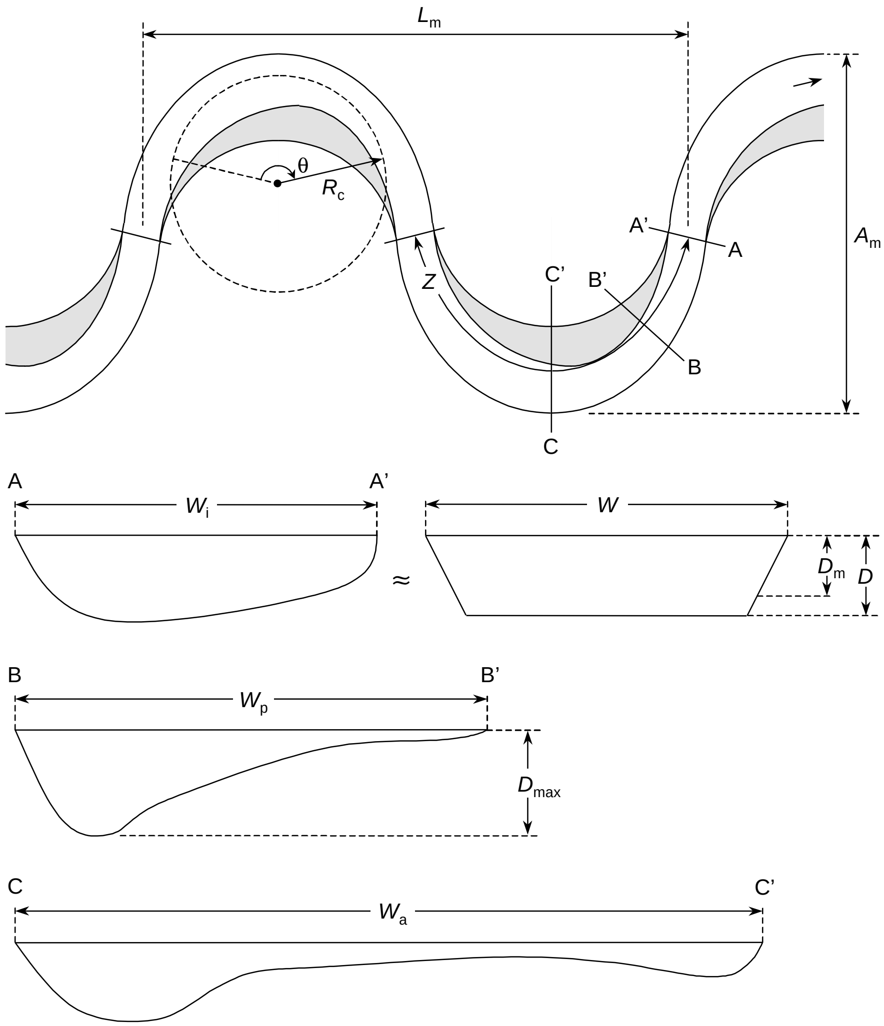
There is currently little design guidance for laying out the planform geometry of meandering channels. Detailed studies of meander bend patterns have revealed that regular meander paths are very rare in nature because most rivers exhibit considerable variability in meander form and orientation. In particular, simple geometric alignments fail to account for the downstream asymmetry in meander bends that is an essential feature in natural migrating rivers (Carson and Lapointe, 1983). However, on a reach-scale level, a regular meander shape is a reasonable average condition from which to design an initial channel alignment. The preferred geometric shape is the sine-generated curve proposed by Langbein and Leopold (1966) as the model is based on energy principles and accounts for the fact that meandering rivers often exhibit straight reaches in between bendways that are not provided by circular or parabolic curves. From the sine-generated curve, sinuosity and wavelength, the radius of curvature and required meander belt width can be determined. The sine-generated curve should be interpreted as a template or reach-average configuration and not a fixed solution for successive meander bends. In extreme cases, where the floodplain is very constricted, the desired meander belt width may exceed the available rights of way and grade control structures may be implemented to reduce the energy slope, thereby allowing a reduction in the design sinuosity. However, grade control structures are difficult to design, implement and maintain (Watson *et al.*, 1999) and so this solution should only be adopted when floodplain constraints dictate it.

Following reach-average channel design, modifications should be made to the cross section template and regular meander path to account for local morphological variability. Meander planform parameters are defined in Figure 3, together with cross sections at three significant locations around a meander bendway: i) the meander inflexion point with width, W_i; ii) the maximum scour location with width, W_p, and maximum scour depth, D_{max}, and; iii) the meander bend apex with width, W_a. These cross sections are shaped by the relative magnitudes of downstream and transverse velocities. To mimic natural meander form and process, variability in width between the apex and inflection point cross sections should be specified in the design of a restoration project. In an active meandering channel, the width and cross-sectional geometry vary systematically with location. Generally, width and cross-sectional asymmetry are greater at bend apices than crossings. The degree of width variability is minimal in equiwidth channels, but increases in meandering rivers with point bars and is greatest in meandering rivers with point bars and chute channels. The likely maximum scour depth in the bendway pools should also be specified and facilitates design of asymmetric cross sections at bends. Existing procedures for estimating width variability and bend scour depth have been enhanced by Soar (2000).

The final stage of the channel design is a channel stability assessment. Bank stability charts expressing critical bank height as a function of bank angle and sediment properties should be consulted to investigate the stability status of the bank-lines. If required, there are numerous bank protection methods available to the engineer. These methods are beyond the scope of this study and are well documented elsewhere (e.g. FISRWG, 1998; Biedenharn *et al.*, 1997). A sediment impact assessment is required at the end of the design procedure to: i) validate the efficacy of the restored channel geometry; ii) identify flows which may cause aggradation or degradation over the short term, and; iii) recommend fine tuning of the channel design to ensure that dynamic stability will be ensured over the medium- to long-term. The assessment involves calculation of the Capacity-Supply Ratio (CSR). Ideally, the sediment transport 'capacity' of the restored reach should exactly match the 'supply' of sediment from the stable reach upstream, that is the CSR should equal unity. In practice, the CSR is used to refine the initial design configuration, by making the adjustments necessary to bring its value close to unity and improve potential stability. Achieving an optimum CSR, within ten percent of unity, should ensure dynamic stability while allowing the river itself to create sedimentary forms and morphological features that cannot be engineered. If parity of supply and capacity cannot be achieved by adjusting the design parameters within confidence limits, it may be necessary to delicately adjust the slope until the CSR is within the optimum range.

Finally, a sensitivity analysis on discharge and sediment load should be undertaken to examine the potential sensitivity of the restored channel to changes in flow regime and/or watershed sediment inputs. In watersheds with predicted or projected changes in land use, results from the sensitivity testing could indicate appropriate levels of post-project monitoring and maintenance.

To complete the design procedure, a Design Brief should be produced comprising engineering drawings of the planform and typical cross sections using an appropriate Computer Aided Design (CAD) system, for use by site engineers and the construction contractor.



Note: point bars are 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.

Figure 3 Meander planform and cross section dimensions for restoration design.

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STREAM RESTORATION AND STABILIZATION STUDIES FOR HURRICANE MITCH RECOVERY PLAN IN NICARAGUA

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Abstract: Hurricane Mitch battered the Caribbean coast and parts of Honduras, Nicaragua, El Salvador, and Guatemala, in Central America October 27 through November 1, 1998. The storm is considered the most destructive hurricane in the history of the western hemisphere. The U.S. Army Corps of Engineers (USACE) received the mission from USAID to assist in the Hurricane Mitch Recovery Plan in July 1999. Nashville and Huntington district offices have assembled teams that are responsible for the recovery efforts in Nicaragua.

Hurricane Mitch severely affected Nicaragua, causing significant damage to agricultural lands, irrigation systems, levees, and other agricultural infrastructure. The river systems experienced extreme degradation and deposition, which caused the failure of many roads and bridges. The hurricane resulted in an estimated 2,863 deaths and left another 968 persons missing. About 90 percent of the deaths resulted from one event, the mudslide in Posoltega, a volcanic mudflow. The Central Bank of Nicaragua has estimated the cost of reconstructing damaged property and infrastructure at more than US $1.3 billion.

The Government of Nicaragua is preparing watershed reconstruction plans that will identify key actions necessary to reestablish productive activities and mitigate the effect of disasters in the future. The work plan includes watershed assessments for several basins, which will provide insight on potential flood hazards and opportunities for improvement of water utilization within the basin. These studies will include assessment of potential multi-purpose dam sites and a cursory channel stability and sediment assessment of aggradation / degradation tendencies. A Demonstration basin study will be performed on the Estero Real Basin, which will include detailed stream gradient, erosion, and bank stabilization analyses. The intent of the study is to involve local technical representatives in the process so they could perform similar studies in other basins as necessary.

This paper describes the status of the river mechanics and sedimentation studies being performed under the work plan including data collection, initial assessment, methods of analysis, and potential results.

INTRODUCTION

Observed Problems: Generally, the Government of Nicaragua (GON) has described a variety of problems throughout the country that need to be addressed. As a result of Mitch, rivers were significantly enlarged and re-aligned, villages were relocated and the transportation infrastructure was destroyed. The aftermath of Mitch has left river systems that exhibit totally different regimes from the pre-Mitch era. Villages and roadways were replaced or relocated using expedient methods without benefit of long-term watershed assessments. In many locations, rivers totally breached/departed from their historic alignment and new channels were formed. In other locations where the river maintained the original alignment, dramatic changes in channel geometry occurred. Bank stabilization measures are needed in the vicinity of villages to prevent future loss of roadways/structures/dwellings as the system seeks equilibrium for the altered conditions.

Many river crossings have been rebuilt with structures that were intended to be permanent. However, many of these sites are already experiencing stability problems. Materials and funding are available for construction of new bridges through groups such as the Catholic Relief Fund, Peace Corps, and other non-governmental organizations. Engineering analysis of proposed sites needs to be performed which incorporate appropriate hydrologic, hydraulic and sedimentation considerations prior to initiating construction. The following photographs depict typical channel and bridge conditions.

Typical Post Mitch Bridge

Post Mitch Rio Coco River

PROPOSED STUDIES

Estero Real Demonstration Basin: This basin experienced dynamic changes in river alignment and configuration as a result of Mitch. As the river system continues to seek equilibrium, channel stability, flow conditions and channel capacity remain unstable. As an example, in the vicinity of the community of Israel, the Pan-American Highway has been endangered due to changes in channel alignment and flow paths. The site in question has experienced a significant change in topography and a total change in the alignment of the primary conveyance of flows through this reach of the Villanueva River. As a direct result of Hurricane Mitch, a series of channel breaches occurred. One of these breaches has redirected discharges from the Villanueva River to an unnamed tributary that flows through Israel. This unnamed tributary has questionable outlet capacity, which results in much of the flow going into storage, causing flooding of the nearby village. The redirection of flow has stressed a single span bridge at the crossing of the Pan-American Highway in that it must currently (post-Mitch) convey the majority of flows of the Villanueva River. Previously, this same structure only handled small tributary discharges.

The proposed work associated with this project will be accomplished in multiple phases. Phase 1 will include an assessment of structures that need repair/protection under emergency conditions to stabilize and prevent failure. Structures such as the single-span bridge at Israel cannot withstand increased discharges that were caused by the altered river realignment without alteration and/or providing stone protection. Even with stone protection the channel will continue to degrade and enlarge, unless a closure is constructed at the breach site and the original channel alignment is stabilized. Preliminary designs consist of a closure dike across the Mitch-formed channel with adjoining stone slope protection to return discharges for normal flood events to the original Villanueva River channel. This will take most of the pressure from the unnamed tributary and divert the flows away from the Pan-American Highway. Concept designs will be provided for all endangered structures in the basin.

Phase 2 will provide a reach analysis of sediment transport and channel stability characteristics throughout the basin with the intent of identifying high-threat or high maintenance areas. The initial analysis will be performed using ***SAM, Hydraulic Design Package for Channels,*** to determine problem areas. Depending on the results of the initial analysis, a more detailed sediment transport model such as ***HEC-6, Scour and Deposition in Rivers and Reservoirs***, will be developed to evaluate the long-term stability of the existing and various alternative channel configurations. This basin was selected due to the significant changes that occurred in the system as a result of Mitch which were directly related to channel stability and sediment transport. The breaches in the vicinity of Israel offer an opportunity to demonstrate sediment transport analysis techniques and provide technology transfer to GON. The lower reach of the Villanueva River essentially disappears as the channel capacity decreases and sediment deposition increases. The model will also assess the advisability of attempting to recover this portion of the channel through dredging and provide some estimate of future maintenance requirements.

Phase 3 will consider streambank protection techniques similar to those included in the ***Streambank Stabilization Handbook,*** such as providing spur dikes constructed of rock or root wads, etc, to improve channel stability. Dredging requirements and projected maintenance for the identified depositional reaches will be addressed and alternatives to minimize impacts on the drainage system will be evaluated. The objectives of these studies will be to prevent future breaches within the system.

Estero Real Multi-purpose Dam Investigation: Agricultural lands in the lower reaches of the Estero Real basin are almost totally dependent on uncontrolled surface runoff for water supply and irrigation. There is a significant need to provide upland storage capability for water to be used during the dry season. Hydropower facilities and flood control detention storage do not exist in this area. This study will review previous studies, topographic information and water availability to determine the viability of providing one or more multi-purpose dams that could resolve many of these issues. Local hydropower capability would enhance future development in the basin. Water supply and irrigation during the dry season would meet the needs of the people and would increase production of agricultural lands. Providing detention flood control during the rainy season would promote development and prevent damages. As a part of this study, sedimentation issues will be addressed with respect to impacts on both the proposed reservoirs and downstream channel stability. Data will be collected from surveys of existing reservoir sites and comparison of typical land uses.

Rio Negro Basin: The initial work plan consists of a general watershed assessment of the basin along with cursory channel stability and sedimentation assessment. Future studies will include analyses of potential multi-purpose dam sites. A separate investigation will be performed concerning the potential realignment of the lower reach of the Rio Negro. During Mitch, a control weir failed and this reach of river changed alignment. The current alignment is not acceptable to Nicaragua or Honduras. Honduras is deprived of the water resource that supported the shrimp industry, while Nicaragua has experienced flooding of agricultural lands and deposition of river sands on prime agricultural properties. Both countries are dealing with the loss of the river as the historic control of national boundaries.

The weir structure that failed during Mitch must be replaced/rebuilt and a channel must be constructed to convey flows along the border to its original outlet. The stability of the new channel configuration will be of significant concern. The proposed channel configuration and alignment will be evaluated for stability from a river mechanics / sedimentation prospective. The intent of the analysis will be to design a reach of channel, which has similar transport characteristics to the upstream and downstream reaches for a broad range of flow conditions. The evaluation will be performed with SAM for the initial phase of the study. If conditions warrant, an HEC-6 model will be developed prior to preparation of final plans and specifications.

Several sites were observed during the initial reconnaissance of the Rio Negro basin that could serve as demonstration projects. These sites would provide guidance on design considerations that will enhance the survivability of roadway crossings by stabilizing flow conditions in the proximity of those structures. Two sites consist of new roadway crossings at El Pedernal, and Gualilica. The crossings are similar in design, with multiple culverts and single span bridge components. The structures are already experiencing downstream scour and the flow conveyance of the structures are reduced due to deposition upstream and poor channel alignments through the structures. The demonstration project would provide guidance for the use of training dikes and protection of structures from downstream scour. At the third site, El Platanares, near San Juan De Limay a school was lost to bank caving, villages were relocated, and roadways were washed out as a direct result of Mitch.

New Bridge Construction @ Gualilica

School Destroyed by Bank Caving

The proposed demonstration at this site would consist of training dikes to control channel alignment and typical bank protection details in the vicinity of villages, schools or other structures that warrant protection. The design components could be implemented in a manner that would provide roadways and dependable access to areas that are not currently available. This should also enhance the redevelopment of this area and provide valuable tools that could be utilized at other sites throughout Nicaragua.

Chinandega / Leon Watershed Assessment: This watershed consists of parallel basins that drain to the Pacific Ocean. Hurricane Mitch devastation in this area includes the site of the Casita landslide above Posoltega. The historic hydrologic and hydraulic information for this basin is no longer representative of current conditions. Dramatic changes in channel configuration, including total relocation along new alignments, were a direct result of the landslide and extreme runoff conditions created by Hurricane Mitch. Gaging information, rating curves and flood risk assessments for communities within the basin do not exist for post-Mitch conditions. Generally speaking, there are newly formed channels that have not reached a state of equilibrium with current flow regimes. This basin represents a different set of runoff characteristics from the other Mitch-affected areas due to different soil types associated with volcanic activity in the basin. The basin has experienced significant changes in land-use management practices when pre-Mitch and post-Mitch conditions are compared. There is a need to perform a watershed assessment to provide insight on potential flood hazards, and opportunities for improvement of water resource utilization within the basin. Assessment of land-use impacts, sedimentation and river stabilization requirements and flood risk assessment is critical to properly manage the water resources. The lower reaches of this basin drain to the ocean and impacts of sediment deposition on ports should be addressed. The potential for additional slides or severe sediment deposition associated with large ash fields on the upper slopes of Volcano Casita is a major concern as the rainy season approaches.

New Channel Cut by Mitch

Deepened Original Channel

The assessment will eventually constitute a complete watershed modeling effort using state-of-the art methodologies. The model will develop runoff hydrographs for a range of hypothetical rainfall events and route the computed discharges through the basin. The model will be calibrated, to the extent possible, to historic events. This may only be of limited value due to the dramatic changes in channel alignment and runoff characteristics under current conditions. Water surface profile models will be developed at critical locations within the basin and rating curves will be developed. By applying hypothetical rainfall frequency techniques, stage and discharge frequency information will be developed. Even though calibration data will be scarce, the basin model will provide a relative representation of flood volumes and frequencies at various points of interest. The initial model will serve as the base condition for evaluating the potential for improvements to the system that could be gained by constructing in-line sedimentation basins, which would retain both sediments and water. This study must be closely coordinated with U. S. Department of Agriculture (USDA) in regard to impacts of changing land-use management practices. Inundation mapping can be provided in communities for use in zoning and re-development studies.

The assessment model will be used to transfer technology to GON by providing training workshops on how the model was developed and how to use it. The model could eventually serve as the basis for providing flood warning to communities. Conceptually, design and installation of retention structures would enhance the ability to control both sediment and water to minimize damages and optimize use of the nations water resources.

Dam Safety Inspections/Spillway Adequacy/Spillway Repair: Nicaragua has four existing dams that survived Hurricane Mitch without losing pool. However, they did experience significant erosion of the emergency spillways due to head-cutting Three of these dams provide hydropower and temporary repairs have been implemented to continue operations, but it is doubtful that the spillways could withstand another discharge event. The present erosion has significantly reduced the length of erodable material between the channel and the reservoir rim and energy dissipaters or crest controls are no longer functional. GON has indicated interest in developing a comprehensive dam safety and dam inspection program. Assessment of hydrologic spillway adequacy and design of permanent repairs for existing structures will be provided as separate work elements.

Spillway Study and Repair at Lago de Apanas de Rio Grande de Tumas: The control sill, several downstream baffle-type energy dissipaters, and the end sill were completely severed, as the spillway eroded as a result of head cutting. The eroded channel is approximately 10-meters deep. Presently this emergency spillway is partially blocked by two parallel, post-Mitch dikes. These features were constructed immediately upstream of the spillway bridge with the intent of preventing spillway discharge except in the most severe storm events. A morning-glory type (primary) spillway is the only unrestricted discharge device that remains as originally constructed. The total discharge capacity of the emergency spillways appears to be substantially reduced due to the recently constructed dikes. Furthermore, it appears that this spillway cannot withstand another spillway flow without breaching the reservoir and losing the pool.

Downstream of Spillway Bridge

Post-Mitch Dike across Spillway

Spillway Study and Repair Lago La Virgen de Rio Viejo: At this site, the original structure was a "fuse plug" spillway. The fuse plug concept reportedly functioned as designed, and the dam did not overtop. However, during the spillway discharge, the exit portion of the spillway was severely eroded by head cutting and existing spillway lining and a roadway were destroyed. The eroded channel is approximately 20-meters deep. Presently, a rebuilt *pervious* rockfill dike essentially restores this emergency spillway to pre-Mitch conditions. This dike was constructed on the same alignment as the original fuse plug. Installation of gabion matting for channel protection is in progress. The intent is to allow some spillway discharge by flow through the pervious dike. For severe flooding, the lake level would overtop the dike and erode it down to some unknown level. The discharge capacity of the new spillways should be equal to the original design. Although this concept performed adequately during Mitch (i.e. the dam was not overtopped), it appears that another spillway discharge would erode up to, and possibly through the reservoir rim at the spillway.

Looking Downstream from Spillway

Looking Upstream from Headcut

CONCLUSION

The devastation caused by Hurricane Mitch produced a river system that has been dramatically altered to such an extent that after two years the channels remain in a continuous state of flux. The final state of equilibrium for this severely altered system will not be reached for decades. The Corps of Engineers should take advantage of the opportunity to apply state-of-the art analysis techniques to this river system that has been forced to respond to an extreme storm event. This effort represents a unique opportunity and challenge to the engineers on the recovery team.

The recovery efforts will require the application of engineering judgment and virtually every methodology available in the assessment of river mechanics and sedimentation problems. Decisions must be made regarding the applicability of techniques used in the United States to assess the dynamics of the system. Questions regarding use of appropriate technology to assess interim conditions should be considered on a case-by-case basis. The results of the proposed studies included in this recovery effort will provide valuable insight on the strengths and weaknesses of the tools currently available for the evaluation of river mechanics and sedimentation issues.

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