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Reliability of Sediment Transport Modeling for Shallow Flow on Initially Dry Areas

Douglas L. Hamilton, Consulting Hydrologic Engineer, 18101 Von Karman Ave., Suite 350, Irvine, CA 92715
Shou-Shan Fan, Special Assistant, Federal Energy Regulatory Commission, Washington, D.C.

ABSTRACT

Large floods may inundate areas of a river valley that are normally dry. When the water contains a significant amount of sediment or when scour and deposition processes are active, the prediction of flow behavior becomes complex. This is well known in the technical and regulatory communities. In fact, it is becoming commonplace for an agency to request that a *sediment transport study* be prepared as part of the identification and mitigation of flood hazards. This paper discusses two issues brought up by making such requests: The reliability of methods and the validity of results for the application of sediment transport principles to the analysis of flood hazards in regions where sediment transport processes affect shallow flow behavior.

Watersheds that yield large amounts of granular sediment often create depositional environments in the downstream regions. Over time, as these regions become populated, flood control facilities are designed and built. Such facilities may fail when the combined discharge of water and sediment exceeds the capacity of the system. Failure may also occur if bridge crossings become blocked with debris or if a channel experiences extreme sediment deposition. These types of events can be studied but there is disagreement on how may arise convincing

In order for computed results to be reliable and valid, several factors must be considered in the analysis of potential sedimentation and flooding situations. These factors include the potential failure locations; The possibility of sediment or debris within the source water blocking off preferential flow paths and impacting the receiving area; The effect of sediment on changing the characteristics of the fluid itself and thus the applicable resistance formulas. The primary purpose of this paper is to present a framework in which one can fully appreciate the results yielded from analytical methods that are applied to complex physical processes.

WHAT DO RELIABILITY AND VALIDITY MEAN?

Prior to discussing the points to consider in the application of analytical techniques, the following definitions of reliability and validity are offered.

> **Reliability** is the quality of a method yielding similar results for successive applications which are implemented by different people. Furthermore, reliable methods have the essential characteristic of confident use based on a tradition of successful applications.
>
> **Validity** is the quality of a result being relevant, meaningful, and verifiable. A result is relevant if it addresses the significant issues of the matter at hand. It is meaningful if it conveys information that can be the basis for specific actions or decisions. A result is verifiable if it is within ranges established by observed data, historical experience, and physical limitations.

Reliability applies to methods, models, or techniques. Validity applies to the results obtained from the application of such methods, models, or techniques. How can these definitions be used to evaluate a method or a result? In order to apply a definition to a given situation, criteria within a definition must include thresholds against which the case at hand can be evaluated. For example, a definition of unsafe driving may use vehicle speed as a criterion with 55 miles per hour as the threshold. To apply the definition, we would apply technology in order to measure the velocity of a moving vehicle. Once accomplished, we compare the measured velocity against the threshold value of 55. If the measured value exceeds 55 then the criterion is satisfied and the definition fits. It is doubtful, however, if there can be any single valued indicia of reliability or validity when applied to a sediment transport study. In fact, the practice has leaned toward adopting subjective criteria as the most appropriate ones, asking questions such as: Which

agency prepared the study?; Who worked on the study; or How do the results compare to those of other studies? These may, indeed, be the appropriate criteria to implement.

EFFECTS OF SEDIMENT ON THE BEHAVIOR OF THE FLUID

High concentrations of sediment and debris in flowing water can cause it to behave differently than clear water flows. Some of these differences, such as the unit weight, are quantitative in nature. Other differences, such as the vertical velocity distribution for a debris flow, display qualitative differences compared to clear water. These differences and their effect on the behavior of the fluid are discussed here.

Classification

The literature contains several classification procedures for distinguishing between the different type of sedimentation hazards. Most of these use the volumetric concentration of sediment in the fluid (Kurdin, 1973; Campbell, 1985). Other procedures such as described in Miranova and Yablonskiy (1992) are based on the incipient cause of the flow such as a landslide, a volcanic eruption, a dam failure, etc. For flood investigations, the primary events of concern can be generally grouped into the following three categories: clear-water flows which can be analyzed with traditional hydraulic methods, hyperconcentrated sediment flows which can be analyzed to a great extent by sediment transport theory, and debris flows which can be assessed by various empirical methods such as the bulking factor, the Bingham model, etc.

Vertical Velocity Distribution

For turbulent clear water flows, the vertical velocity distribution is essentially a function of the logarithm of the ratio of the distance above the bed to the total depth of flow (Rouse, 1936). Vanoni (1953) suggested that as the suspended sediment concentration increased, the friction factor f decreased in association with a decrease in the von Karman coefficient. Figure 1 shows the comparison of two measured velocity profiles. The one on the left is for clear water. The one on the right is for a flow with a suspended sediment concentration of 15.8 g/l. The depth averaged velocity for the clear water flow is about 70 cm/s. Under the same depth and slope, the flow with high suspended sediment load has a depth averaged velocity of approximately 93 cm/s. Although their is some debate as to the actual cause, it is important to recognize that high sediment concentrations can result in flows with a higher average velocity compared to clear water.

Takahashi (1978, 1980) developed equations of motion for a debris flow. He determined the vertical velocity distribution obeys the relationship:

$$u = K[h^{3/2} - (h-y)^{3/2}] \tag{1}$$

where u is the velocity at a distance y above the bed, h is the total depth of flow, and K is a lumped parameter that describe site specific geometric and fluid properties. Flume experiments were conducted that indicated this velocity distribution had better agreement than that of typical laminar Newtonian flow.

Zhang and Ren (1982) conducted similar flume experiments using higher sediment concentrations. Their studies focus on viscous flows with low Reynolds numbers. The experiments indicate that as suspended sediment concentration increases, the top layer of the flow behaves as a rigid plug flowing over a relatively narrow shearing zone that is closer to the bed. Many of these experiments were for flows in the laminar regime. The existence of a flowing plug on the top layer would explain the observation of non-rotating boulders that apparently *float* on the surface of a debris flow. Higher velocities compared to equivalent clear water flows were also reported.

Umeyama and Gerritsen (1992) present a theoretical model for the velocity distribution in sediment laden flow assuming that the von Karman coefficient remains at its traditional value of 0.4. The work suggests a modified method of determining the Prandtl mixing length can better explain the results of flume experiments. The research concluded that mixing of water and suspended sediments in the turbulent boundary layer is less effective. The meaning of this conclusion (as with the earlier work) is that, up to a point, higher sediment concentrations result in faster moving flow.

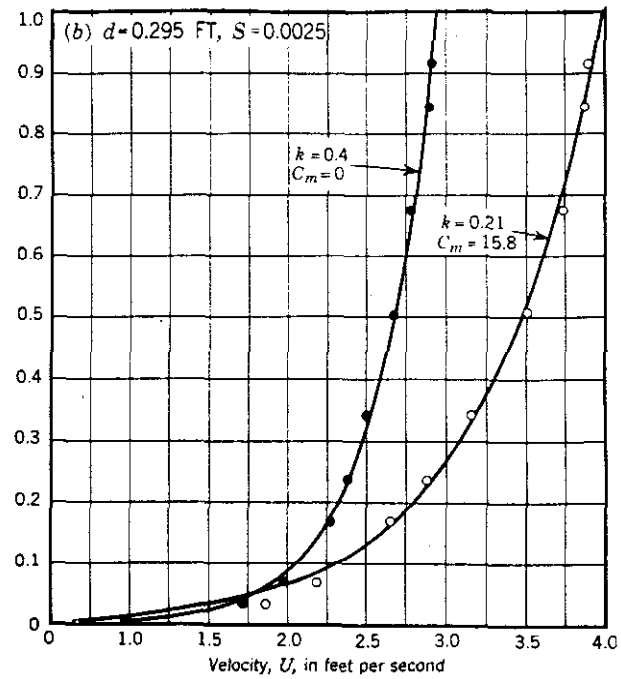

Figure 1. Vertical velocity distributions from Vanoni's (1953) flume studies.

Rickenmann (1991) performed a series of flume studies using clay suspensions in a recirculating flume. Although his work did not focus on the determination of the velocity distribution, he concluded that an increase in bulk fluid density increased bed load sediment transport rate. The bed load transport rate began to decrease once the clay content of the suspension exceeded 17%.

The laboratory flume studies of Lyn (1991, 1992) conclude that examination of the velocity profile alone is not sufficient to conclude that overall flow resistance is always increased or decreased in comparison to the equivalent clear water flow. In fact, near-bed velocities in many sediment laden flow experiments were lower than for equivalent clear water flows. In these cases the velocity in the upper layers of flow as well as the depth averaged velocity was always higher than clear water flow.

Even though there remains debate as to the exact relationship between the friction factor and the vertical velocity distribution, there are two points of interest for use when modeling flows in one or two dimensions. First, the depth averaged velocity increases when flows contains high concentrations of suspended sediment load. Even though it may not be entirely correct to attribute this increase to a reduction in flow resistance, the net effect of this for most engineering or risk assessment purposes is the same as if it was entirely correct. Second, high suspended load concentrations can result in increased bed load transport capacity.

Sediment Transport Characteristics

The presence of high sediment concentrations in flood waters can have other effects besides the increased transport of bed load sediments. Bagnold (1954) recounts a *river of gravel* flowing through a mountain canyon with no apparent interstitial fluid. He explains it using the concept of dispersive stress which results from the collision of

individuals particles in the flow. These collisions result in the larger particles being transported on the top of the flow rather than rolling on the bed.

Wang and Zhang (1990) investigated the interaction of clay suspensions flowing over a gravel bed. Although their investigation was primarily concerned with the behavior of debris flows, they arrived at several conclusions related to the types of sediment transport that may occur. The front of a debris flow often travels as a bore or shock and often carries large boulders. The trunk zone behind the bore carries coarse sediments (that are in suspension due to inter-granular collision) at a higher velocity than the bore itself. The coarse material rolls over the debris flow front continuously providing energy to it. This is one explanation for the presence of coarser material at the front of a debris flow.

Li, et al. (1983) describes field investigations of debris flows in the Jiang-Jia Ravine in Yunnan, China. On a field scale, the process described above results in a series of waves or surges each transporting large amounts of coarse sediment. Boulders larger than 6 meters in diameter have been carried downstream by this process. Wang and Zhang (1990) present a reason for the sorting process which results in the coarser material residing on the surface of the debris flow front. Resistance to the motion of a given particle in a debris flow is primarily due to collisions with materials on the bed. When a large particle collides with smaller bed material particles, its forward momentum remains relatively unchanged. On the other hand, when a smaller particles collides with materials on the bed it may completely lose its forward momentum. Large particles, therefore, have a much higher tractive force than resistive force and therefore tend to move faster, concentrating at the front of the debris flow.

Colby (1964) presents a procedure for estimating the effect of high suspended load concentrations on the bed load transport rate. His work is based on the Einstein bed load function and various sources of measured sediment transport data from both natural streams and laboratory flumes.

The sediment transport potential and flow characteristics of a flood event can be affected by changes in the bulk fluid properties. Traditional bed load transport equations and hydraulic calculations may need to be supplemented with other methods in order to adequately describe the flow behavior.

Turbulent v. Laminar Regime

Most analysis of flooding events relies on theory developed for full turbulent flows. In these cases, resistance to flow is determined primarily by the roughness characteristics of the flow surface. Flows with high concentrations of sediment, however, can often be dominated by viscous rather than inertial forces. In this case the properties of the sediment-water mixture deviate significantly from clear water. The fluid may possess both a finite yield strength and a non-linear relationship between the shear stress and shearing rate. Bingham (1922) proposed the following model:

$$\tau = \tau_y + \eta\left(\frac{du}{dy}\right) \qquad (2)$$

where τ is the shear stress in the fluid; τ_y is the Bingham yield strength; η is the viscosity; and du/dy is the velocity gradient in the primary flow direction. More complex models have been developed by Chen (1988) and Julien and Lan (1991) which also consider dispersive and turbulent stresses.

Zhang S. (1992) indicates that the potentially most destructive sediment flow event is the viscous debris flow. A schematic representation of this type of event is shown in Figure 2. Levees often form on the perimeter of the debris flow which help to channelize it (Figure 2a.). Deposition lobes often form at the end of these channels. Slopes in excess of 4% may result in flow surges as the debris travels downstream (Figure 2b) while milder slopes result in debris flows of uniform thickness (Figure 2c). Deposits are sometimes left along the edges of a debris flow (Figure 2d). Large boulders can be supported and carried by the frontal surge of the flow (Figure 2e). Figure 3 shows a post-event photograph of a viscous debris flow in gravelly material.

Assuming the top layer of a viscous debris flow behaves as a rigid plug, Zhang S. developed a method to estimate the potential flow velocity (Hamilton, et al., 1993). This method uses a revised version of Equation 2:

$$\frac{u_{max}-u}{u_{max}} \approx (1-\frac{\tau}{\tau_0-\tau_y})^2 \tag{3}$$

Where u_{max} is the velocity of the plug layer; and τ_0 is the shear stress on the stream bed. Integrating this expression gives:

$$V=\frac{\tau_0 h}{3\eta}[1-\frac{3}{2}(\frac{\tau_y}{\tau_0})+\frac{1}{2}(\frac{\tau_y}{\tau_0})^3] \tag{4}$$

By noting the $\tau_0=\gamma RS_f$ it can be shown the mean velocity of a one dimensional debris flow is given by:

$$V=\frac{R}{\eta}(\frac{32\gamma RS_f}{M}-\frac{\tau_y}{2}) \tag{5}$$

The above equation is valid for viscous debris flows in the laminar regime.

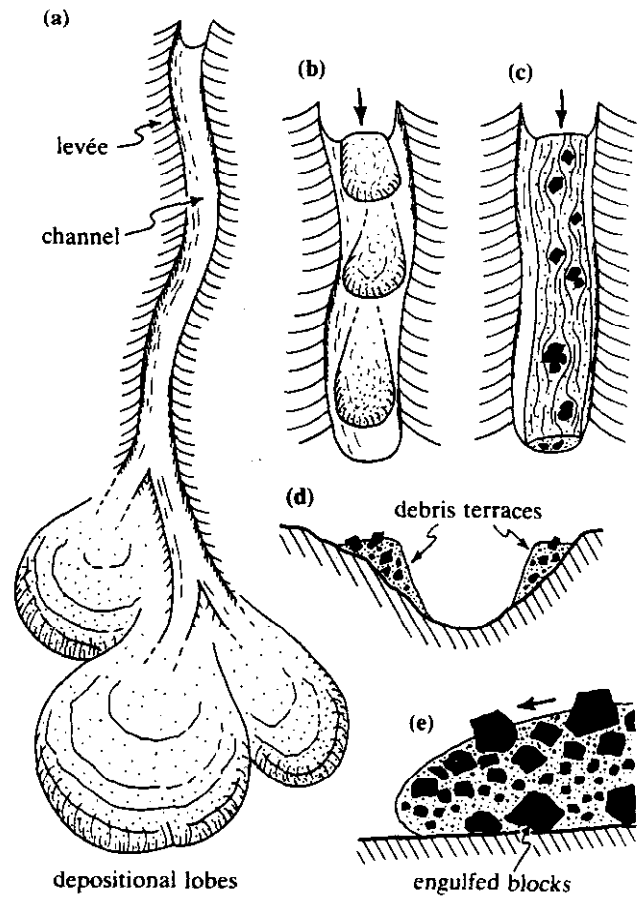

Figure 2. Debris Flow Characteristics (Source: Allen, J.R.L., 1985, Principles of Physical Sedimentology, Chapman and Hall, London.

EFFECTS OF SEDIMENT ON THE INITIALLY DRY AREA.

Erosion Process

Sediment transport processes can drastically change the behavior of a flood both in a defined channel and after it leaves the channel. During an event, the erosion process may have the effect of augmenting or *bulking* the hydrograph so that peak flows are significantly larger than those estimated from rainfall intensities. Furthermore, the sediment deposition process may alter the geometry that governs the flow depth, velocity, and direction in the initially dry area.

Blackwelder (1928) describes how steep channels in semi-arid mountains create ideal mixing troughs for water and sediment. During a heavy rainfall season the sides of steep canyons become saturated, fail, and slide into the stream channel. Runoff from subsequent events picks up this loose, saturated material and conveys it downstream. The increased bulk density creates favorable conditions for picking up additional sediment stored in the stream bed. The result is a debris flow that has a larger volume and peak flow than one can attribute solely to runoff from its coincident rainfall event. Examples of similar occurrences are discussed in Marsell (1970), Slosson (1986), and Anderson (1984).

Tagomori (1988) describes the New Year's Eve 1987 flood in the eastern part of Oahu, Hawaii. A storm dropped as much as 60 cm of rain in a 24 hour period with intensities exceeding 10 cm/hr. The Hahaione Valley channel eroded wider and deeper to such an extent that an existing settling basin was quickly filled, blocking a culvert entrance. Flows left the constructed channel and were conveyed through downhill sloping streets. Shearing forces created by the diverted flows were so great that they ripped up slabs of asphalt pavement and eroded a channel approximately 2 meters deep and 5 meters wide inside the street alignment. Such changes in geometry are difficult to predict from a computational standpoint. They do, however, have a significant effect on the flood behavior in the previously dry area.

MacArthur et al. (1991) indicate that the peak flows of the 1987/88 Oahu event described above were initially characterized as having 500-year recurrence interval. Landslides in the upper watershed and saturated soil scoured from the channel bed contributed to the severity of these flows. Absent this bulking effect, it was later determined that the flows attributable to direct runoff from rainfall had a recurrence interval of 20 to 50 years. The point of this narrative is that sedimentation processes may affect the design hydrograph which is an essential boundary condition for many flow modeling efforts.

The bulking factor is a common means of incorporating the potential effects of watershed erosion upon the design hydrograph. For example, assuming that the maximum concentration by volume, C_v, of a debris flow is 0.5, the runoff attributable to rainfall may be multiplied by a factor of $1/(1-C_v)$ or 2 to come up with a hydrograph that represents the effects of a debris flow. This approach is commonly used as a safety factor for hydraulic calculations done for the design of flood control facilities. Quite unfortunately its misuse is also becoming common.

Volcanic deposits, burned watersheds, and landslide blockages can create the potential for extreme flow events that cannot be characterized by the simple assessment of maximum concentration (Guy, 1970; Hamilton, et al., 1986; MacArthur, et al., 1990; Pierson, et al., 1992; and Slosson, et al., 1986). Erosional processes often mobilize additional water that saturates the pores of underlying strata. Schaefer (1992) demonstrates that for certain events, the *ultimate* bulking factor can range from 1.1 to more than 10.0. He developed the following relationship:

$$BF_{ult} = \frac{1+\phi}{1-\phi\theta} \qquad (6)$$

where ϕ is the ratio $1/(1-C_v)$ with C_v being the maximum expected concentration by volume of the event; and θ is the effective void ratio (the product of the void ratio and the fraction of voids filled with water). Equation (8) was used to estimate a bulking factor of 3.32 for peak flows from a potential failure of a landslide lake blockage near Mount St. Helens in the U.S. (MacArthur, et al., 1990).

On the other side of the spectrum, the bulking factor is sometimes incorrectly used as a calibration factor for bed and suspended load calculations. Since its primary purpose is to introduce a factor of safety into the hydraulic design of a facility, it will not necessarily match with observed or computed sediment transport data. Furthermore it may distract one from the more significant sedimentation issues at hand (Saarinen et al., 1984).

Deposition Process

When considering the impacts of floods on initially dry areas, the process of sediment deposition can influence the direction and extent of flood flows. The alluvial fan, which is itself the product of deposition, illustrates this point. Often, the existing channel or preferential flow path is under capacity compared to the design event. When the design event occurs, it is difficult to determine in advance whether or not the flow will proceed along its historical course or seek a new direction. Two dimensional flow models have been used to analyze such cases but their applicability is limited. Since the boundaries between flooded and dry areas are not know a priori, solving the two-dimensional shallow water equations involves what amounts to a moving internal boundary condition. Furthermore, the transcritical flow conditions that characterize shallow flooding have not been readily incorporated into existing computer models.

CLOSURE

The application of clear-water, fixed boundary hydraulic models to many shallow flooding situations may not adequately characterize the overall flood behavior. However, merely requesting additional studies or developing more complex models that attempt to quantify the effects of sedimentation may not necessarily be the complete solution to our dilemma. Modeling applications combined with an adequate appreciation for the importance of both hydraulic and sedimentation processes are essential elements of achieving reliable and valid results.

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RIVER MECHANICS EXPERIENCE ON THE MIDDLE RIO GRANDE

Drew C. Baird, River Analysis Team Leader, U. S. Bureau of Reclamation, Albuquerque New Mexico.

INTRODUCTION

The alluvial 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. Attempts have been made for many hundreds of years to divert the Rio Grande by brush and rock dams for irrigation, and the existing diversion dams were completed by the Middle Rio Grande Conservancy District (MRGCD) in 1936. Most of the activities discussed in this paper have taken place during the period from 1955 to present. After the formation of the MRGCD, soil levees (from riverside drain excavation) were constructed parallel to the river. These levees were not engineered to be effective against erosion resulting from direct flows of the river. MRGCD installed sediment fences, encouraged vegetation growth and attempted to maintain the levees to provide flood protection to the adjacent lands. However, 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, by the late 1940's, the MRGCD sought assistance from the Federal government. This assistance was provided by Reclamation and the Corps of Engineers (Corps) through the Middle Rio Grande Project authorized by the Flood Control Acts of 1948, 1950 and 1960. The purpose of the authorized work is to efficiently transport water through nearly 300 miles of the Middle Rio Grande Valley, under the Rio Grande Compact and to Mexico under International Treaty. To reverse the river bed aggradation trend through sediment detention/flood control structures, to improve land drainage, and increase the number of irrigated acres. To protect lives and property along the Middle Rio Grande including irrigation facilities, agricultural, Native American, and other improved lands. Protect Federal and State Wildlife Refuges, and Bandelier National Monument.

The two agencies set out to accomplish the assignment guided by a plan that emphasized the use of large dams and channel rectification to control floods and sedimentation. Thus, it was planned that the reduced sediment inflow combined with regulated releases would result in sufficient stream carrying capacity to transport all uncontrolled inflowing sediment and to pick up sediment from the stream bed. The resulting degradation, together with other changes planned, was to have brought about the desired effect in the valley. Among the channel rectification measures, use of Kellner jetties became the primary means of providing bank protection. The purpose of this paper is to document river channel geomorphology before and after reservoir construction, past and present river channel improvement works particularly the period from 1984 through 1994, and to present a brief plan for future river improvement works.

HISTORICAL RIVER GEOMORPHOLOGY

The Rio Grande originates near the Continental Divide in South Central Colorado; bisects the State of New Mexico from North to South; then forms the international boundary between Texas (USA) and the Republic of Mexico on its journey to the Gulf of Mexico. The location of the works described in this paper is the central portion of the Rio Grande Valley in New Mexico from Velarde, New Mexico, to Caballo Reservoir. Figure 1 shows the general location and the various river divisions.

The Middle Rio Grande Valley physical features vary widely. Mountains delineate the basin at upper elevations, while the valley floor lies surrounded by desert. The character of the drainage basin changes from mountainous to plateau at the head of the Cochiti Division. Ephemeral tributaries of the Rio Grande are entrenched in the plateau area.

As in most similar streams, which have large variations of discharge from year to year, the Rio Grande transports a widely varying range of water and sediment. Streamflow records on the Rio Grande began in 1889. Large floods occurred in 1941, 1927, 1920, 1884, 1874. These floods occurred before significant reservoirs existed on the upstream reaches. Floods result from a large snowpack which upon melting in the

spring or summer results in large volumes and discharges of water or summer thunderstorms which may generate a large discharge, but usually do not contain a large volume. Both types of floods can move tremendous amounts of sediment.

The afore described flood events resulted in the deposit of significant amounts of sediment. The maximum aggradation usually occurred in the mouths of tributaries which flowed into the Rio Grande. These delta deposits were then flushed downstream to be redeposited at locations where the sediment transport capability of the Rio Grande diminished. The tremendous sediment load resulted from the native soil types, the lack of vegetation cover, and the arid climate. Over grazing and drought in many areas of the watershed increased the potential for erosion due to the unstable land masses. It is reported that the Middle Rio Grande Valley in 1800 had about 100,000 acres in irrigated cultivation; in 1880, 125,000 acres; and in 1925, 40,000 acres. This striking reduction in acreage was due in a large part to sedimentation and consequent waterlogging of lands adjacent to the river (MRGCD, 1928).

The Middle Rio Grande has one of the highest sediment loads of any river in the world. In a study conducted by Janson, et. al., (1979) the average sediment concentration of the major rivers in the world were compared. Of the 38 rivers compared, 27 have mean sediment concentrations less than 1000 mg/l. According to Janson et. al. (1979) the four rivers with the highest sediment concentrations are: (1) Hwang Ho (Yellow river) (China) 15,000 mg/l, (2) Waipapa (New Zealand) 7,500 mg/l, (3) Ganges (India) 3,600 mg/l, and (4) Missouri (USA) 3,200 mg/l. The rate of annual sediment inflow into Elephant Butte has been as high 24,000 mg/l during the 1950's to the mid 1960's, about 13,000 mg/l during 1966-1977, and currently is about 5,000 mg/l. Individual years have had average sediment concentrations as high as about 200,000 mg/l. Currently, about half of the sediment inflow into Elephant Butte is silt and clay and half, sand sizes, with the inflow of silt and clay amounting to approximately 2,500,000 tons per year (Slater and Baird, 1991). The river has been aggrading due to the high sediment loads for about the last 11,000 to 22,000 years (Leopold et. al. 1964, Hawley et. al. 1976) and most recently during this century (Bureau of Reclamation, 1967).

Prior to measurable human influence on the system, up to the 14th century (Biella and Chapman 1977), the river was an aggrading channel with a shifting sand substrate. Its pattern was, as a rule, braided, relatively straight, or slightly sinuous. The river would migrate across the floodplain, the extent being limited only by the valley terraces and bedrock outcroppings. The Rio Grande's bed would aggrade over time; then, in response to a hydrologic event or series of events, it would leave its elevated channel and establish a new course at a lower elevation in the valley. This process is called river avulsion (Leopold et al. 1964). Although an aggrading system, there were periods of stability that allowed riparian vegetation to become established on riverbanks and islands alternating with periods of instability (e.g., extreme flooding) that provided, by erosion and deposition, new locations for riparian vegetation.

An analysis of the total valley flow on an annual basis at the San Marcial Gage (See Figure 1) shows that from 1896-1945 there was an average of 1,110,000 acre-ft of water, and from 1946-1978 there was 573,000 acre-ft, and from 1979-1993 1,095,000 acre-ft. This was a significant reduction in available water supply during the period from the 1940's to the 1970's.

RIVER CHANNEL WORKS

The works of MRGCD completed in 1936, included El Vado Reservoir, four major diversion dams, two canal headings, and over 1,500 miles of canals and laterals, drains, and levees. While the works were a valuable aid to irrigation, the irrigators continued to be plagued with problems of water shortages, floods, a high water table, and sediment deposits in canals and laterals.

Jemez Canyon Dam was completed in 1953 to provide flood and sediment control near the mouth of the Jemez River, approximately 25 miles above Albuquerque. Abiquiu Dam was completed in 1963 to provide flood and sediment control on the Rio Chama, the major tributary of the Rio Grande in New Mexico. Cochiti Dam, which began to store water in 1973 and completed in 1975, affords flow and sediment control on the

Rio Grande primarily above the mouth of the Rio Puerco. In addition to flood and sediment control, the reservoir contains a permanent recreation pool. Galisteo Dam completed in 1970, provides control of sediment inflow from Galisteo Creek which empties into the Rio Grande about 13 miles above Angostura Diversion Dam.

In the 98 miles from Cochiti Reservoir to above the mouth of the Rio Puerco, urban and rural developments lie along both sides of the Rio Grande except where they are precluded by natural topographic features. Channelization of this reach consists of a cleared channel with physical characteristics compatible with hydraulic and sediment transport requirements determined through data collection and statistical analysis (Schembera, 1963). The channel is mostly defined by permeable steel jetties, in combination with existing native vegetation and natural topographic features. Where it was deemed necessary, the realigned channel was cleared of native vegetation and relocation channels were excavated through high points in the new alignment. In the Cochiti Division, where Indian interests desired a more natural channel, limited pilot channel work was done, nor were Kellner jacks installed extensively.

Kellner jetty systems were investigated by Reclamation (1961, 1962). They used a prototype study area on the Middle Rio Grande and a hydraulic model study to define and develop design parameters. Throughout the Albuquerque and Belen divisions channelization works included establishing banks using Kellner jacks (Woodson, 1961). In the Espanola, Cochiti, and Socorro Divisions channelization included some channel straightening, and relocation channel construction without any bank stabilization.

During the severe and prolonged drought period between the 1940's and the 1970's the river channel between San Marcial and the narrows of Elephant Butte Reservoir became totally obliterated by sediment deposition and phreatophyte growth. The reservoir pool at Elephant Butte shrank to about 30,000 acre-ft. The water loss between San Marcial and the reservoir pool was estimated to be 140,000 acre-ft annually. In response to this drought condition the Rio Grande Conveyance Channel was constructed into the upper 15 miles of Elephant Butte Reservoir, between 1951 through 1959. The channel now extends an additional 60 miles upstream of the reservoir. Annual water conservation amounts to about 60,000 acre-ft. when the Conveyance Channel is in full operation.

POST DAM CHARACTERISTICS

The improvements made in the basin has initiated channel degradation and improved water and sediment transport efficiency, a major goal of the Congressional authorizations. This is particularly true from Cochiti Dam south to the mouth of the Rio Puerco. From the Rio Puerco to Elephant Butte the rate of aggradation has been reduced, but still continues. River bed slopes range from about 0.001 near Velarde NM to about 0.0005 ft/ft in the headwaters of Elephant Butte Reservoir. The average bed material sizes range from a high of 85 mm near Velarde NM, to 0.2 mm in the headwaters of Elephant Butte Reservoir. Suspended sediment loads range from 770,000 tons per year at the Albuquerque Gage to about 4,000,000 at San Acacia and San Marcial, between 1978 and 1993. Since the closure of Cochiti Dam the sediment load at Albuquerque gage has dropped considerably with most sediment coming from upstream ephemeral tributaries and the main channel. Between 1970 and 1983 the bed of the river degraded an average of 3 ft in the Albuquerque area. Between Bernardo and San Acacia the average increase in suspended sediment load is 3,200,000 tons/yr with about 2,400,000 tons/yr coming from the Rio Puerco.

The Espanola Division, upstream of Cochiti Dam (See figure 1) has changed from a braided, slightly sinous system to a single channel system with unprotected banks as a result of channelization and Dam construction on the Rio Chama. River slopes ranges from .001 to .0028 ft/ft. and bankfull discharge is about 5,000 above the mouth of the Rio Chama and about 8,000 below the Rio Chama to Cochiti Dam. River straightening in some areas increased the slope by shortening the channel length. There are high rates of bank erosion and channel migration in the Espanola Division. Cochiti Division slope ranges from 0.0013 to about .0021 ft/ft. After construction of Cochiti Dam and resulting degradation, the river changed from a sand bed to a gravel and cobble bed channel. The channel has sought to become longer through bank erosion and meandering in many areas as degradation has progressed over time. The bed of the river changes from gravel to sand in the

northern part of the Albuquerque Division. In the Albuquerque and Belen Divisions there are numerous islands and alternate bars that are forming as the channel is narrowing as a result of the reduced peak flows, and bed material load. The Socorro Division from San Acacia Diversion Dam to the headwaters of Elephant Butte is still an aggrading channel, with fine sand bed material, low banks, and a relatively straight alignment with low sinuosity. There have been several channel avulsions in the last ten years in this reach.

Regulated, and unregulated peak flow frequency analysis was done by Bullard and Lane (1993) on the Middle Rio Grande. At the Otowi gage the unregulated 2 and 5 yr. return period peak flow was 9,770, and 14,550 cfs respectively, while the regulated peaks are 8,050, and 11,990 cfs respectively. At the Albuquerque Gage downstream of Cochiti Dam the unregulated 2 and 5 yr. return period peak flow was 10,650, and 15,110 cfs respectively, while the regulated peaks are 5,830, and 7,660 respectively.

PAST AND PRESENT RIVER CHANNEL MAINTENANCE

During the construction phase of the project, the river channel was constructed from a braided and mildly sinuous channel into a fairly straight alignment with erodible banks unprotected in the Espanola and Cochiti Divisions. The constructed channel cross section was generally trapezoidal with 10:1 side slopes. Islands were mostly removed. The unprotected banks eroded annually in some cases to a curved alignment. Maintenance consisted of moving bar material across the river placing it on the eroded banks (pilot channeling). Thereby re-establishing the fairly straight trapezoidal channel, with 10:1 side slopes. This type of channel maintenance took place annually in many areas. In addition, islands were removed. These procedures kept the river channel well away from levees, and riverside and irrigation facilities in most cases. This was the approach of that era on many rivers throughout the world (Parker and Andres, 1976). These practices however, did not contribute to the long-term stability of the river channel and resulted in constant maintenance and pilot channel reconstruction being required in some areas. Areas where pilot channels were built often eroded back to a more curved configuration after high flows. The last time pilot channel reconstruction work was done was in 1985.

During the late 1950's and early 1960's researchers in the field of geomorphology began to investigate the patterns of river channels. Leopold and Wolman (1957) determined that rivers have three basic planform shapes: meanders, braids, and straight reaches. Meandering and braided rivers can accommodate a wide range of flows, and have riffles and pools which provide aquatic habitat. Rivers adopt these planforms to adjust their length, width, and depth, in relation to the local geology, valley slope, bed and bank material composition, and sediment load. Schumm (1977) reports that when the sediment load decreases in a river there is an increased tendency for meandering. This is indeed the case for the Middle Rio Grande.

River straightening or channelization as was done on the Middle Rio Grande has been known to have adverse impacts (Parker and Andres, 1976). Nunnally and Keller (1979) identified several general problems with past channelization practices. Channelization can cause destruction of riparian vegetation, reduction or elimination of pools and riffles, and a reduction in aquatic habitat. Channelization offers no erosion protection from high flow velocities during bank-full discharges, and results in increased bank erosion from increased stream slope.

Nunnally and Keller (1979), and Vanoni (1975) advocated stabilizing the river banks and preserving the existing river planform as much as possible in maintenance activities, while providing erosion protection for the outside of bends. Nunnally and Keller (1979) gave several advantages of this approach: 1) destruction of trees and other vegetation is minimized, 2) river banks are more stable because gradients are not increased, and the river can accommodate wider ranges of flow without increased erosion, 3) stream reaches are almost never straightened or relocated, however there is sometimes the need to remove constrictions or bank obstructions and to narrow the stream where erosion has produced an overly wide channel, 4) during construction there is minimal disturbance of the streambed or to vegetation except where heavy undergrowth is removed, but trees and saplings often are left undisturbed.

The final result using these procedures is a more stable stream with aesthetic value. Pool and riffle sequences associated with meanders recover quickly after high flow events and there is better habitat and higher fish and wildlife populations than on channelized streams (Nunnally and Keller, 1979). In addition there is improved water quality through reduced turbidity from bank erosion, after the projects are completed. Many reaches of the Middle Rio Grande are not in a state of dynamic equilibrium. The Cochiti and Albuquerque divisions are degrading and the Socorro division is aggrading. However, some of the general principles of geomorphology have been successfully used.

Beginning in 1985, a new approach was implemented. Factors in the choice of river maintenance prior to 1985 were efficient water and sediment transport through relatively straight channels; and keeping the river channel away from riverside facilities. The factors used between 1985 and 1994 to determine the choice of river maintenance activities are Section 404 and Section 401 permit requirements, aquatic and riparian habitat, State Water Quality Standards, post reservoir river conditions, endangered species, efficient water and sediment transport and protection of riverside facilities. Instead of annually removing bars and islands and re-establishing the trapezoidal channel, the river channel was allowed to erode and meander. Until the erosion of unprotected banks threatened levees, agricultural lands, residential lands, and irrigation and other riverside facilities (Note the levees are unprotected as well as the eroding banks). When erosion reaches this level, the outside of riverbends are stabilized with riprap, groins or Kelner Jetties while preserving the existing river shape as much as possible. This has allowed fluvial processes to take place to the maximum extent within the physical confines of the levee system and other riverside facilities. There are several advantages to this approach: 1) stabilizing bends in nearly the same shape as is existing reduces the cost of maintenance, these stabilized banks generally pay for themselves in 7 to 15 years through reduced annual maintenance costs, 2) river banks are more stable and the river can accommodate wider ranges of flow without increased erosion, 3) pool and riffle sequences associated with meanders recover quickly after high flow events and there is good fish utilization of riprap stabilized banks (Baird et. al, 1993). An economic analysis is performed on all maintenance work to determine the lowest cost alternative. Most areas maintained using this approach in the Espanola Division have passed the design peak flow without any damage.

Velocity and scour in bends in the Espanola and Cochiti Divisions has been estimated using the approach documented by Baird and Achterberg, (1989, 1991). This methodology using stream tubes and the Blench equation has been successfully used and is simple to apply.

Some rivers (Vanoni, 1975) are stabilized with bend radius divided by the width (radius/width ratio) of 10. On the Middle Rio Grande there has been success leaving curves at or near the natural radius/width ratio at values of between 2 and 3. This approach has reduced significantly the changes to the river channel, and the amount of work required (and hence cost). Designs on other rivers (Vanoni, 1975) indicate that about two-thirds of both banks should be stabilized for optimum conditions. On the Middle Rio Grande, at most sites only the outsides of eroding bends are stabilized, and only where there is a direct erosional threat to riverside facilities. In the Cochiti division there was one site with 9 bends threatening the levee where all of the curves were stabilized, and then only about 1/2 of the total bankline.

In the Espanola Division, bars that used to be removed annually are now being left and the corresponding natural backwaters provide habitat. Many of these bars are now growing vegetation. In the Espanola Division several small ponds were constructed by Reclamation which now have fish in them and are frequented by numerous bird species. Downstream of the Rio Chance confluence, the river is generally wider and groins can be used to protect eroding river banks. Sediment deposits on the downstream side of the groins creates small side channels. The groins also create areas where snags lodge, and areas of slow and fast water enhancing the previous fish habitat. Boulders along with dead trees cabled to the banks are often placed to provide habitat diversity.

In the Cochiti Division, the clear water releases from Cochiti dam have increased the meandering tendency of the river channel. In the Santo Domingo area a geomorphological study was completed which showed that the river had reached a dynamic equilibrium length and was stabilized in this planform. Mitigation for this

project included construction of backwaters on the downstream sides of the point bars and revegetation. River bar material was used to shape the banks prior to riprap placement. The procedure for bar excavation was to excavate the center portion of the bar leaving a dike at the edge of the water. This minimized the amount of time equipment was in the water and reduced turbidity during construction. The bars reformed during the first spring runoff after construction.

Throughout most of the Albuquerque and Belen Divisions where the river banks are mostly defined by Kelner Jetties, maintenance consisted of replacing Jettees that have sunk into the sand bed. Bars and islands have become established reducing the effective channel width. These bars have been mowed annually to pass large flows.

In the lower part of the Socorro Division sediment deposition has been significant and the river channel has become entirely plugged with sediment deposits. The past practice was to remove the entire plug with land based equipment. However, Reclamation has purchased several amphibious excavators capable of working in wet soft conditions. Using these excavators after the spring runoff of 1991 a small pilot cut was made through a sediment plug. The excavation volume was about 5,000 cubic yards. Flow from a thunderstorm flowed through the pilot cut and removed about 300,000 cubic yards of sediment. Under the past practice about 300,000 cubic yards of material would have been excavated. After the trench is cut through the plug water flow will erode the plug in 4 to 5 days and remove the small amount of spoil in about three weeks. This technique eliminates the damage to riparian vegetation on the channel banks where the excavated material would have been deposited. The pilot cut technique can be used in erodible sand deposits but not in clay deposits. In areas of clay deposits, and in the reservoir delta area, Reclamation has been able to excavate a channel using amphibious machines. This resulted in a channel that maximized sediment transport while minimizing mechanical removal of sediment deposits.

RECOMMENDATIONS FOR FUTURE RIVER MAINTENANCE WORK

Two important factors effecting future river channel maintenance work are the presence of endangered species and an initiative to preserve and enhance the riverine riparian forest (Bosque). While from an engineering view point the channel improvements and reservoir construction in many areas, has had the desired effect, the cumulative impact of man's activities in the Middle Rio Grande Basin has adversely effected the riverine and riparian biology. Numerous factors have effected the ecological decline, including but not limited to: (1) river water quality deterioration due to wastewater effluents and non-point sources of water pollution, (2) construction of irrigation diversion dams that eliminated fish migration, (3) irrigation water withdrawals, (4) construction of large dams that reduced peak flows, (5) channelization activities that changed the type and diversity of aquatic habitat, (6) levee construction that reduced the width of channel migration, (7) floodplain development in areas where levees have not been constructed, (8) reduced amounts of substrate (bed sediments) sources important for the survival of some aquatic species, (9) introduction of exotic predatory fish species, and (10) introduction of exotic riparian tree species. No analysis of the respective individual magnitudes of these factors will be made and only those factors that are currently within the authorized responsibilities of Reclamation will be considered in determining future river maintenance activities.

It is recommended that river maintenance approaches used since 1985 continue in cases where they are determined to be the best overall management practices. This could included strategic bank stabilization at critical locations where erosion is encroaching upon river side levees or other facilities. The method of stabilization may continue to be riprap, groins or Kelner Jetties, or new techniques that are proven to be useful. For example, rootwads buried in the bank together with boulder groupings and bank vegetation plantings are planned as a test case for one site in the Espanola Division and one site in the Cochiti Division. Detailed velocity and cross section measurements were made at each site during the 1995 spring runoff and are planned after construction to determine the effects of this method on near bank velocities and bank erodability. It is recommended that more innovative and less structural approaches be used where river maintenance objectives can be met in a manner that contributes to the biological integrity of the river system.

One of the goals of the Bosque Preservation Initiative is to have more natural hydrographs, and higher peak flows. Higher peak flows would flood overbank areas and cause erosion and deposition along river banklines to encourage new cottonwood growth. Larger peak flows can be released during spring runoff provided the safe river channel capacity is not exceeded. On the Middle Rio Grande the safe channel capacity is exceeded at discharges less than the historic 2-yr. return period peak flow, or channel forming discharge. At the safe channel capacity, flows do not go overbank except near the headwaters of Elephant Butte Reservoir, and several other small areas. One bridge and part of the levee do not have the capacity to pass the historical 2-yr peak flow because of channel aggradation. There are numerous other sites where bank erosion threatens the levee toe. It is recommended that adequate future river channel maintenance continue to increase the safe channel capacity to meet the goals of the Bosque Preservation Initiative, and pass peak flows that are more representative of unregulated peak flows.

ACKNOWLEDGEMENTS

The author would like to express grateful acknowledgement to Messrs. Henrique Barreto, Robert Padilla, and William Lane for compiling some of the data included in this paper, and Ms. Gene Gray for manuscript preparation.

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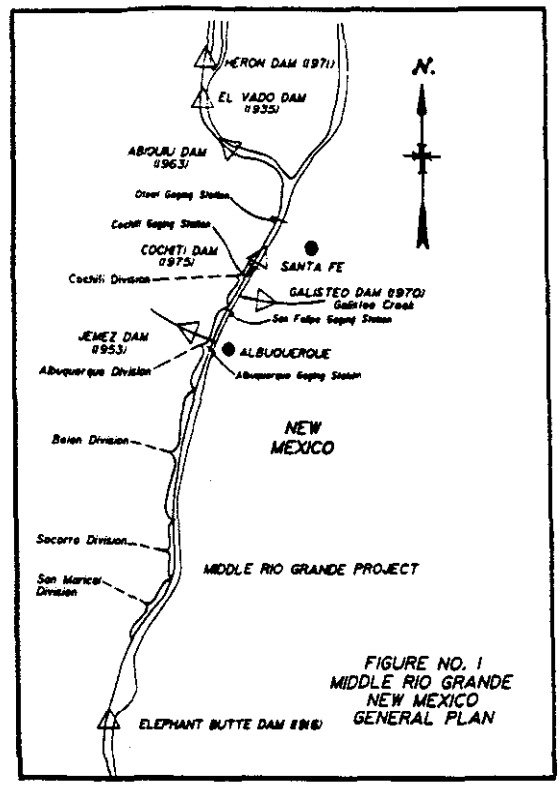

FIGURE NO. 1
MIDDLE RIO GRANDE
NEW MEXICO
GENERAL PLAN

PASSAIC RIVER FLOOD PROTECTION PROJECT
UPPER BASIN SEDIMENTATION STUDY

David Abraham, Research Hydraulic Engineer, Waterways Experiment Station, Vicksburg, MS; William A. Thomas P.E., Mobile Boundary Hydraulics, Vicksburg, MS.

INTRODUCTION

Description of the Project: Passaic River and its watershed are located in northeastern New Jersey and southeastern New York. The basin is about 935 square miles, and drains the Passaic, Whipppany, Rockaway, Pequannock, Wanaque, Ramapo, Pompton, and Saddle Rivers. Details of the watershed descriptions can be found in the report "Passaic River Flood Protection Project, Preconstruction Engineering and Design", dated August 1992. See Figure 1 for an overview of the project features.

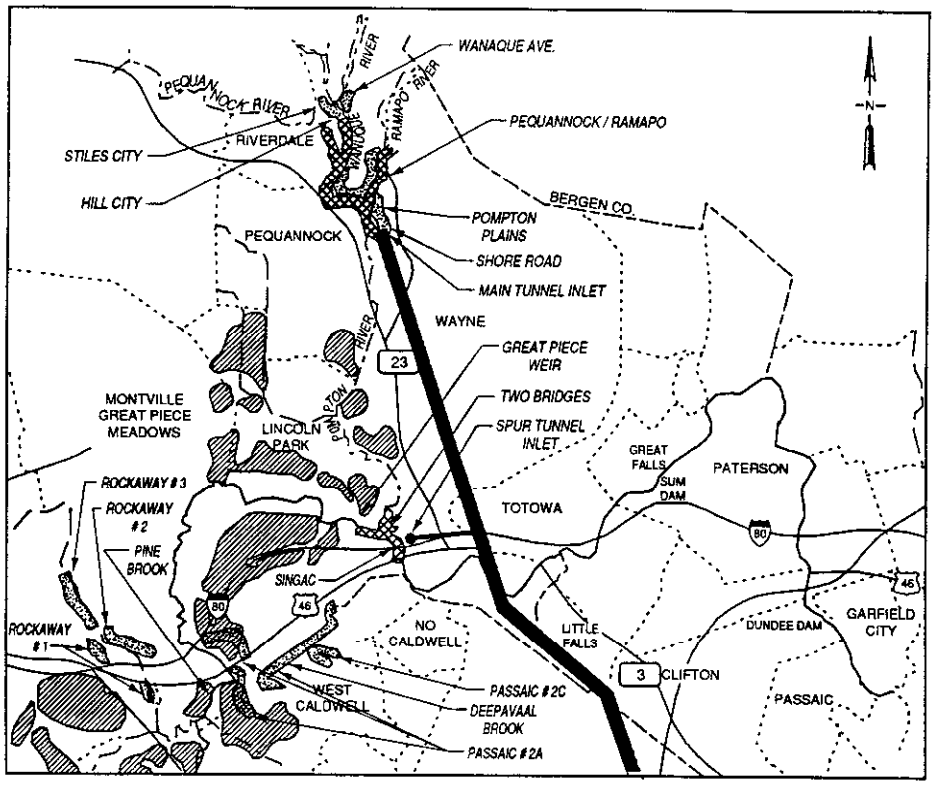

FIGURE 1. PROJECT FEATURES

The area with the greatest flooding potential is the Pompton River. To alleviate this problem the construction of a 42 foot diameter tunnel is proposed. This tunnel will have a main inlet at the confluence of Pompton, Ramapo, and Pequannock Rivers, and a spur inlet on the Passaic River just south of Two Bridges. The tunnel will empty into Newark Bay. The design will divert the peak of the flood waters underground from the upper Pompton River and Passaic River at Two Bridges, to Newark Bay.

The diversion through the tunnel will lower water surface elevations in many of the project reaches. Although this is desirable from the standpoint of flood protection, it could alter sedimentation patterns. Such alterations are especially important near the tunnel inlets, in the vicinity of the Great Piece Meadows wetland, and along the Pompton River. This paper addresses these concerns.

Approach of Sedimentation Investigation: Prior to beginning the study, it was known that the upper reaches of the Passaic River Basin contain branched river networks whose peak flows occur at different times and locations. These phased peaks can result in reverse flows in some reaches. Bed material also varies from reach to reach.

HEC-6, a one-dimensional multi-grain-size sediment transport code, was selected for the major portions of the study. At the time the study was initiated, HEC-6 had no provision for closed loop networks or reverse flows. Revisions to the code had to be made during the course of the study so that reverse and branching flows could be modeled. These revisions are incorporated into a new code and are not part of the HEC-6 library code. In order to simulate the phasing of flow peaks, the flow hydrographs from UNET, a one dimensional, fixed bed, unsteady hydraulic model, were taken as hydraulic input to the HEC-6 model.

The HEC-6 model network was developed as shown in figure 2 and appropriate geometric, sediment, and hydrologic input files created. The model was verified to hydraulic and sediment field data where possible, and otherwise to UNET hydraulic data for water surface elevations. First base, and then plan project conditions were simulated, with the final intent being to learn about any differences in aggredation or degradation between these two conditions. A more detailed description of the study and findings follows.

BASE CONDITION

Geometric Model: The geometric model is based on the network schematic as shown in Figure 2. It consists of a file of cross sections arranged sequentially from downstream to upstream. Thirteen reaches in all were used. Four hundred and six cross sections were needed. In this file, n values for flow resistance, moveable bed limits, moveable bed depth, channel limits, and conveyance limits were also specified. The values of the different parameters were selected based on field observation, prior UNET files where applicable, information gleaned from topographic maps, and experience. The dams and weirs were modeled by setting the height of the dam as the channel bottom, and preventing it from scouring. With respect to cross sectional area, bridges were modeled using a special feature of HEC-6 which subtracts the areas of piers and embankments out of the cross sectional area.

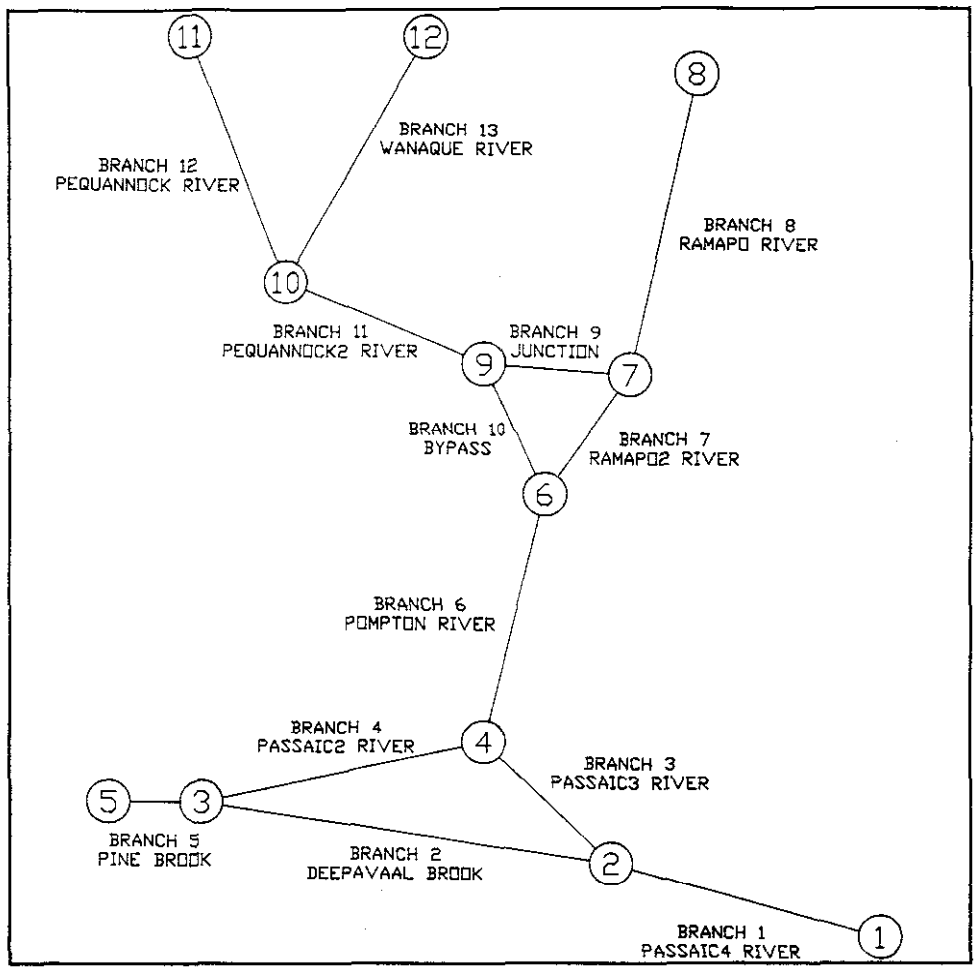

FIGURE 2. HEC6 NETWORK SCHEMATIC

Sedimentary Model: The portion of the model that moves sediment requires the specifying of a transport function, inflowing concentration verses flow data, and bed gradations. The hydraulic analysis package 'SAM' was used to assist in selecting the most suitable transport function. Based on the factors of channel slope, velocity, width, depth, and D50 the Yang function was used. The concentration verses flow graphs were developed using field data collected by the USGS in the spring of 1994. Information was available from the Pine Brook, Singac, and Pompton Plains gaging sites. This information is used by the model to specify inflowing concentrations at model boundaries and local inflows, for varying flowrates. Finally, information about the channel bed material properties is required by the model. With the exception of the Wolman count done on the Pequannock River, all of the bed gradations were obtained from sieve analysis of bed samples. These bed gradations are entered in the model according to the locations from which they were taken.

Hydrologic Model: Flow and stage hydrographs produced by the UNET unsteady flow model were obtained from the District. The flood frequencies were the 2-year, 25-year and 100-year events. The stage data were used mainly in the verification of the HEC-6 hydraulic output. The flow data were used as direct input to the HEC-6 model. The hydrographs for

all 406 cross sections in the network were obtained from the UNET run at six hour time steps. A program was written which arranged the order of the input file to coincide with the order in which HEC-6 processes each cross section. So, for a given time step, the flowrate for each cross section was listed in its sequential order. The same procedure was used for all following time steps. For each flood frequency there were about 56 6-hour time steps, representing a hydrograph of about 14 days duration. By using the available hydrologic data in this way, both the spatial and temporal integrity of the various flood waves moving through the network were preserved in the HEC-6 model runs. This was especially important in areas of reverse flow and at times where phased peaks occurred. Throughout the simulation no correction was made to the flows for changes in bed geometry.

One other very important aspect of the hydrology of the model was the channel forming discharge concept. In short, this concept proposes that the major features of channel geometry throughout the life of a river or stream are brought about by the floods that run near bankfull. Floods of this dimension have generally been found to be associated with 2-year flood frequencies. For this reason it was considered very important to simulate the 2-year flood. This was done with the objective of simulating the changes which might occur to the channel over a longer period of time. The model results of these changes were not meant to be definitive in terms of absolute magnitude, but only in the sense of defining trends of deposition or scour. The changes to the network over 50 years were important to assess. In order to simulate such changes, historical hydraulic data were required. Such data as needed for the HEC-6 model were not available. There were historical data, but not at all the locations and over the time periods required. In the absence of such data, the channel forming discharge concept was invoked. From 100 years of historical records the District informed us that the average daily flow was 1148 cfs at Little Falls. This translates to about 36 billion cubic feet of water flowing through the channel in a years time. The peak flow of the 2-year flood is approximately 5220 cfs at Little Falls after diversion. One would need to run that flowrate through the channel continuously for about 80 days in order to move the same volume of water that is moved in a years time by the average daily flow. In order to simulate 50 years of volume using the 2-year flood, it would be necessary to run a simulation for 80 days per year for 50 years; or a 4000 day simulation.

Verification: When the model was complete in all aspects; geometry, sediment, and hydrologic data, the initial runs were made. The year 2000 was arbitrarily used to represent time equal to zero, or the initial condition. The initial condition runs were made with a single time step of six hours so that no significant bed changes were allowed to occur. In this way it was assured that the HEC-6 bed profile matched that of the UNET runs, and thus a valid comparison could be made between the water surface elevations calculated by HEC-6, and those calculated by UNET. The 2-year peak flow was selected as the flow against which to verify. The purpose of these verification runs was to check the hydraulic performance of HEC-6. With the proper hydrologic input, HEC-6 should reproduce correct water surface elevations. In the absence of correctly calculated hydraulic parameters, the subsequent sediment calculations would be meaningless. After some minor adjustments to Manning 'n' values, the water surface profiles of the UNET runs were matched very closely. Figure 3 shows the comparison of HEC-6 and UNET water surface profiles for Branch 6, the Pompton River.

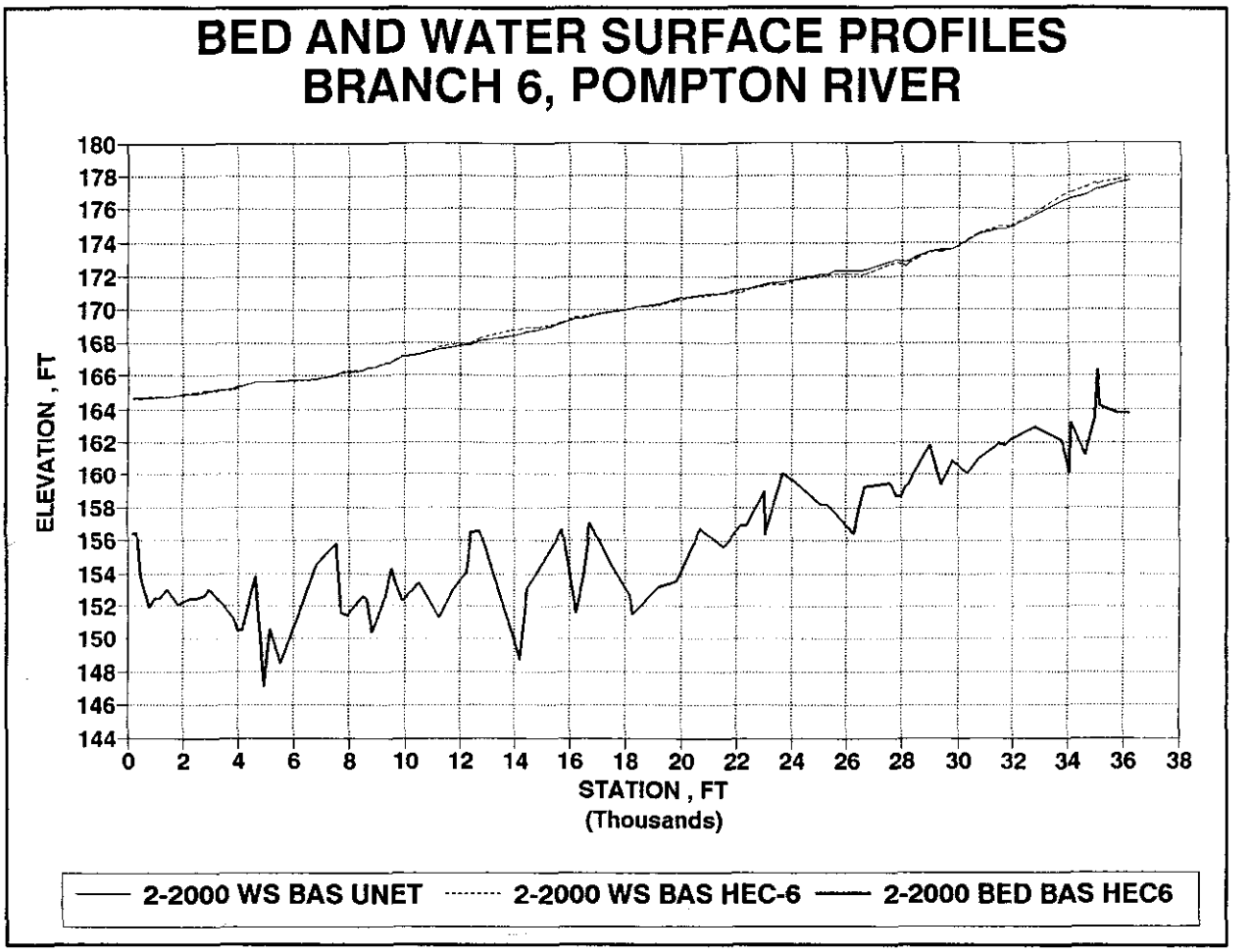

FIGURE 3. WATER SURFACE VERIFICATION PROFILES

The match was adequate to assure correctly calculated hydraulic parameters. The legend of the graph shows from left to right: the flood frequency, the year of the simulation, water surface or channel bed, base or plan, and specific notes. For example, the legend **2-2000 WS BAS HEC-6** indicates that the graph is for a 2-year flood in the year 2000 and shows the water surface elevations for the base condition for an HEC-6 simulation.

PLAN CONDITION

Geometric/Sedimentary/Hydrologic Model: The District requested that WES investigate both the base (un-improved) and plan (improved) condition 50 years (year 2050) after the initial condition (year 2000). The plan geometry differed from the base geometry in that levees, channel modifications, and the diversion tunnel were added to the plan. These can be seen in Figure 1. The plan hydrology also differed from the base hydrology and was obtained from UNET simulations which used plan geometry.

RESULTS

Base and Plan: With the plan in place, reductions in water surfaces were attained in most branches of the upstream network. The greatest reductions were for the 100-year flood frequency. In general, differences between the plan at year 2000 and year 2050 were relatively small. This indicated that the plan itself was not self defeating at any noticeable location. In other words, the plan in general was stable once it was in place and in operation over 50 years and through the various flood hydrographs. Thus there was no indication that excessive long term maintenance would be required.

The sedimentation characteristics of the network require a bit more discussion. The main places of concern were Two Bridges, the Great Piece Weir, the tunnel inlets, and the Pompton River. At Two Bridges there was much concern with the amount of deposition noticed during field observations and from the model long term runs. Especially in view of the fact that the Great Piece Weir was proposed for this vicinity. It was concluded from field observations and the model runs, that in the vicinity of Two Bridges deposition of up to 3 feet might occur for the 100 year flood. This could indicate a necessity for short term maintenance after extreme events. The proposed location of the Great Piece Weir 700 feet upstream of Two Bridges appeared to put the weir just at the edge of this deposition zone. Upstream of the Great Piece Weir there was minimal deposition, probably fines supplied from the wetland.
The spur inlet of the tunnel was just downstream of Two Bridges on the Passaic River. Without exception every run showed this area as a zone of deposition. It is most probably due to the initial delta development caused by the backwater influence of Beatties Dam. The amount of deposition at this location seemed to be reduced by the plan, possibly as a result of the reduced degredation in the Pompton River.

The Pompton River appears to be the most unstable reach in the network. It showed degradation in the upper sections, and some deposition in the lower reaches. See Figure 4.

The rate of change of either seems to be reduced by the plan. Initially it was thought that the increased water slope due to the tunnel operation could increase the tendency of degradation in this River. However, the 100 year plan runs compared to the 100 year base runs showed that not only were the water surface elevations lower in the plan, but the slope of the water surface was less. This accounted for the reduced rates of degradation. However, it was recommended that since this area was already known to be unstable, that the District photo-document the stream banks to show that such was the case. If not, future degradation could be attributed to the project, when in fact the opposite appeared to be true. The project seemed to reduce the rate of degradation due to flows equal to or exceeding the 2-year flood.

At the upper end of the Pompton River in the vicinity of the main inlet, there was a very slight indication toward scour. In Branch 10, which adjoins Branch 6 in the same vicinity but upstream of the main inlet, a tendency for deposition seemed evident. Due to the channel improvements at this location, some deposition was expected as the system tries to return to equilibrium. As with the spur inlet, defining the percentages and distribution of sediments that would actually enter the tunnel was beyond the capabilities of the HEC-6 model. It was recommended that a physical model be used to help determine the types and amount of

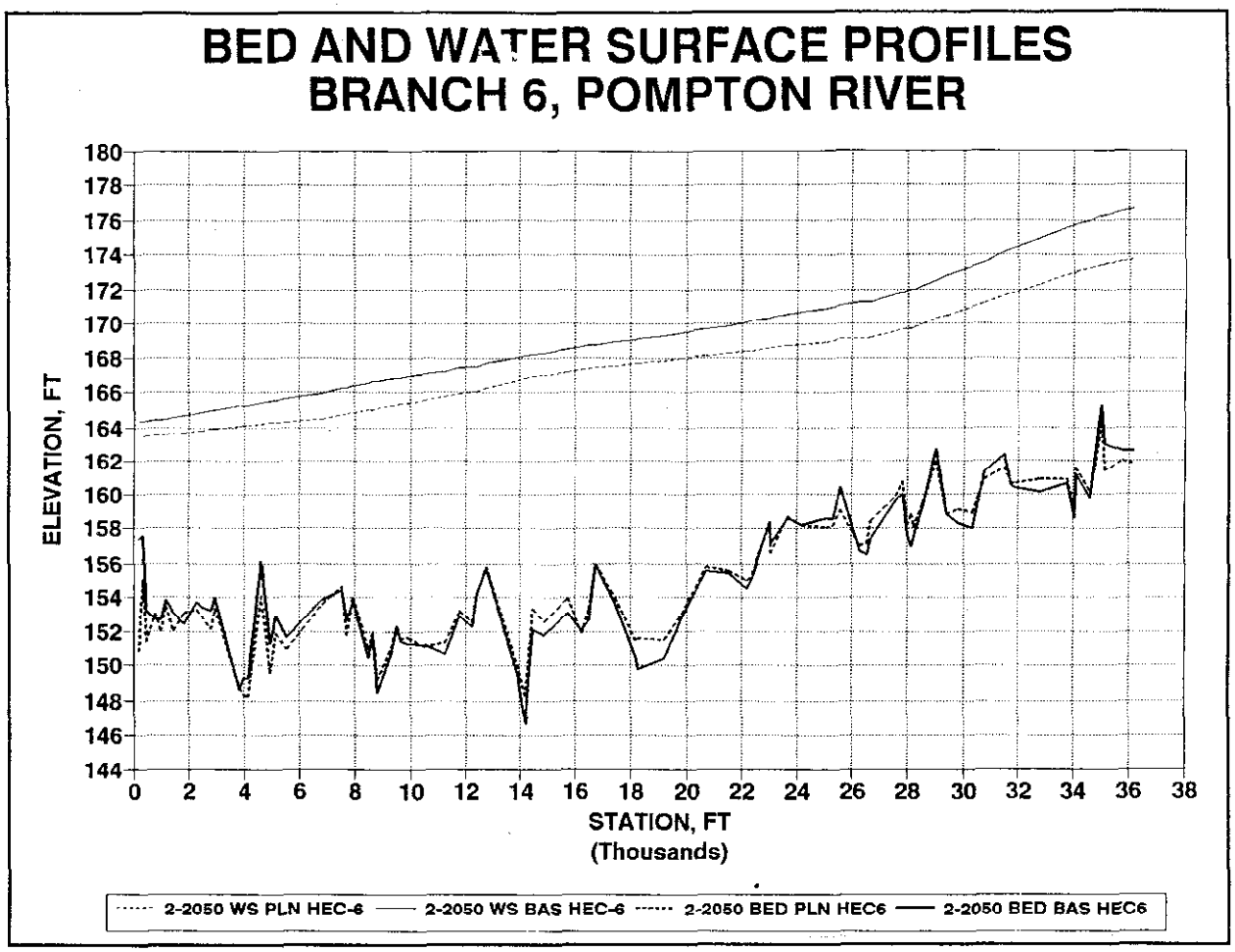

FIGURE 4. POMPTON RIVER BED PROFILE

materials that would accumulate directly in front of the tunnel inlets, and thus help define the maintenance requirements to keep the sediment traps and skimming structures operating effectively. No simulation was made for the 500 year flood. Such an event could possibly carry large material into the tunnel which could not be flushed out and would therefore require maintenance for removal.

The last area to address is the section near the Pequannock Weir which adjoins the Bypass Branch. The plan runs showed that over 50 years, some scour might occur just upstream of the Weir. This was not expected since deposition normally occurs behind dams. However, a check was made of the model sediment concentrations in this location for the 2-year flood. These were compared with measured sediment concentrations taken in March of 1994 in the vicinity of Pompton Plains. The measured values averaged 5.2 mg/l. From HEC-6 the base run sediment concentrations from day 3 hour 1800 was 5.4 mg/l, and for the plan run on day 3 at hour 0600, 2.2 mg/l. Therefore the model agreed well with measured data. With such low concentrations, it was possible that some scour could occur in this area, even though the trend to do so appeared to be minimal. Due to the uncertainty caused by this model result it was suggested to monitor the channel cross section at this location with sedimentation

ranges as in EM 1110-2-4000, appendix K paragraphs 6 to 17. Although reservoirs are explicitly described, the procedures could easily be used for rivers and streams. It was highly suggested that the tunnel inlets and Great Piece Weir also be monitored in such a manner.

Acknowledgments: The information presented herein, unless otherwise noted was obtained from a study conducted by the Waterways Experiment Station with the support of the New England Division United States Army Corps of Engineers. Permission was granted by the Chief of Engineers to publish this information.

MICRO SCALE PHYSICAL SEDIMENT MODELING (MICRO MODELING) OF INLAND WATERWAYS

By Robert D. Davinroy, District Potamologist, Potamology Section, U.S. Army Corps of Engineers, St. Louis, Missouri

INTRODUCTION

A new sediment modeling tool and methodology has recently been developed giving engineers the capability of conducting physical sediment transport studies of rivers and streams on a micro scale. The micro scale enables the user to quickly and economically address design alternatives in the prototype. Micro modeling achieves the same accuracy as the large physical models used in the past and challenges the present day numerical modeling efforts. Most importantly, the tool serves as a visual communication tool to the general public.

METHODOLOGY AND CONCEPT

Principle. Micro modeling is based upon the principle that small streams behave similarity to large rivers. In actuality, the micro scale model (micro model) is a small river patterned after a larger river and adjusted to reproduce the characteristics of the larger river (Franco 1978). This is the same principle that has been historically applied to typical large movable bed models used by the Corps of Engineers, the Bureau of Reclamation, and other laboratories throughout the United States and the world abroad. Micro modeling is an empirical approach to sediment transport modeling. By using simplistic operational devices including an electronic pump control, the digital micrometer, the personal computer, and synthetic lightweight bed material, the sediment response of a large river or stream can be modeled on an extremely small (micro) scale with remarkable success.

Scales. Micro model scales used with reliability have been as small as 1:20,000 horizontal and 1:1200 vertical. Scales may be enlarged or reduced, depending upon the particular nature and complexity of the study. By maintaining a micro scale, the operation of the model can be conducted on a table top flume by one person, comparable to that of a numerical model. The entire hydrograph may take as little as 5 minutes to simulate. These factors enable the user to visually observe the hydrodynamic/sediment transport response through the entire model during all segments of the hydrograph. This is not possible using larger models because of the size of the model and the time scale of the hydrograph (typical movable bed models use time scales ranging from several days to weeks, depending on the bed material and model scale).

Past Studies. Micro modeling was first conducted in 1994 on a reach of Mississippi River (Mile 39 to Mile 22) known as the Dogtooth Bend Reach (Figures 1 and 2). This same reach was also studied in a large movable bed model at the Waterways Experiment Station in Vicksburg, Ms. (Figure 3). This model study (WES 1994) was used to developed the Bendway Weir concept now being used extensively on the Mississippi River.

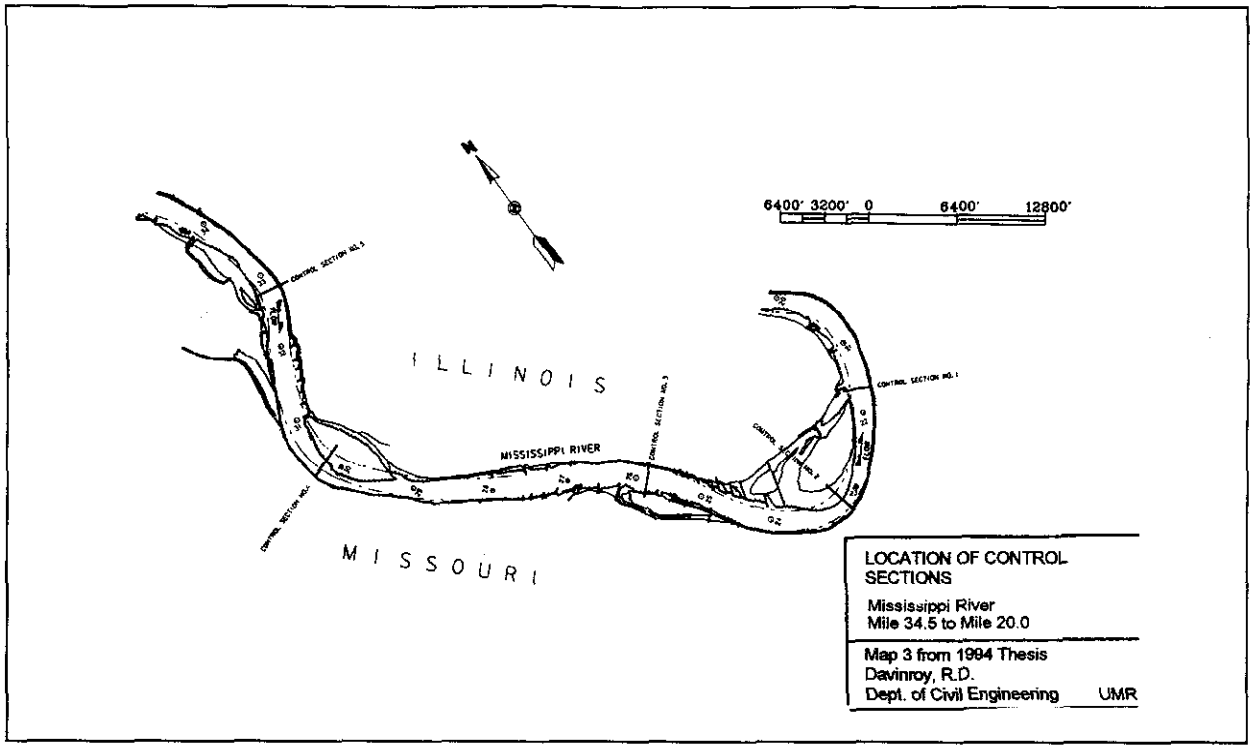

Figure 1. Dogtooth Bend Study Reach, Mississippi River

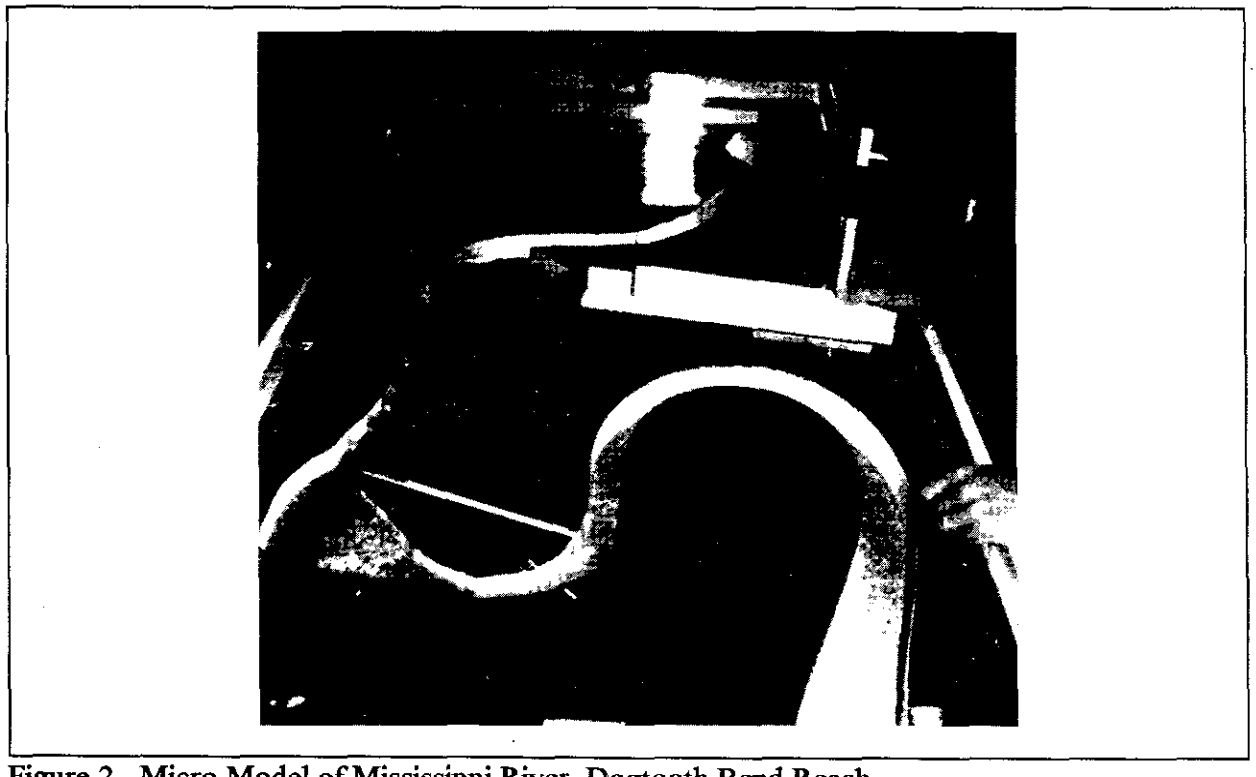

Figure 2. Micro Model of Mississippi River, Dogtooth Bend Reach

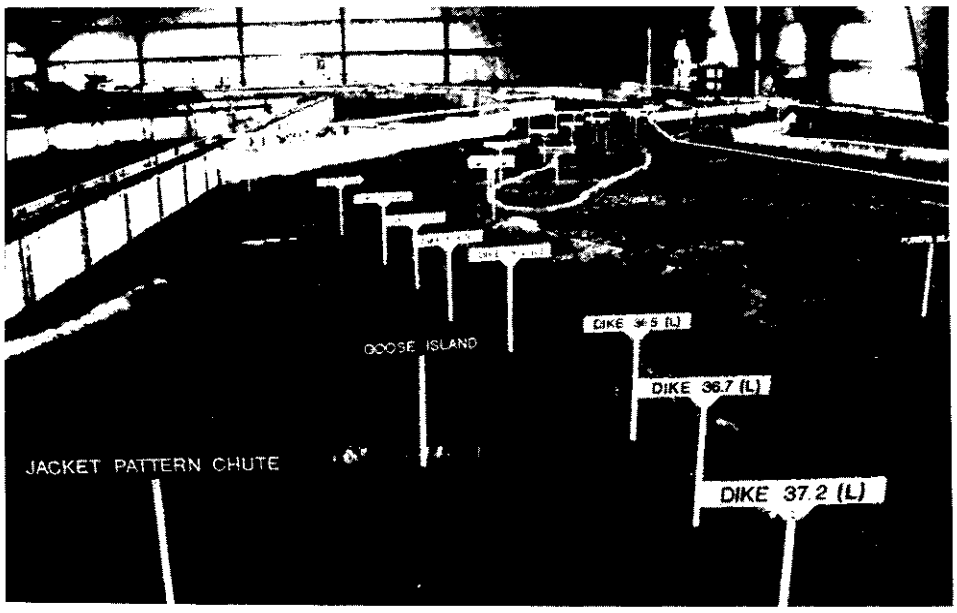

Figure 3. Dogtooth Bend Model at WES

The relative volumetric scale comparisons between the two models and the prototype is illustrated in Figure 4. An in-depth comparison of Froude and Reynolds numbers using these scales can be found in the research conducted at the University of Missouri-Rolla (Davinroy 1994). A direct comparison was made on the calibration and verification capabilities of the micro model versus the large model and the prototype. Five control sections were established as a reference. Figure 5 is a cross sectional bed comparison at one of the five sections (Control Section 2 or Mile 23.3, as located in Figure 1).

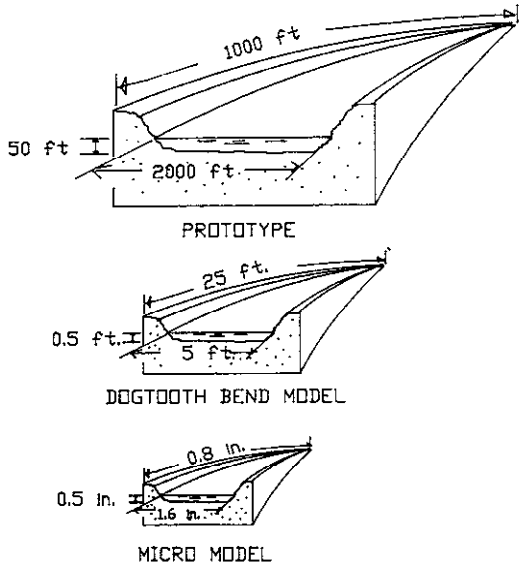

Figure 4. Conceptual Volumetric Scale Comparisons

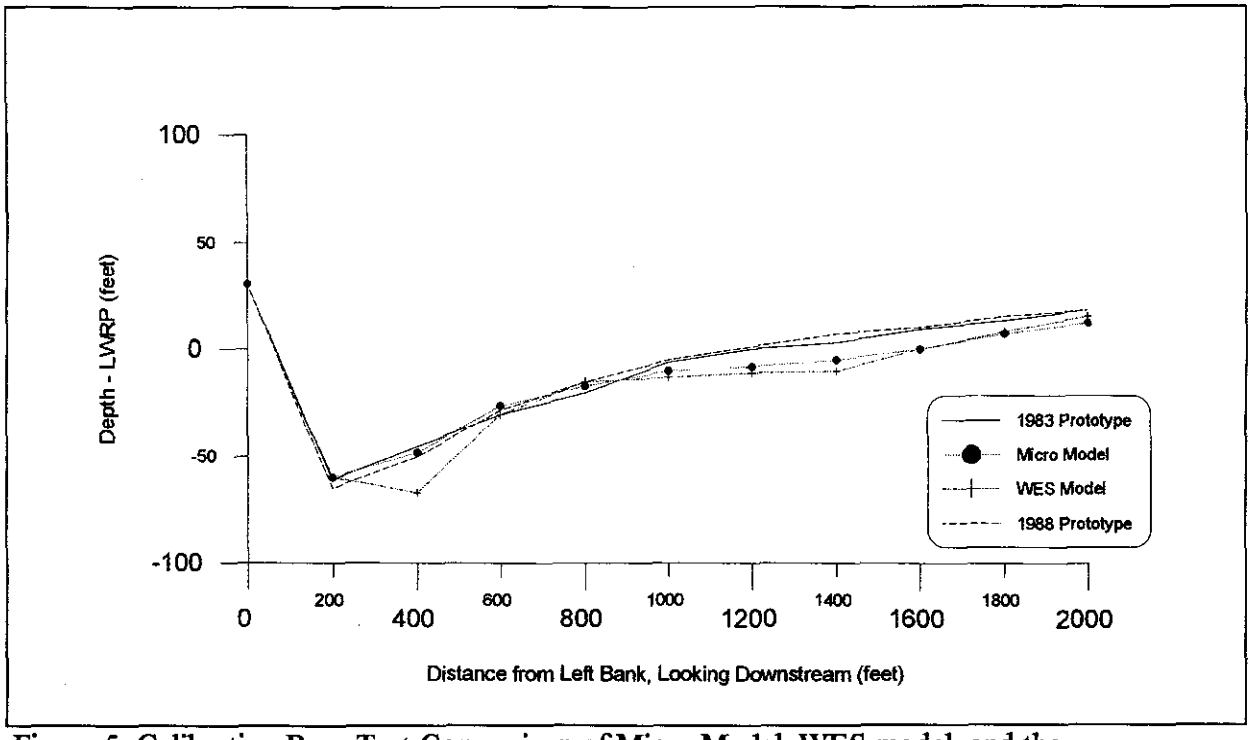

Figure 5. Calibration Base Test Comparison of Micro Model, WES model, and the Prototype at Control Section No. 2 (Mile 23.3)

The accuracy by which the micro model can simulate the expected sediment response of the river has been further verified by a particular navigation study conducted for the Memphis District Corps of Engineers in 1995. A segment of reach of the Mississippi River known as the New Madrid reach (Mile 908 to Mile 880) was micro modeled to investigate shoaling conditions within the navigation channel (Figure 6).

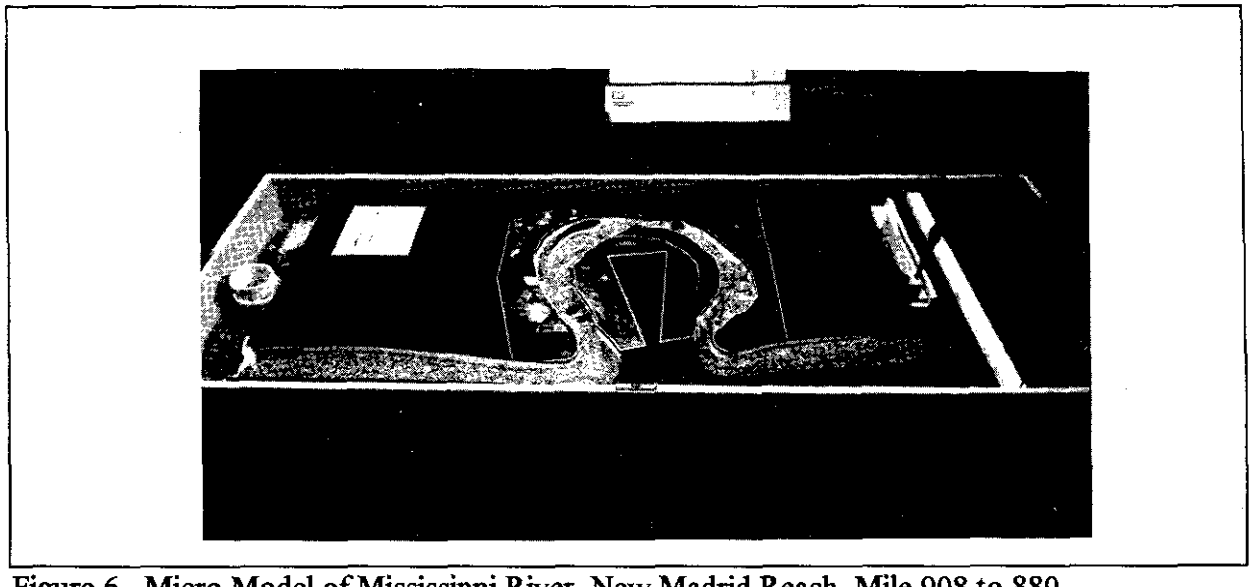

Figure 6. Micro Model of Mississippi River, New Madrid Reach, Mile 908 to 880

During initial calibration tests, the micro model displayed a particular scour and depositional pattern that was not apparent from hydrographic surveys. Material from the scour hole was generating deposition in the downstream navigation channel (Figure 7).

Figure 7. Plan View Survey Map of Micro Model Results Showing Scour at Upstream Head of New Madrid Bar Island

The District conducted a more detailed river survey in this area (Figure 8). The survey indeed revealed the scour hole pattern that had developed in the micro model. This scour hole pattern, which was the causative agent to deposition in the downstream navigation channel, would have otherwise gone undetected to river engineers using the original surveys for analysis.

The micro model study produced two design solutions to alleviate the above problem. The first solution involved the reconfiguration of New Madrid Bar Island, while the second alternative involved the implementation of Bendway Weirs in the navigation channel.

Of particular importance to this study was the political constraints imposed upon the river engineers of the Memphis District. The city of New Madrid, Missouri did not approve of dikes or any other river regulation structures placed within sight of the city front. The city was also concerned about increased deposition within New Madrid Harbor. The micro model not only satisfied these major constraints, but also gave the Memphis District river engineers a portable and highly visual communication tool by which to physically demonstrate alternative solutions to the people of New Madrid.

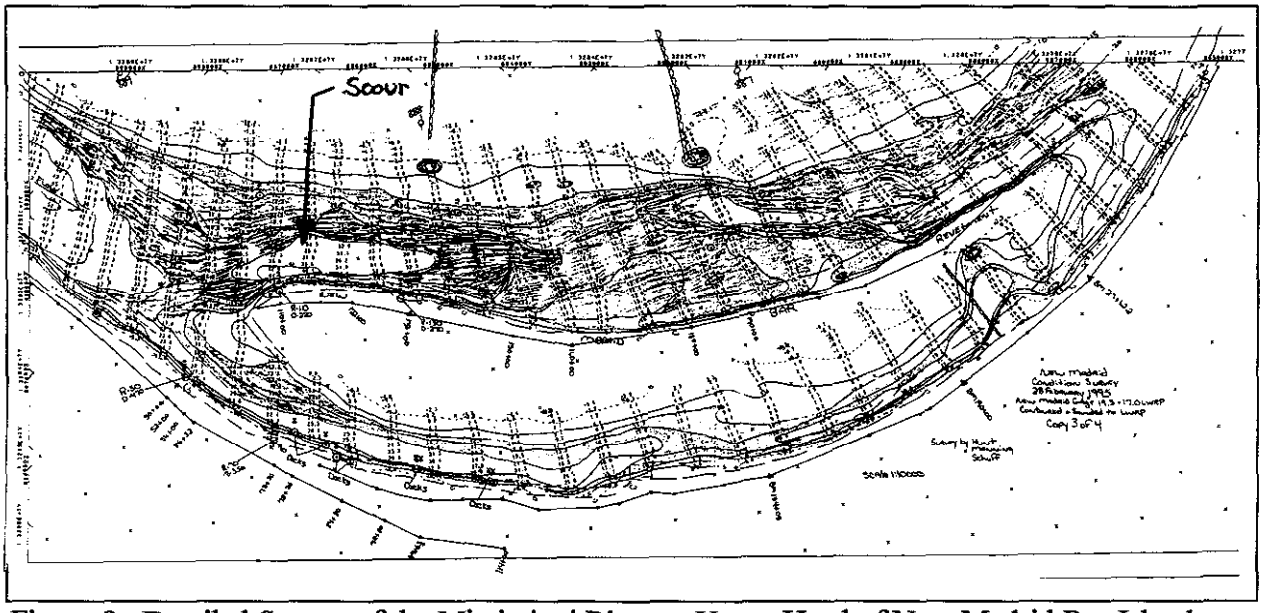

Figure 8. Detailed Survey of the Mississippi River at Upper Head of New Madrid Bar Island, Mile 888.5

Operation. The micro model concept incorporates lightweight bed material (plastic, specific gravity of 1.23) and precision photogrammetric alignment inserts placed within a table top hydraulic flume. The hydrograph is controlled by a function generator, simulating a user specified sinusoidal pump response. Floodplain tilt, discharge, sediment load, and vertical scale adjustments are made during calibration. Because of the micro scale, the duration of the typical hydrograph is extremely short. This enables the user to conduct a multitude of tests in a relatively quick time period. By examining many runs using statistical averaging, the method of bed molding required by large models is eliminated and replaced by an "average expected sediment response" (Davinroy 1994). As a result, one can compensate for anomalies in the sediment response between runs and make realistic comparisons between base conditions and design alternatives.

CONCLUSIONS

Direct comparisons between the sediment response of the micro model and the prototype verify the validity of this technology. Recent tests conducted on the Lower Mississippi River at New Madrid Bend exemplify the sensitivity of the model in defining intricate details of scour and deposition (boundary effects) as found in the prototype.

Micro modeling gives engineers an inexpensive, accurate, and highly visually table top tool by which to address the complex problems associated with sedimentation (loose boundary hydraulics). Once model calibration/verification is achieved, the engineer may then gage the effectiveness of various structural design alternatives for stabilization or navigation channel improvement of inland waterways. This includes revetments, dikes, closure structures, bendway weirs, etc. The physical modeling concept may also be applied to smaller rivers and streams.

Engineers that would normally not consider the use of a physical model because of time and cost now have an option. In addition, micro modeling enables the user to visually demonstrate and communicate to the general public various design alternatives and their effects in the prototype.

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BENDWAY WEIRS ON THE MISSISSIPPI RIVER, A STATUS REPORT

By Robert D. Davinroy, District Potamologist, Potamology Section, U.S. Army Corps of Engineers, St. Louis, Missouri; and Stephen L. Redington, Chief, River Engineering Unit, Potamology Section, U.S. Army Corps of Engineers, St. Louis, Missouri

INTRODUCTION

In 1989, a new river regulation structure called a Bendway Weir (Figure 1) was placed in the outside of a bendway channel of the Mississippi River at Dogtooth Bend, Mile 23.3. By the end of 1995, 114 of these structures will have been constructed in 13 bends of the Mississippi River. This report includes an inventory of these structures and a general assessment of their performance in the river.

Definition. Bendway Weirs are submerged, rock structures placed directly in the navigation channel beneath passing traffic. They are angled upstream 20 to 30 degrees to a line drawn perpendicular to the low flow line. These structures are designed to negate natural secondary current development in bends and redistribute flow for desired sediment management (Davinroy 1992). Bendway Weirs encourage deposition of bed material on the outside of the bend and scour on the inside of the bend. The resulting redistribution of velocity produces a wider and safer navigation channel through the bend while eliminating or alleviating repetitive maintenance dredging.

Figure 1. Artist Rendering of Bendway Weirs on the Mississippi River, Looking Downstream Through the Bend

Background. Navigation related problems and repetitive maintenance dredging in bends of the Mississippi River dictated the need for a new structural design approach to sediment management of big rivers. To satisfy this need, Bendway Weirs were conceived and designed from extensive physical sediment modeling performed at the U.S. Waterways Experiment Station for the St. Louis District Corps of Engineers (WES 1995). Through model tests, structural design details such as angle, orientation, height, length, and spacing were developed. These details were directly implemented in the prototype at two bends within the model study reach (Prices Bend, Mile 30.0 and Dogtooth Bend, Mile 23.0). Intensified monitoring was conducted before, during, and after construction. Almost immediately after construction the structures displayed similar tendencies as observed in the model.

The two initial Bendway Weir fields worked so well that river engineers gained confidence in applying the concept toward other problem bends. In 1993, a project plan was developed including a tentative construction schedule of future Bendway Weir work on the Mississippi River between the confluence of the Missouri River (Mile 195) and the Ohio River at Cairo, Ill. (Mile 0.0)

IMPACTS

Construction. Table 1 illustrates Bendway Weir construction to date and resulting impacts. This table is based upon data collected from both the St. Louis District and the United States Coast Guard.

BENDWAY WEIRS ON THE MISSISSIPPI RIVER

LOCATION OF BEND	RIVER MILE / Date of Installation	BEND LENGTH (ft.) / Degree of Curvature	NUMBER OF WEIRS IN BEND	TOTAL REQUIRED ROCK TONNAGE	AMOUNT OF AVERAGE CHANNEL WIDENING (ft.)	HISTORIC DREDGING (Millions of Cubic Yards). 1978-1995	REQUIRED DREDGING AFTER WEIRS (Millions of Cubic Yards)	HISTORIC NUMBER OF ACCIDENTS IN BEND (1980-1995)	ACCIDENTS AFTER WEIRS
Ft. Chartres	130.5 L Mar 1994	4000/ 60	9	339,000	100	0.63	0	15	0
Kaskaskia	116.7 L Feb 1993	5200/ 58	11	200,000	110	1.5	0	17	1
Red Rock	94.5 R Jan 1993	5600 / 90	9	147,000	500	1.5	0	44	0
Fountain Bluff	83.5 L Under Const.	5400/ 65	10	360,000 (est.)	Under Const.	0.26	0	11	N/A
Picayune	57.0 L Feb 1995	8800 / 22	14	197,000	210	1.45	0	2	0
Cape Rock	54.5 R Feb 1992	8600 / 30	8	160,000	550	0.6	0	8	1
Cape Bend	49.0 R Mar 1995	4800 / 30	13	194,000	120	2.17	0	11	0
Prices	30.0 R Nov 1991	9000 / 50	9	174,000	200	2.89	0	10	0
Dogtooth	23.3 R Dec 1990	13,000 / 65	13	358,000	190	0.98	0	39	5
Scudders	17.0 L Under Const.	8000/ 62	5	300,000 (est.)	Under Const.	2.3	0	19	N/A
Eliza Point	6.2 L Under Const.	7500 / 64	6	360,000 (est.)	Under Const.	1.88	0	10	0
Greenfield	3.5 R Feb 1995	5400 / 63	7	146,000	90	1.77	0	11	0
Victoria	595 L Aug 1995	N/A	6	1,600,000	N/A	N/A	N/A	N/A	N/A
TOTALS	13 BENDS	———	114	———	———	———	———	———	

Several comments to the table are as follows:

1. Bend Length. This value is based upon the lateral streamline distance of the bend.

2. Bend Curvature. The bend curvature in degrees is based upon the average curvature (most of the bends of the Mississippi River contained multiple radii). This variable may be used as an indicator of the severity of the navigation alignment.

3. Total Rock Tonnage. This quantity is based upon the amount of rock placed in the river either estimated before construction or recorded after construction from daily inspection logs.

4. Amount of Channel Widening. This is computed as the average increase in navigation width based upon the lateral change in the 10 foot Low Water Reference Plane (LWRP) contour, taken at the apex of the bend. Detailed hydrographic surveys before and after construction were used.

Observations in the Prototype. Generally, the greater the degree of bend curvature, the more severe the initial scour development against the outside bank at the apex of the bend. Under these conditions, the adjacent point bar formation is more pronounced. Correspondingly, the more severe the bend, the more efficient Bendway Weirs become. This is apparent at Red Rock (Mile 95), where the degree of bend curvature is approximately 90 degrees. An average navigation width of approximately 500 feet was gained after construction at the apex. Intuitively, a more conservative design could be employed under these conditions. However, two major constraints occur to the river engineer during design;

1. The spacing of the structures must be minimized (the spacing must be approximately the same as the width of the localized navigation channel, or the -10 foot LWRP contour width) in order to ensure a continuous curvature on the inside navigation alignment. Increasing this minimum spacing will enable the tremendous secondary current effects to incur irregular point bar deposition, resulting in an unstable channel alignment on the inside of the bend.

2. The elevation of the weirs cannot be compromised. Model study results indicate there is not a one to one relationship between channel widening and structural height (WES 1995). If weir height is compromised, then secondary current effects can again cause an unstable navigation alignment.

The fact remains that three dimensional bend flow mechanics and associated secondary currents make for complex effects that cannot be "cookbooked" as with traditional dike design. Additional research is being conducted at the Waterway Experiment Station with the goal of developing more concrete design parameters and relationships by which to apply the Bendway Weir concept.

An important observation not reflected in Table 1 is the varying geomorphology of the Middle Mississippi River. For example, extensive rock outcroppings occur at several bends including

Red Rock, Cape Rock, and Fountain Bluff. Conversely, the river bottom through some of these reaches is armored, possible effecting Bendway Weir effects.

In addition, the backwater effects from the Ohio River influence velocities as far north as Prices, Mile 30. This factor probably limits the full development of Bendway Weir design along this stretch of river.

Prototype Monitoring. Figures 2 and 3 are detailed hydrographic surveys depicting the bed configuration at Red Rock, Mile 95. These surveys were collected using multi swath technology. The resulting bathymetric plan view maps show approximately 500 feet of additional navigation width gained at the apex of the bend.

Improved Conditions in Bends. Since 1990, there has been a steady trend toward favorable navigation conditions in bends after construction. The increased navigation widths and measured redistribution of velocities have enabled downbound pilots to steer through the bends instead of flanking through during low water. These changed conditions have virtually eliminated point bar dredging and have made for a safer and more reliable navigation channel. The wider widths have also deterred the formation of ice jams during the winter months.

Environmental Impacts. In 1994, an environmental study to address the impact of Bendway Weirs on fish habitat was initiated by the St. Louis District. The study has employed state of the art deep water fish assessment techniques including hydroacoustics (Kasul and Baker, 1995), explosives, and electrofishing (Atwood, Peterson, and Sheenan, 1994). A team effort in the field has been accomplished by resources and personnel from the St. Louis District, the Lower Mississippi River Valley Division, the U.S. Fish and Wildlife Service, the Illinois and Missouri Departments of Conservation, and the University of Southern Illinois Fisheries Department.

The data collected to date has been very encouraging. The study has examined fish population differences between bends with and without Bendway Weirs. Typically, the bends with Bendway Weirs have shown increases in fish populations as much as fivefold. Also, a greater variety in size distribution and location are noted within the Bendway Weir fields.

Additional data will be collected in September of 1995. A new an innovative data collection procedure will be tested to qualify or "ground truth" the hydroacoustics. Results of this effort should be available early in 1996.

Implementation of the Bendway Weir concept has not negatively impacted an endangered species of bird, the Least Tern. This bird nests on sand bars of the Mississippi and Missouri Rivers. Bendway Weir construction has eliminated the need for the construction of traditional bar dike structures that would otherwise impact established and future nesting sites.

Future Work. Bendway Weir construction will continue at other troublesome bends on the Mississippi River. Studies are underway for other applications including straight reach sediment management and bank stabilization.

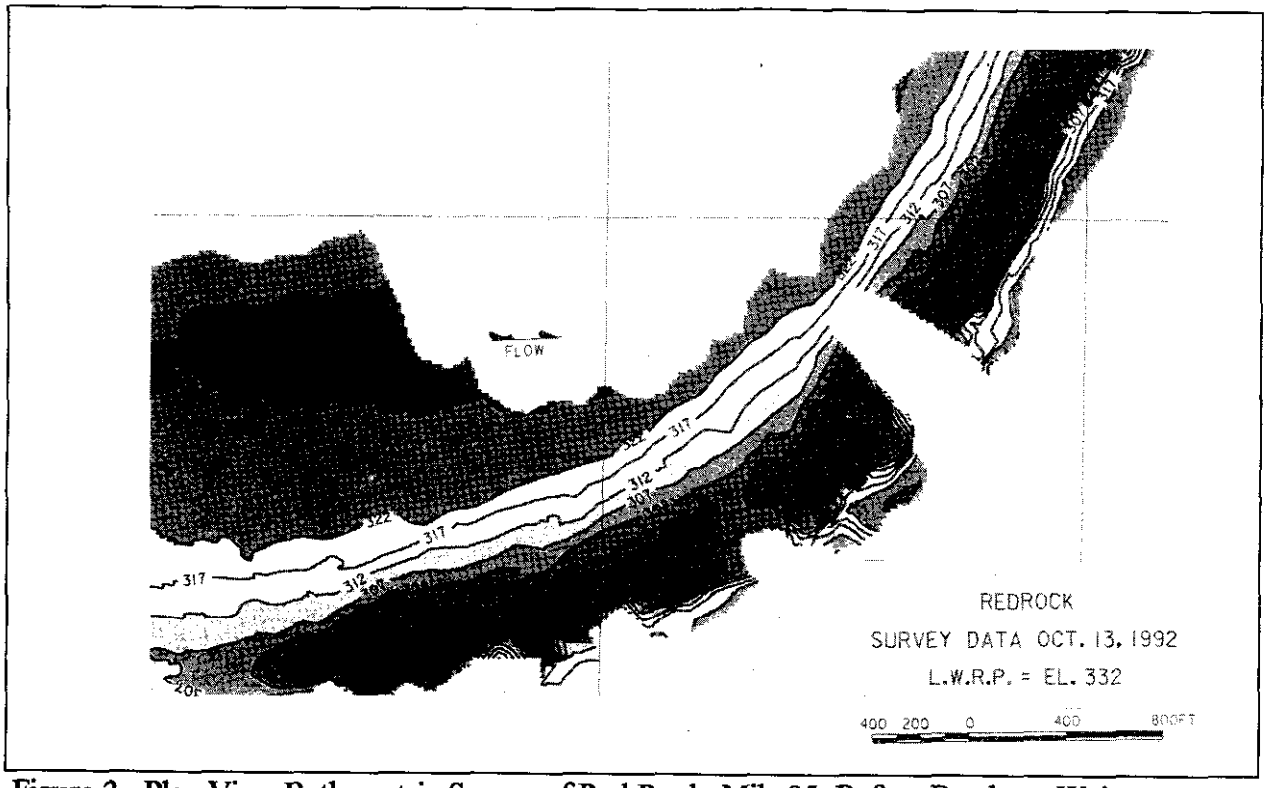

Figure 2. Plan View Bathymetric Survey of Red Rock, Mile 95, Before Bendway Weir Construction

Figure 3. Plan View Bathymetric Survey of Bendway Weirs at Red Rock, Mile 95, After Construction

SUMMARY AND CONCLUSIONS

The Bendway Weir concept is alive and well on the Mississippi River. Prototype trends are encouraging and verifying that the structures are behaving as expected in the prototype. Traffic accidents and dredging have been substantially reduced after construction.

Engineering monitoring will continue. Detailed design relationships will eventually be developed as more structures are placed in the prototype.

Environmental monitoring will continue. Physical parameters such as the bed configuration and velocity distribution will be correlated with environmental parameters such as fish population, size distribution, species type, and water column utilization.

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CHANGES ON A SMALL SIERRAN MEADOW STREAM DUE TO HIGH SPRING SNOWMELT FLOWS

By T.J. Myers, Research Associate, University of Nevada, Reno, Nevada; S. Swanson, Associate Professor, University of Nevada, Reno, NV

Abstract: We investigated stream morphologic changes due to high spring runoff on a mountain meadow stream in the northern Sierra Nevada mountains. The stream has high sedge (*Carex spp.*) density and height, sinuosity (k=1.6), slight entrenchment (floodable area greater than five times the channel width, low width/depth ratio (about 7.0), low gradient (S=0.017) and a gravel substrate. Sheep graze several one-day periods in August and September. Heights of sedge after grazing ranged from ten to fifteen cm, with taller plants in marshy areas. In 1993, the 185% of normal snowpack melted very slowly resulting in long durations of well above normal flow, but no high peaks. This high flow period occurred after five of six years with below normal flows. The soil-binding properties of the sedge, low velocities due to high roughness and easy access to floodplains should not allow major channel changes to occur. High flows should affect lower bank properties because of the longer duration of high velocity flow on apparent fresh deposits from the drought years. Embeddedness should decrease because of flushing and water depth should increase because of increased summer flows. We surveyed 98 transects at a six-meter spacing in 1992 and 251 transects at a two-meter spacing in 1993. We analyzed only transects at 24 meter spacings because of significant autocorrelation of channel width/depth ratio at twelve meter lags and because pool and substrate variables were measured over 24 meter stream reaches. There were 20 independent observations of all variables excepting bank angles, depths and undercuts for which there were 40 independent observations. Changes of channel width and width/depth ratio were insignificant, but gravel increased while cobble and pool area decreased significantly. Bank angle, depth, and undercut changes were insignificant contrary to expectation. Embeddedness increased significantly contrary to the expected decrease. Water depth increased, primarily in the reduced number of pools. Physically, the stream formed a pronounced pool-riffle sequence from one that was primarily a long, flat, shallow pool due to deposition. High flows caused a close to one-to-one pool/riffle ratio on the portion of the stream with random boulders spaced at five meter intervals. The long- duration high flows sorted sediment which removed gravel when forming pools and uncovered cobbles in the renewed riffles. The lack of cross-sectional change reflected the strength of high density sedge roots and the velocity decreasing roughness of sedges on the channel banks. Embeddedness increased primarily in deeper pools where sedimentation of fine suspended sediment was delivered from upstream of the meadow during base flow after the peak snowmelt. This mountain meadow provided an example of the strength of banks caused by dense herbaceous vegetation not apparently adversely impacted by relatively light sheep grazing.

INTRODUCTION

Many researchers have investigated stream channel morphologic changes due to low frequency floods (e.g., Newson, 1980; Bray, 1987; Costa, 1987; Myers and Swanson, accepted). They documented channel widening and incision and linked these effects to channelization (Bray, 1987), mining (Touysinhthiphonexay and Gardner, 1984), and natural basin geomorphology (Newson, 1980; Costa, 1987). Others investigated the role of vegetation in forming narrow, sinuous channels (Zimmerman et al., 1967; Smith, 1976) and showed that conversion to forested channels (Murgatroyd and Ternan, 1983) leads to wider channels. However, while much research focused on the process of channel widening by cantilever failure of upper banks when undercut (Thorne and Tovey, 1981; Pizzuto, 1984), few researchers focused on the role of vegetation in stabilizing these banks.

Stream channels require certain flows to maintain their shape and keep silt from depositing in the substrate (Rosgen et al., 1986). There is typically a dominant discharge with 1.5 to 2 year return interval associated with the channel shape (Pickup and Rieger, 1979; Carling, 1988). Pool formation by scour is frequently linked to this dominant discharge (Milne, 1982). However, few researchers focused on the role of drought and relatively high frequency, long duration, runoff events in shaping the channel.

In 1992, we established a long term study reach on a high mountain meadow in the Sierra Nevada Mountains. The objective of the site is to investigate the long-term evolution of channel morphology in a mature meadow.

In the spring of 1993, an approximate five-year snowmelt runoff peak following six years of drought occurred. However, the long duration of the runoff made this an unusual event. This event should have had major impact on the channel shape because of it long duration. The results presented here document that meadows with high density sedges and substantial overhangs have stable banks and that scour occurs on the channel bottom during long-duration, high-frequency flows.

STUDY SITE

Big Meadow is a wet mountain meadow in the Sierra Nevada Mountains about 20 km. southwest of Reno, NV. It is at 2500 m. above sea level. The meadow has high sedge (*Carex spp.*) density and height. Soils are silty sand. The small, meandering stream has high sinuosity (k=1.6), slight entrenchment (floodable area greater than five times the channel width), low width/depth ratio (about 7.0), low gradient (S=0.017) and predominately a gravel substrate. There are random boulders on both the channel bottom and in the banks. Based on embeddedness, the sediment supply is predominately suspended and results from open, coniferous woodland and jeep trails above the meadow. It is stream type E4 in the Rosgen (1994) classification system. The average channel width is 1.14 m. Approximately 700 sheep graze the meadow for several one-day periods in August and September each year. Heights of sedge after grazing ranged from ten to fifteen cm. in 1993 with taller plants in marshy areas that sheep avoid. The soil-binding properties of the sedge (Kleinfelder et al., 1992; Dunaway et al.,1994), low velocities due to high roughness and easy access of high flows to floodplains should resist major channel changes. High flows should affect lower bank properties because of the longer duration of relatively high velocity flow on apparent fresh deposits from the drought years.

FLOW HISTORY

The northern Sierra Nevada mountains and most of the Great Basin had flow years that were far below normal from 1987 to 1992 (Myers and Swanson, accepted). The snowpack of winter 1992-93 was 185% of normal in the Truckee River basin of which Big Meadow is a tributary. Runoff volume was well above normal, but the peaks were just a little above normal due to the low temperatures and lack of precipitation during the snowmelt period. Big Meadow does not have a flow gage, but there are several in the area. We gathered annual peak flow data from four gages within 40 km. that had drainage areas less than 30 km.^{2} and at least 20 years of record including 1993. We used the log-Pearson type III frequency analysis (WRC, 1981) with regional skewness coefficient equal to 0.3 to estimate a return interval for flows in Big Meadow in 1993. The return interval was 2.2, 4.8, 3.3 and 1.9 years for the Galena Creek near Steamboat, NV, Independence Creek near Truckee, CA, Sagehen Creek near Truckee, CA and Third Creek near Crystal Bay, NV (numbers 10348900, 10343000, 10343500 and 10336698), respectively. The Galena Creek flows were extremely skewed because of summer thunderstorm runoff in 1965 that exceeded all other flow events by a factor of ten. Using the spring flows for 1965, the 1993 flow had a 5.8 year return interval. Because of the low gradient nature of the watershed, it is unlikely that Big Meadow has high thunderstorm runoff. For these reasons, we estimate the return interval in 1993 on Big Meadow to be near 5.0 years.

The flow rate during the survey period in late July, 1992 was 0.00172 cms. In late August, 1993, the flow was only 10% higher measuring 0.00189 cms. During each year, the meadow had dried considerably and we assume the flow had reached baseflow in each year.

METHODS

Field Methods: We surveyed 500m. of the Big Meadow stream both years using transects. In 1992, we spaced 98 transects six m. apart; in 1993, we spaced 251 transects two m. apart. Across these transects, we measured the channel and water widths, depths at 1/4, 1/2 and 3/4 of the width and the maximum depths, stream bottom substrate categories (silt, sand, gravel, cobble, boulders), pool fractions of width, and bank angles, undercuts and water depths. From these measurements we calculated the average depth (average of depth at each bank and the three midchannel locations), channel width/depth ratio (ratio of channel width to the average depth) and entrenchment

were calculated according to:

$$FR = \frac{\sum_{i=1}^{n} f_i \times w_i}{\sum_{i=1}^{n} w_i}$$

where f is the transect fraction of pools, boulders, cobbles, gravel or silt, w is wetted width, i represents transects from 1 to n. The definition of pools followed Grant et al. (1990). At each transect, we also measured the length of bank dominated by general vegetation types (on this stream, either herbs or bare ground) and the length of bank determined to be stable or unstable (defined as the obvious presence of erosion or depositional material).

In 1992, we surveyed the profile of the reach. With this data, we calculated slopes for each subreach. In each year, we surveyed centerline coordinates from which we calculated sinuosity for any desired subreach.

Statistical Methods: In this study, the high flow rates of the spring, 1993, snowmelt runoff is the experiment. We tested the null hypothesis that there is no change from 1992 to 1993. Because there is no similar meadow nearby that did not receive high snowmelt runoff, but did receive similar management, we have no control for this study. Based on similar flow rates, the length of stream surveyed, and our avoidance of pseudoreplication explained below, our assumption that this high flow period caused observed changes is legitimate. However, we cannot rule out other influences that we are unaware of. For these reasons, to increase the power of the test, we accepted H_a if $\alpha \leq 0.1$.

Our comparisons between years were based on measurements along a stream. It is possible to increase n as much as desired, but the assumption of independence would be violated and the analyst would be guilty of pseudoreplication (Hurlbert, 1984). Much research addressing autocorrelation along a stream found that it decreased to insignificant levels at no more than two channel widths (Furbish, 1985). Prior to performing our comparisons, we determined the autocorrelation of channel width and width/depth ratio using our closely-spaced 1993 transects which were significant ($p \leq 0.05$) to the seventh and twelfth lag, respectively (Figure 1). Autocorrelation of bank angle, undercut and depth were also significant to the third lag (not shown). For these reasons, we utilized for comparison subsets of our data including every fourth (24 m. or 21 channel widths) and thirteenth transect (26 m. or 22.5 channel widths) in 1992 and 1993, respectively. Also, we calculated pool and substrate fractions using the same spacing for 19 adjacent reaches.

We compared normally distributed data (channel width and width/depth ratio and maximum and average depth) using two-sample T-tests (Sokal and Rohlf, 1981). We compared data which we could not consider to be normally distributed (pool area, substrate fractions and, for each side, bank angle, undercut and depth) with Mann-Whitney U-tests (Sokal and Rohlf, 1981). The U-tests yield a test statistic Z analogous to the t statistic in the two sample T-tests. The non-normal data was a proportion, bounded by 0 and 1, which approached the bounds. Finally, to investigate relations of pool area with geomorphic variables and changes with year, we performed a variety of multiple regression analyses to determine which, if any, variables best explained variation in pool area. We describe these analyses in the RESULTS section.

RESULTS

Cross-sectional Variables: Neither channel width or width/depth ratio varied between years ($t=-0.65, p=0.518$; $t=0.75$, $p=0.458$). The average channel width and width/depth ratio is 1.15 m. and 8.2, respectively. However, the maximum and average water depth increased by about 40% ($t=-2.19$, $p=0.035$; $t=-2.40$, $p=0.021$, Figure 2). The ration of maximum to average depth on each transect decreased about 20% ($t=1.88$, $p=0.068$, Figure 2). This indicates that a general scouring occurred that made the cross-sections more uniformly deep. In 1993, over all 251 transects, the depth ratio was higher on nonpool transects ($1.75>1.65$, $p=0.073$) which was a reversal from 1992.. This suggests that more scouring occurred in pools as the high flows removed sediments resulting from six years of drought.

Neither bank angle, undercut or depth changed between years (Z=0.490, p=0.624; Z=0.085, p=0.932; Z=1.04, p=0.300). The median bank angle, undercut and depth were near 110°, 0 cm and 2 cm. The length of undercut bank along the reach, measured at each transect, increased from 28 to 30 %. This suggests the banks were stable during the long, high runoff period. The lack of an increase in bank depths, however, suggests that the general scouring of the stream bottom did not affect the bank depths.

Substrate Variables: Gravel fractions increased and cobble fractions decreased between the years (Z=2.16, p=0.031; Z=3.45, p=0.001; Figure 3). As expected, boulder fractions remained stable at about 0.025 (Z=0.029, p=0.978). However, silt fractions decreased insignificantly from 0.12 to 0.098 (Z=0.759, p=0.448). That silt decreased by near 18% but was insignificant reflected its wide range of values (Figure 4). We observed in 1992 many locations of silt deposition around sedges that had apparently impinged on the baseflow channel. In 1993, we observed less rooted vegetation within the wetted perimeter suggesting scour of sediment that had occurred around the sedges. Rooted vegetation occurred on half as many transects in 1993 (40% to 20%) and the fraction of those transects having vegetation decreased about 20% (.4 to .32). Embeddedness increased (Z=3.30, p=0.001; Figure 4) suggesting the higher flows carried more sediment than the stream could transport. The decrease of silt substrate suggests that erosion of drought induced deposition caused a suspended sediment loading in excess of the streams capacity to transport. Much additional embeddedess occurred in the pools suggesting that flushing of deposited silt settled downstream in pools.

Variation of Pool Area: Pool area decreased between the years (t=2.45, p=0.014) from about 0.77 to 0.60 (Figure 5). This is counterintuitive due to the increase in depth. Because of the many near vertical banks, the differences in pool area can not be explained by a decrease in edgewater or other low velocity surface water. Throughout the reach about 85% of all transects in both years were either pool or riffle. The decrease of pool area indicates a shift from pool to riffle sections. The removal of vegetation which caused shallow backwater pools was apparently responsible for this shift and the decrease in pool area.

We investigated the relation with stream morphology and potential pool forming features by performing a best subsets regression of pool area with gravel, cobble and boulder fraction, channel width/depth ration, channel width and year coded as an indicator variable. There is no coarse woody debris which is frequently responsible for pool formation (Myers and Swanson, 1994). Cobbles and boulders frequently cause pools to form (Myers and Swanson, 1995). We included gravel because in gravel bed channels, it is frequently sorted (Milne, 1982) indicating the presence of vertical meandering, or scouring. The chosen model is (adjusted R^2 = 0.456):

$$PR = 1.3 - 0.011\,BLDR - 0.0089\,COBBLE - 0.0034\,GRAV - 0.11\,CHWID - 0.22\,YEAR \qquad (1)$$

where PR is pool area, BLDR, COBBLE and GRAV are boulder, cobble and gravel fraction, respectively, CHWID is channel width and YEAR is the year indicator (0 for 1992, 1 for 1993). All coefficients were significant except for CHWID. We then dropped CHWID to yield (adjusted R^2 =0.439):

$$PR = 1.2 - 0.012BLDR - 0.0083COBBLE - 0.0035GRAV - 0.22YEAR \qquad (2)$$

All coefficients became more significant as a result of dropping CHWID. The coefficient of an indicator variable represents the difference in the mean of the dependent variable between conditions defining the indicator variable. However, because 0.22 is a larger decrease than the 0.17 actually measured, we then tested for interactions by multiplying the three substrate terms by YEAR and including them in equ. 2. The additional variables were insignificant and reduced the adjusted R^2 to 0.410. We settled on equ. 2 as the appropriate model to explain the variability in pool area.

Silt and embeddedness are often related to pool area because of deposition (Myers and Swanson, accepted). We found silt to be insignificant and the relation with embeddedness also required YEAR to reduce the variance so that the relation of pool area with embeddedness could be determined (adjusted R^2 = 0.289):

$$PR = 0.67 + 0.0064EMB - 0.27YEAR \qquad (3)$$

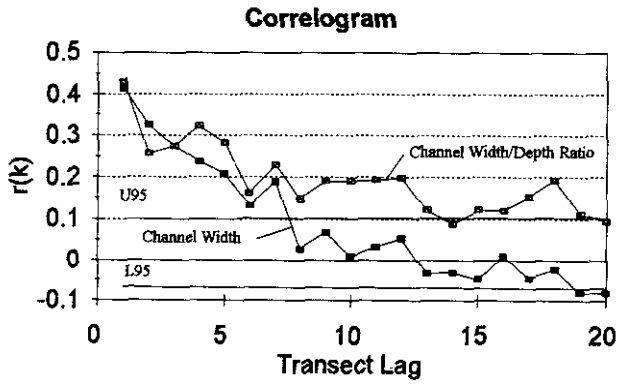

Figure 1 - Correlogram of channel width and channel width/depth ratio. r(k) is the correlation coefficient at k lags.

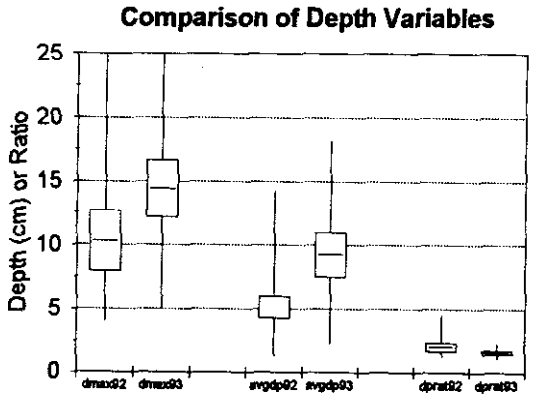

Figure 2 - Comparison of maximum transect depth (dmax*), average transect depth (avgdp*) and ratio of maximum to average (dprat*) between years (*). The bar represents the upper and lower 90% confidence limits, the vertical line is the range and the horizontal line is the mean.

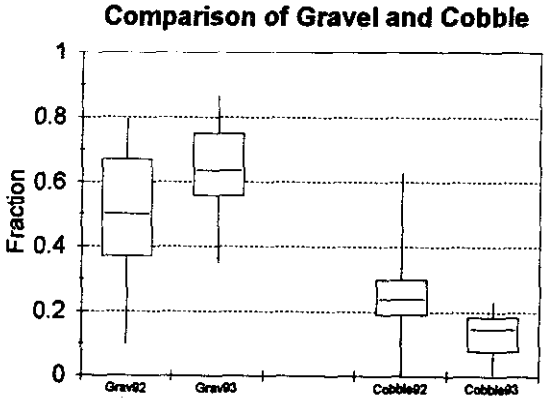

Figure 3 - Comparison of gravel (grav*) and cobble fraction (cobble*) between years (*). The bar represents the third and first quartile, the vertical line is the range and the horizontal line is the median.

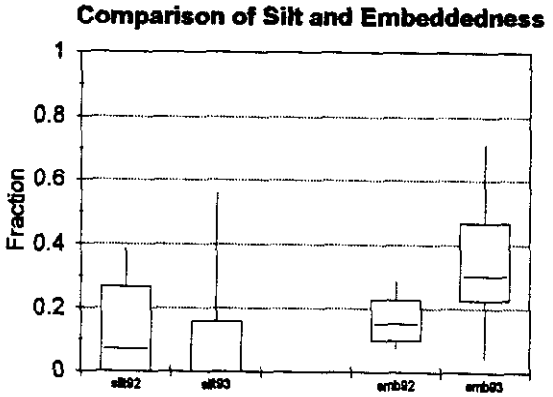

Figure 4 - Comparison of silt fraction and embeddedness (emb*) between years (*). The bar represents the third and first quartile, the vertical line is the range and the horizontal line is the median.

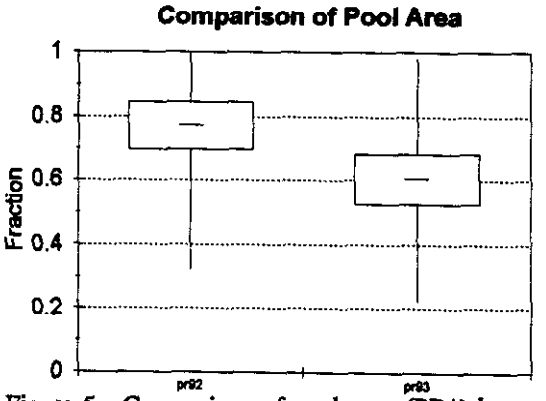

Figure 5 - Comparison of pool area (PR*) between years (*). The bar represents the upper and lower 90 confidence limit, the vertical line is the range and the horizontal line is the mean.

Here, EMB is the embeddedness fraction. We also tested for variation with gradient as found by other researchers (Wohl et al, 1992). The relation was (R^2 = 0.236):

$$PR = 0.94 - 9.7\,SLOPE \qquad (4)$$

where SLOPE is the gradient over the 26 m. for which PR was calculated. For each additional 1% of gradient, pool area decreased 0.097. Gradient ranged from 0.005 to 0.032, therefore, PR ranged about 0.26.

DISCUSSION

Pools became fewer and covered less area because high flows scoured previously filtered sediment and removed backwater causing vegetation. However, the pools became deeper relative to the rest of the reach because of this scour. Apparently after six years of drought, there was plenty of deposition to be removed. As seen by the increased embeddedness, primarily in pools, the sediment released by erosion exceeded that which could be moved through pools.

The fact that pool area decreases with all pool forming features in equ. 2 indicates that pool area in 1992 (PR=0.77) was too high for this stream. Shallow, flat water forms much less on reaches where cobbles and boulders direct and deepen the scour by increasing the velocity. That silt is not in equ. 3 indicates that the decrease with year coincides with the elimination of backwater from vegetation growing in silt. The decrease of pool area with gradient in equ. 4 is a further indication of the complex relationships among pools, velocity, silt and vegetation. Flow velocity and shear stress increase with gradient. There is less backwater with high gradient or velocity. High flows have a similar effect. Although we did not measure velocities during the high water, the relation of pool area with gradient is similar to the expected relation with velocity. Physically, the stream formed a pronounced pool-riffle sequence from one that was primarily a long, flat, shallow pool due to deposition and vegetation. High flows caused a close to one-to-one pool/riffle ratio on the portion of the stream with irregularly spaced random boulders. The long duration, high flows sorted sediment removing gravel when forming pools and uncovering cobble in the renewed riffles. This flow may have been a necessary channel maintenance flow (Rosgen et al, 1986).

The lack of change of the channel cross-section indicates that a long-duration, five-year flow event was unable to overcome the bank protection provided by dense sedges that had been lightly grazed. This protection occurred even on the 28% of the stream that was undercut suggesting that vegetation can resist (Kleinfelder et al. 1992) the expected cantilever type failure (Thorne and Tovey, 1981). The resistance is similar to that found by Smith (1976) for meadow grass and scrub willow. The results are similar to those of Rhoads and Miller (1991) who found that drought and a 100-year flood on a low-energy river had only subtle geomorphic effects. They attributed the lack of response to low stream power, low flow variability and cohesive banks.

However, we did observe that sheep closely cropped (to two to five cm.) the meadow away from the stream. This suggests that sheep do not "like to get their feet wet". Often, well-herded sheep cause few problems with streams or riparian areas (Glimp and Swanson, 1994). When designing grazing strategies for mountain meadows, managers should realize that light sheep grazing may not denude and trample meadow stream banks. With this fact, managers may want to set grazing objectives based on the more mesic meadow away from the stream.

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THE WAX LAKE OUTLET WEIR AND CHANNEL RESPONSE

by: Nancy Powell, Chief, Hydrologic Engineering Section,
U.S. Army Corps of Engineers, New Orleans, LA

Abstract: Flow and sediment are conveyed by the Atchafalaya River, through the basin floodway system, including the Grand and Six Mile Lakes and Atchafalaya Basin Main Channel, to two outlets, the Lower Atchafalaya River, a natural outlet, and the Wax Lake Outlet, an artificial outlet constructed by the Corps of Engineers in 1941. In 1988, the Corps of Engineers completed construction of the Wax Lake Outlet weir and connecting levees in the Grand and Six Mile Lakes area to control the amount of flow entering the Wax Lake Outlet. The purpose of the weir was to reverse a trend of aggradation in the principle outlet, the Lower Atchafalaya River, by reducing the flow conveyed by the Wax Lake Outlet from over 42 percent to 30 percent of the low to normal basin flows.

The weir was constructed in a shallow lake bed approximately 14,500 feet (ft) downstream from the Atchafalaya Basin Main Channel. The weir included a low flow notch on the left side of the channel, 1,100 ft wide, with a crest elevation of -2 ft NGVD. Prior to construction, the bottom elevation of the lake upstream of the weir averaged -8 ft NGVD. The channel width varied from 2,500 ft, at the confluence with the Atchafalaya Basin Main Channel, to 4,000 ft. Immediately after construction was completed, the channel upstream of the weir began to change. In the six years since the weir's completion, a sandbar developed upstream of the weir, reducing the channel area. A channel varying in width from 850 ft to 1,600 ft with an average bottom elevation of -8 ft NGVD formed in line with the weir's notch.

This paper will address the changes that have taken place in the channel upstream of the weir and examine these changes using guidance developed by the Corps of Engineers for analyzing the morphological response of a channel to channel modification.

INTRODUCTION

At the beginning of human settlement in the Atchafalaya basin region, an area of tidal lakes existed in the lower portion of the basin, the largest being Grand and Six Mile Lakes. By the mid-1800's, the Atchafalaya River was established as a distributary of the Mississippi River, transporting sediments to these lakes. Deltaic deposition commenced, facilitated by construction of a central channel through the lakes to increase the flood control capacity of the basin. Today, flow and sediment are conveyed by the Atchafalaya River, through the basin floodway system, including the Grand and Six Mile Lakes and Atchafalaya Basin Main Channel, to two outlets, the Lower Atchafalaya River, a natural outlet, and the Wax Lake Outlet, an artificial outlet constructed by the Corps of Engineers.

When completed in 1941, the original design capacity of the Wax Lake Outlet was 20 percent of the project flood flow in the floodway. Since its completion, however, the outlet enlarged at the expense of the Lower Atchafalaya River. Over time, the Wax Lake Outlet carried an increasing amount of flow. In the 1980's the distribution of low to normal flows exceeded 42 percent. The trend continued until 1988 when the New Orleans District constructed the Wax Lake Outlet weir and connecting

levees in the Grand and Six Mile Lakes area to control the amount of flow entering the Wax Lake Outlet. The purpose of the weir was to reverse a trend of aggradation in the principle outlet, the Lower Atchafalaya River, by reducing the flow conveyed by the Wax Lake Outlet to 30 percent of the low to normal basin flows. Figure 1 shows the Grand and Six Mile Lakes area and the weir.

The Wax Lake Outlet weir slowed or reversed the aggradation trend in the Lower Atchafalaya River as evident in channel surveys taken in the six years since its completion. However, with the increase in flow down the Lower Atchafalaya River came an increase in normal to high flood stages. The weir increased stages in an industrial area along the Lower Atchafalaya River riverfront by approximately 1 foot. This increased the frequency and duration of flooding in the riverfront area. Because of the flooding, the Corps removed the weir in 1994 and restored most of the area upstream of the weir to preweir conditions.

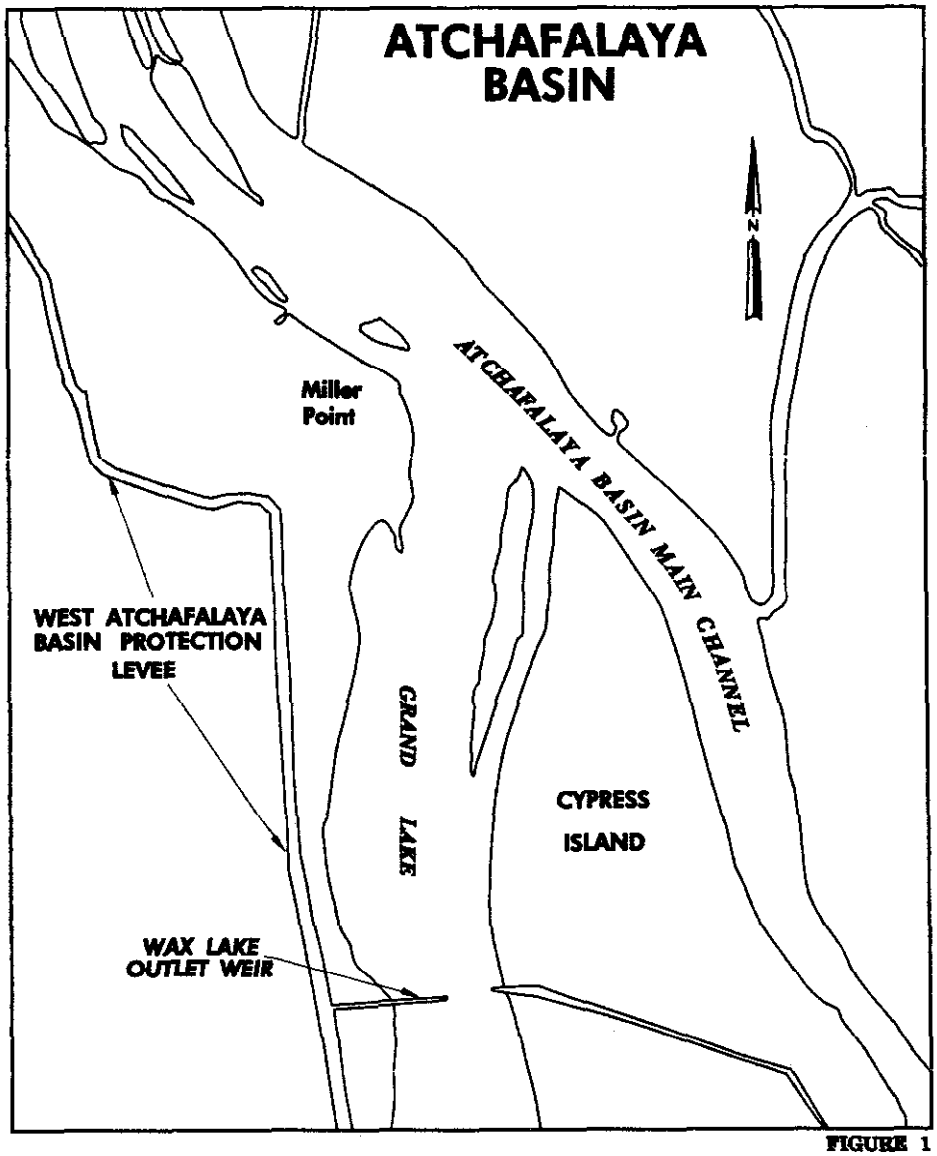

FIGURE 1

THE HYDROLOGIC SETTING

The Atchafalaya Basin floodway system is an integral part of the Mississippi River and Tributaries flood control system. The floodway system conveys a portion of the Mississippi River and all of the Red River flows to the Gulf of Mexico. The Atchafalaya River is a relatively young river with its growth and development basically commencing after removal of a log raft in 1855 at its upper end. The floodway system contains numerous flood control features, such as levees and channels, that were constructed by the Corps of Engineers after the 1927 flood. The average daily flow of the Atchafalaya River, as computed at Simmesport, approximately 100 miles north of the weir site, is 236,000 cubic feet per second (cfs). The average annual high flow is 450,000 cfs. Channel training works have been constructed along the Atchafalaya Basin Main Channel to confine this flow to the channel. The average daily suspended sediment load in the Atchafalaya River, as computed at Simmesport, is 221,000 tons per day. Approximately 20 to 25 percent of the suspended sediment is sand. The predominant bed material is sand; therefore, the sand load can be considered the bed material load.

In the Grand and Six Mile Lakes area upstream of the weir, the channel has undergone changes. Prior to the weir's construction, the width of the channel decreased; in 1950 the width varied from 4,000 to 10,000 ft, decreasing to 2,000 to 4,000 ft in the early 1980's. At the entrance, the cross section width changed from approximately 3,600 ft in 1963 to 2,300 ft. A major width adjustment took place between 1963 and 1971 with the formation of a longated island on the left side of the channel to the west of Cypress Island. (see Figure 1) The bed elevation changed from an average of -5 ft NGVD in 1963 to -8 ft NGVD in the 1980's prior to the weir's construction.

The weir, spanning approximately 3,100 ft, was constructed approximately 14,500 ft downstream from the Atchafalaya Basin Main Channel. It included a low flow notch on the left side of the channel, 1,100 ft wide, with a crest elevation of -2 ft NGVD. Approximately 1,100 ft of the weir on the right side of the channel was at elevation 7.5 ft NGVD. The remaining portion of the weir was at elevation 11.3 ft NGVD. Levees connecting the weir to the existing Atchafalaya Basin Protection levees were also constructed. The levees were designed to be overtopped when flows in the basin exceeded 550,000 cfs.

Immediately after its completion, the weir altered the distribution of flow between the two outlets from over 42 percent of the low to normal outlet flows to 30 percent. Progressive changes took place in the channel upstream of the weir after its completion. Over time, a longitudinal sandbar formed, bisecting the channel, and concentrating the flow to an area 850 to 1,600 ft wide and an average bottom elevation of -8 ft NGVD in line with the weir's notch. The height of the sandbar progressively increased to an elevation exceeding 8 ft NGVD in the middle of the bar by 1994. The overall channel area in Grand and Six Mile Lakes area upstream of the weir decreased by about 5,000 square ft. The growth of the sandbar did not appear to affect the quantity of flow entering the Wax Lake Outlet via the Grand and Six Mile Lakes area, as evident in flow data for the two outlets. In fact, the percentage of flow down the Wax Lake Outlet increased from 30 to 35 percent of the combined outlet flow. Downstream of the weir, the channel was scouring; a scour hole 10 ft deep formed about 1,000 ft downstream of the weir notch. Cross sectional area in this reach increased by about 2,000 square ft. On the Atchafalaya Basin Main Channel, channel widths and depths decreased

upstream of the entrance to the Grand and Six Mile Lakes area as evident in discharge data at Miller Point. Channel areas downstream of the entrance decreased somewhat.

Water surface elevations in the Atchafalaya Basin Main Channel upstream of the weir increased with the weir in place. At the Corps of Engineers gage at Myette Point approximately 4 miles upstream of the weir, the increase in stage for a given Atchafalaya Basin flow ranged from 0.5 ft for low to normal flows to over 1.0 ft for high water and flood flows. At the weir, model studies conducted by the Waterways Experiment Station showed an increase in water surface elevation ranging from 1 to 2.5 ft for the flows experienced in the with weir period. Gage records at the weir showed similar increases in water surface elevations.

Downstream of the weir at the Wax Lake Outlet at Calumet gaging station approximately 6 miles below the weir, stage, discharge, and suspended sediment measurements are made routinely. No significant variation exists between preweir and weir suspended sediment rating curves at Calumet. Any variation is well within the scatter of the data. The discharge stage rating curves also do not vary significantly.

Bed material data are available for the reach of the Atchafalaya Basin Main Channel in the vicinity of the Grand and Six Mile Lakes area. Comparison of 1975 and 1991 data, representing preweir and with weir conditions, shows a small increase in the median diameter of the bed material, from 0.20 mm to 0.25 mm. Almost all (> 99%) of the bed material is sand, with the majority of the material in the fine sand range. In the Grand and Six Mile Lakes area, prior to construction of the weir, the first 2 to 4 feet of bottom material was extremely fine. Boring crews were not able to successfully capture this material in borings taken at the weir site because of the material's fineness. Bed material samples taken in 1995 upstream of the weir show a median diameter between 0.003 and 0.07 mm, with the majority of the material in the silt and clay range.

CORPS GUIDANCE

EM 1110-2-1418, "Channel Stability Assessment for Flood Control Projects," provides guidance for determining potential channel instability and sedimentation effects in flood control projects. The approaches in the manual are mainly qualitative and serve to assist the engineer in estimating the type and magnitude of channel stability problems. The manual describes a systematic approach to channel stability involving data assessment and analysis. It focuses on the equilibrium concept of channel stability; stable channel width, depth, and slope can be expressed as functions of controlling variables such as discharge, boundary materials, and sediment supply. The two predominant driving variables are the inflow discharge and sediment hydrographs. A single discharge value is often used in hydraulic geometry relationships to represent the discharge hydrograph. Many terms have been used to describe this discharge, including channel forming discharge, bankfull discharge, dominant discharge, and effective discharge. EM 1110-2-1418 states that in natural streams, the channel forming discharge can be taken as equivalent to the bankfull discharge and usually has a return period of 2 years.

The concept of effective discharge combines two driving variables, flow and sediment. The Corps of Engineers' research and development program for flood control channels is involved in stable

channel design methods. Hey has performed an extensive literature review of channel response and channel forming discharge for this program. He describes the concept of effective discharge as the flow transporting the bulk of the sediment. Effective discharge is computed by determining the cumulative percent of time that a cumulative percent of the load was transported and establishing the associated discharge. A flow duration curve defines the frequency of events and a sediment rating curve defines the magnitude of each event. The curves are combined to calculate the total sediment load transported and to determine the effective discharge.

Lane, Leopold and Maddock, and others have investigated channel response to natural and human changes. Their studies support the following conclusions:

1. depth of flow is directly proportional to water discharge
2. channel width is directly proportional to both water discharge and sediment discharge
3. channel shape, as represented by the width to depth ratio, is directly related to sediment discharge
4. channel slope is inversely proportional to water discharge and directly proportional to both sediment discharge and grain size

Several qualitative equations have been developed from these conclusions. The charts and equations contained in EM 1110-2-1418 for the width-discharge relationship are based on a general relationship first formulated by Lacey,

$$W = CQ^{0.5} \qquad (1)$$

where
W = width
C = coefficient
Q = discharge

The coefficient C varies from 2.7 to 1.6 according to the nature of the channel banks. For sandy alluvial banks, the coefficient is 2.7. For average cohesive banks, the coefficient is 2.1, and for stiff cohesive banks, the coefficient is 1.6. The depth-discharge relationships in the EM are based on work presented in a report by Northwest Hydraulic Consultants. The depth-discharge relationships and the slope-discharge relationships contained in the EM are indicative for channels with low bed sediment transport and therefore may not be suitable for the Atchafalaya River.

In his research on stable channels, Lane looked at factors that may enter into a determination of stable channel shapes such as slope and the nature of material transported. He developed a qualitative expression for equilibrium:

$$Q_s D \propto QS \qquad (2)$$

where
Q_s = quantity of bed material load
Q = discharge
S = slope
D = bed material particle size

This equation can be an effective qualitative predictive tool to identify channel response. If one of the variables in the equation changes, any one or all of the other variables can change as a stream tries to restore equilibrium. A decrease in discharge can result in a decrease in depth. Channel width can reduce or remain unchanged depending on the availability of sediments. The slope of the channel can increase or remain unchanged depending on the sediment. If discharges increase, widths increase, depths increase, and slopes are reduced. If bed sediment inflows are reduced, depths are likely to increase and slopes to decrease. If the bed sediments are increased, depths decrease, and slopes increase. In these two cases, the expected change to width is unclear.

When a weir is placed in a channel, there can be changes in all the variables of Lane's expression. According to Hey, upstream of the weir, the decrease in slope is the controlling variable; a decrease in discharge, bed material load, and/or bed material size can be likely responses. Channel depths are likely to decrease; channel widths may reduce depending on the availability of sediment for deposition. Downstream of a weir, the controlling variable is the decrease in sediment load, which in turn causes the slope to decrease as the channel scours to increase its load. Flow may increase or bed material size may increase.

ANALYSIS OF CHANNEL CHANGES

In the area upstream of the Wax Lake Outlet weir, the slope, width, width to depth ratio, and discharge all decreased. Channel depth did not appear to decrease. The amount of bed material sediment transported through the reach decreased, as evident by the formation of the sandbar. The median bed material size in the Atchafalaya Basin Main Channel increased slightly; no data were available to assess the bed material in the channel upstream of the weir. Downstream of the Wax Lake Outlet weir, the channel responses observed for the most part agree with those determined from Lane's qualitative expression with a weir in place.

Using the techniques described by Hey, effective discharges were computed for the Atchafalaya River at Simmesport and the channel upstream of the weir for the preweir period (6 years prior to construction) and the with weir period (6 years after construction). Both suspended sediment and bed material (sand) rating curves were used. For the channel upstream of the weir, discharge and sediment data at the station Wax Lake Outlet at Calumet were used. It is assumed that the sediment load at Calumet is representative of the load in the channel upstream of the weir. Effective discharges were also developed from data for the Atchafalaya Basin Main Channel at Miller Point, upstream of the weir, using the sediment rating curves for Simmesport. The quantity of data at this site is small, however, and only covers the with weir period. Table 1 shows the effective discharges.

Using Equation (1) with a coefficient of 2.1 for cohesive banks and the effective discharge for suspended sediment from Table 1 for the channel upstream of the weir, a channel width of about 730 ft is determined for with weir conditions, or 85 percent of the observed width. The equation, witha coefficient of 2.7, results in a channel width of 1,620 ft at Simmesport, again about 85 percent of the observed width. Applying the equation to the Miller Point area with a coefficient of 2.1 greatly underestimates the width, however. Equation (1) yields a width of approximately 1,150 ft; the observed channel width for a flow of 330,000 cfs is around 3,350 ft.

Table 1
Effective Discharge

Preweir	Suspended Sediment	Sand Only
Simmesport	330,000	330,000
Miller Point	N/A	N/A
Channel above Weir	140,000	200,000
With Weir		
Simmesport	360,000	440,000
Miller Point	300,000	450,000 *
Channel above Weir	120,000	140,000

all flows in cfs
N/A = not available

* computations result in two peaks, one at 300,000 cfs and the second at 450,000 cfs the 450,000 cfs peak is selected here

Using the effective discharges for the with weir period from the sand discharge rating curves, channel widths computed for Simmesport and the channel upstream of the weir are within 6 to 8 percent of the observed channel widths. At Miller Point, the computed width is 42 percent of the observed.

DISCUSSION

The channel response upstream of the Wax Lake Outlet weir was similar to that determined using the guidance. Other factors, however, may have contributed to the channel response. As discussed in the Corps' EM, it is often difficult to determine to what extent post project instability represents a response to the project of whether it might have occurred in any case. Historically, the Atchafalaya River has been in an ongoing state of change; the width to depth ratio has declined continually since the introduction of a significant amount of flow and sediment to the river and lakes. Widths have decreased while depths have increased, either naturally or by human intervention as in the case of the construction of a central channel.

Long term and short term climatic changes can also affect channel response and make it difficult to assess the impact of a project using post project data. A climatic change where flow and therefore sediment are increased, similar to that experienced after construction of the weir, can result in rapid aggradation in response to the increase in sediment input. Upstream aggradation can increase the slope of the channel or reduce the bed material size which in turn increases the transport capacity of the channel. The net result is a shift in the effective discharge to one that is less than it had been during the previous period. The deposition upstream of the weir can also result in an opposite shift in the effective discharge. The reduction in sediment supply leads to erosion, which in turn increases channel gradients, flow depth, and sediment transport capacity. The erosion causes the channel slope to reduce and the bed material size to increase, thereby reducing sediment output. The effective discharge becomes greater than at the previous stable state.

In all probability, factors such as historical delta development and climatic change contributed to the changes that took place at Miller Point and in the area upstream of the weir in the six years following the weir's construction. This may in part explain the reemergence of a small sandbar at the site of the original sandbar after the 1995 high water season. As the flows entering the Wax Lake Outlet continue to change as a result of the weir's absence, changes in hydraulic geometry are expected. If the percentage of flow down the Wax Lake Outlet increases to 50 percent of the low to normal flows, the Corps guidance in the form of Equation (1) tells us that channel widths on the order of 850 ft can be anticipated. Time will tell if such a channel response is achieved.

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EROSION PROTECTION OF CHANNELS USING PERMANENT GEOSYNTHETIC REINFORCEMENT MATTINGS

David T. Williams, President,
WEST Consultants, Inc., Carlsbad, CA

Deron N. Austin, Supervisor - Engineering Services,
Synthetic Industries, Chattanooga, TN

Marc S. Theisen, Director - Geosynthetic Product Division,
Synthetic Industries, Chattanooga, TN

Abstract

This paper presents a complete state-of-the-practice procedure for the design analyses and selection of permanent geosynthetic reinforcement mattings as a channel lining system for grassed inland waterways. A computer program has been recently introduced to the erosion and sediment control industry that allows users to easily select the most appropriate geosynthetic matting for different conditions.

After inputting specific site parameters (*i.e.*, channel geometry, climatic data, and expected hydraulic conditions), the program calculates maximum expected velocity and shear stress conditions and selects the most appropriate product for the project. Independent calculations for the side slopes and channel bottoms are performed and different materials can be selected, if appropriate.

The procedure itself is modeled after the Federal Highway Administration's Hydraulic Engineering Circular No. 15 (HEC-15). The performance limitations for each product are defined and determined from results of extensive full-scale laboratory testing performed at a major western University. The final printout gives the user "stability" factors of safety against both velocity and shear stress failure for each product considered in the analysis. Engineering specifications and CAD cross-sections are provided for manipulation and incorporation into the user's construction documents.

INTRODUCTION

Function of Permanent Geosynthetic Mattings

Biotechnical Composites™ is a family of geosynthetic materials comprising non-degradable components that furnish temporary erosion protection, accelerate vegetative growth, and ultimately become synergistically entangled with living plant tissue to permanently extend the performance limits of natural vegetation used in channel stabilization. Exhaustive field and laboratory studies have verified that flexible, three-dimensional geosynthetic erosion mats can create a "soft armor" channel lining system capable of providing twice the performance of unreinforced vegetation (Theisen, 1991). Proven performance has allowed geosynthetic mattings to occupy one of the fastest growing niches in the erosion and sediment control industry (Austin and Theisen, 1994). Turf reinforcement is a method or system by which the natural ability of plants to protect soil from erosion is enhanced through the use of geosynthetic materials. A flexible three-dimensional matrix retains seeds and soil, stimulates seed germination, accelerates seedling development and most importantly, synergistically meshes with developing plant roots and shoots (International Erosion Control Association, 1992).

Classifications and Categories

Permanent geosynthetic mattings are composed of durable synthetic materials, stabilized against ultraviolet

degradation and inert to chemicals normally encountered in a natural soil environment. These mattings consist of a lofty web of mechanically or melt-bonded polymer nettings, monofilaments or fibers which are entangled to form a strong and dimensionally stable matrix. Polymers include polypropylene, polyethylene, nylon, and polyvinyl chloride (Theisen, 1991).

Geosynthetic mattings generally fall into three categories: Permanent Erosion and Reinforcement Matrices (PERMs), Turf Reinforcement Matrices (TRMs), or Erosion Control Revegetation Matrices (ECRMs). PERMs consist of UV stabilized polypropylene monofilament yarns oven into a dimensionally stable, uniform configuration of resilient pyramid-like projections. These products are designed for challenging erosion control applications where high strength, durability, and long-term performance are essential. Woven three dimensional geotextile matrices provide many times the strength of traditional geosynthetic mattings. The rugged construction, durability, and functional longevity differentiate woven geotextile matrices from the variety of turf reinforcement mattings (TRMs) currently available. The resilient structure of PERMs provides maximum resistance to construction and maintenance induced stresses (Austin and Johnson, 1994). TRMs provide sufficient thickness and void space to permit soil filling and retention and the development of vegetation within the matrix. TRMs are installed first, then seeded and filled with soil. Seeded prior to installation, ECRMs are denser, lower profile mats designed to provide long-term ground cover and erosion protection. ECRMs rely upon sediment capture for increased long-term stability. By their nature of installation, PERMs and TRMs can be expected to provide more vegetative entanglement and better long-term performance than ECRMs. However, denser ECRMs may provide superior temporary erosion protection (Theisen, 1991).

Scope

This paper describes the procedures used for the hydraulic analyses and selection of a permanent geosynthetic matting as channel lining materials featured in a new computer program. EC-DESIGN™ is a complete erosion control design package recently published by Synthetic Industries, Construction Products Division. The program allows the user to select the most appropriate synthetic erosion control material for either his/her slope or channel application. The materials available for selection in the program consist of polypropylene roving, open weave geotextile meshes, and a wide variety of permanent geosynthetic reinforcement mattings. This paper will describe only the channel lining, design and selection process for permanent geosynthetic mats. Slope erosion control analyses will not be covered.

Detailed design guidelines, justification and supporting technical information can be found in the EC-DESIGN™ Users Manual. The program can be executed with English or Metric units.

BACKGROUND

Market Growth

New erosion and sediment control legislation, coupled with enhanced public awareness toward environmental issues, has led to a rapid increase in the use of flexible geosynthetic lining systems as lining materials in inland waterways. Permanent geosynthetic mattings are specified by some designers in channels receiving maximum flow velocities from two to six m/sec and shear stress or tractive forces up to 39 kg/m^2. These materials are being selected as alternatives to rigid linings because of several advantages they offer. Geosynthetic mattings:

- Allow vegetative establishment
- Impede water flow/capture sediment
- Promote infiltration/groundwater recharge
- Are easy to install
- Extend performance limits of natural vegetation
- Reduce sediment transport
- Conform to uneven subgrades
- Offer greater than 50% cost savings

Review of HEC-15

Published by FHWA in 1988, HEC-15 endorses flexible lining materials by providing step-by-step, well-documented design procedures (Chen and Cotton, 1988). Limiting shear stress design values for bare soil, rip rap, vegetation, jute, roving, natural erosion control blankets, and synthetic mats are given to assist the user in selecting the most

appropriate material. However, in this day and age of microcomputers, the tables, procedure, and diagrams are somewhat cumbersome and time-consuming Lastly, the permissible shear stress of 9.76 kg/m^2 is a drastic underestimation of the performance of permanent geosynthetic reinforcement mattings.

Dozens of well-documented laboratory investigations, field studies and case histories have demonstrated performance limits of reinforced vegetation that far exceed guidelines given in HEC-15 (WCHL, 1979; Hewlett, *et. al*, 1987; Theisen, 1992; Dodson, 1990; Hoffman and Adamsky, 1982; Carroll *et. al*, 1991; Keller and Middlebrooks, 1989). In fact, biotechnically-reinforced systems have resisted velocities in excess of 4.3 m/sec and shear stresses in excess of 35 kg/m^2 for durations of two days (50 hours), providing twice the erosion protection of unreinforced vegetation (Carroll, *et al.*, 1991; Hewlett *et al.*, 1987; Theisen, 1991, Theisen, 1992). The need for this computer-aided channel lining design program has been established by the discrepancies and cumbersome nature within HEC-15.

GENERAL OVERVIEW

Organization of Computer Program

The program has six main menu options. They are as follows:

Designs allow you to enter the program section that determines the geosynthetic material requirements for channel and land slope protection. It also allows you to export and import project and run files for use with others who have the program.

Reports lists all projects and/or runs within each project and allow you to send the list to the screen, printer, or write to a computer file.

Options allows you to reindex the database, activate the screen saver, gives you access to the *Preference* submenu. The *Preference* submenu allows you to specify the default printer to be used, the color scheme for the display, and activation and time limit before activation of the screen saver.

Help permits you to access the help descriptions of any input field, the notepad editor for each project and the on-line calculator.

Specs is used to obtain specification information on the geosynthetic materials used in this program. It can also be used to display the specifications or output them to a printer or computer file.

About Synthetic Industries

provides the user information about Synthetic Industries, its sales offices, and general information about the geosynthetic materials used in this program.

When the program is installed, it creates a subdirectory called DATA. This subdirectory contains all the project and run information.

Computer Requirements

The computer must have the following minimum requirements to efficiently execute the program:

- 512 kilobytes of conventional RAM
- 286 or greater CPU
- 30 files, 20 buffers specified in the CONFIG.SYS file
- 3 megabytes of hard drive memory
- MSDOS or PCDOS 2.1 or higher

Although a math co-processor is not required, it greatly decreases the execution time of the program.

CHANNEL LINING COMPUTATIONS

Manning's Equation

The hydraulic computations performed in the program are based on the assumption of uniform flow. In uniform flow, the channel dimension, flow depth, and discharge are considered to be constant along the channel. For design purposes of this nature, uniform conditions are usually adequate. Under uniform flow conditions, the hydraulic parameters can be determined by the Manning's equation (Chow, 1959).

$$Q = \frac{KAR^{2/3}S^{1/2}}{n} \tag{1}$$

Where:

K	= 1.0 for Metric units	Q	= Discharge in cubic meters per second
A	= Flow area in square meters	S	= Energy slope or channel slope
n	= Manning's roughness coefficient	R	= Hydraulic radius, in meters = Area/Wetted Perimeter

If the bottom or sides of the channel have different "n" values, a composite "n" value is determined for use in the Manning's Equation. This is computed by the following equation (Chow, 1959):

$$n_c = \frac{(WP_l n_l^{1.5} + WP_b n_b^{1.5} + WP_r n_r^{1.5})^{2/3}}{(WP_t)^{2/3}} \tag{2}$$

Where:

n_c, n_l, n_b, n_r	=	Manning's "n" of the composite, left side slope, bottom and right side slope, respectively
WP_l, WP_b, WP_r, WP_t	=	Wetted perimeter of the left side slope, bottom, right side slope, and total channel, respectively, in meters

Manning's Roughness Coefficients, "n"

The allowable values of Manning's "n" are between 0.01 and 2.0. If the entire wetted perimeter is initially selected to be protected with a geosynthetic mat and soil filled, an "n" value of 0.020 is automatically assigned for the analysis. If the mat will not be soil filled or a fully vegetative system is desired to be analyzed, Table 1 can be used to select the appropriate "n" value. The shaded boxes indicate Manning's "n" for soil filled, vegetated conditions.

Shear Stress

The force of the water on the channel boundaries is known as the tractive force or shear stress. Stable channel lining design with permanent geosynthetic mats is based on the premise that flow-induced tractive force must not exceed the critical tractive force (shear stress) of the mat. The average shear stress on the channel is determined by the following equation (Chen and Cotton, 1988):

$$\tau_o = wyS \tag{3}$$

Where:

τ_o	=	Average shear stress on the channel, in kg/m^2
w	=	Unit weight of water, in kg/m^3
y	=	Flow depth, in meters

Table 1. Recommended Manning's Roughness Coefficients* for Geosynthetic Mattings
(modified after Chen and Cotton, 1988)

Geosynthetic Matting Type	Flow Depth Ranges		
	0 - 15 cm	15 - 60 cm	> 60 cm
ECRM	0.035	0.025	0.021
	0.023	0.020	0.020
TRM	0.036	0.026	0.030
	0.023	0.020	0.020
PERM	0.038	0.028	0.024
	0.023	0.022	0.021

* Once vegetation has exceeded 40% ground cover, the "n" value of the species for the expected height dominates.

Channel Bottom

For wide channels, the shear stress obtained from the above equation is the same as the shear stress on the channel bottom, τ_d. Shear stress is not evenly distributed over the channel and is sometimes affected by the shape of the channel. For channels with a ratio of bottom width to flow depth (b/y) less than 10 and side slopes less than 4, a correction to the shear stress, τ_o, must be made to obtain the shear stress on the channel bottom (French, 1985). The shear stress on the channel bottom is then obtained by:

$$\tau_d = C_c(\tau_o) \tag{4}$$

Where:

τ_d = Channel shear stress, in kg/m^2
C_c = Correction factor of channel bed shear stress due to channel shape - obtained from HEC-15

If B/Y is greater than 10 or the z (or H) of any side slope is greater than 4, C_c is 1.0.

Channel Bends

For bends, the bottom shear stress must be corrected because of the concentration of flow on the outside of the bend. A bend is considered significant if the radius of curvature divided by the bottom width (R_c/B) is less than 10. This correction, K_b, is obtained from HEC-15 (Chen and Cotton, 1988). The bottom shear stress due to bends is determined by the following equation:

$$\tau_{bd} = K_b(\tau_d) \tag{5}$$

Where:

τ_{bd} = Shear stress on the channel bottom due to a bend, kg/m^2
K_b = Correction factor of channel shear stress due to a bend

Side Slopes

In general, the shear stress on channel side slopes is less than the shear stress on the channel bottom. This is

determined by multiplying the channel shear stress by a factor related to B/Y and the side slope, z, (or H). The factor, termed C_s, is obtained from HEC-15 (Chen and Cotton, 1988).

The shear stress for the sides is determined for both sides. The side slope shear stress, τ_s, is then determined by the following equation:

$$\tau_s = C_s(\tau_{dx}) \tag{6}$$

Where:

τ_s = Side slope shear stress, kg/m^2
C_s = Correction factor of shear stress for side slopes
τ_{dx} = τ obtained from either Equations 4 or 5, whichever is applicable

The correction to the side slope shear stress for bends is made only to the outside bend side.

Water Velocity in Channel Bottom

If B/Y is greater than 10 and there is no bend, the average velocity of the channel is used as the velocity on the channel bottom. If there are corrections to the shear stress because of the channel geometry and/or bends, the corrected water velocity of the channel bottom is:

$$V_d = \frac{K_s R^{1/6} \tau_{dx}^{0.5}}{n_c} \tag{7}$$

Where:

K_s = Conversion for shear stress to velocity, 0.0316 for Metric units (Chen and Cotton, 1988)
V_d = Water velocity for the channel bottom, meters per second

Equation 7 is obtained by dividing both sides of Equation 1 by A (flow area) to obtain velocity (Q/A = velocity). Equation 3 is then solved for R and substituted into Equation 1.

Water Velocity on Side Slopes

The water velocity on the side slopes is determined by the following:

$$V_s = \frac{K_s R^{1/6} \tau_s^{0.5}}{n_c} \tag{8}$$

Where:

V_s = Water velocity for the side slopes, meters per second

GEOSYNTHETIC MATTING SELECTION PROCESS

In general, for a given channel geometry, hydraulic condition, and vegetative condition, the program determines the flow velocities and shear stresses on the channel bottom and each side slope. It then compares these values to the critical values for each geosynthetic matting by dividing the critical velocities and shear stresses by the actual velocities and shear stresses of the proposed lining material, resulting in stability factors. If the stability factor is greater than 1, the mat is adequate for erosion protection. This analysis is performed for the channel bottom and each side slope, allowing the user to select different geosynthetic mats for each channel segment. The program takes into consideration the expected life of the project in relation to the geosynthetic performance life and the increased hydraulic effects of flow around a bend. HEC-15 suggests that the shear stress is the most important parameter for stable channel design (Chen and Cotton, 1988). However, there are occasions, such as high velocities with low flow

depths, where the flow velocity is the controlling factor. Because of this, the program considers both shear stress and flow velocity in its analysis and allows the user to determine which method to use for the selection of the geosynthetic mat.

The maximum velocity and shear stress in the channel from the design storm event is calculated. The worst-case scenario for the mat is considered and the maximum permissible hydraulic properties of the geosynthetic mats are compared. By taking into account the required minimum factors of safety, the most appropriate permanent geosynthetic mat is selected. The maximum hydraulic properties are presented in Table 2. These values are published as a result of extensive testing at a leading western University (Theisen, 1991). The thrust of this study was to develop design guidelines for mats when placed over bare soil, filled with soil, and completely vegetated. The results of the study are the basis for the geosynthetic mat selection process. The critical shear stress or velocity of the mats is at the hydraulic condition when the product has "failed", where failure is defined as any of the following conditions:

1. Tractive forces tore significant chunks of sod from channel floors or from the reinforcing mats.

2. A mat was torn or its structural makeup was altered significantly due to tractive forces of the water flowing over it.

3. The channel itself was eroded significantly in spite of the protective vegetation and/or mats placed in them.

Table 2. Maximum Permissible Design Values for Geosynthetic Mattings

PRODUCT	VELOCITY (m/sec)		SHEAR STRESS (kg/m^2)	
	SHORT TERM	LONG TERM	SHORT TERM	LONG TERM
PERM, Non-Vegetated	6.1	3.1	39.1	14.7
PERM, Vegetated	7.6	4.6	48.9	29.3
TRM, Non-Vegetated	4.3	2.4	29.3	9.8
TRM, Vegetated	6.1	4.3	39.1	24.5
TRM (w/Geotextile), Non-Vegetated	3.7	2.1	24.5	9.8
TRM (w/Geotextile), Vegetated	6.1	4.3	39.1	24.5
ECRM, Non-Vegetated	5.5	2.4	34.2	9.8
ECRM, Vegetated	5.5	3.1	34.2	19.6

Stability Factor

The stability factor is computed by dividing the maximum permissible shear stress or velocity of the geosynthetic matting by the computed shear stress or velocity. Note that the computed shear stresses and velocities have already been multiplied by a factor of safety. This computation is performed for the channel bottom and side slopes if they are to be protected by a geosynthetic mat. If the stability factor is greater than or equal to 1, the matting system is considered stable. If the stability factor of a desired material is just under 1.0, the designer may consider changing the initial design, such as increasing the bottom width or flattening side slopes, to achieve the desired results.

Final Selection and Output

In the program output, the shear stress and velocity stability factors for each mat is presented for the channel bottom and side slopes. The program then lists those materials that have stability factors greater than or equal to 1.0, for both shear stress and velocity, and have an expected performance lifetime greater than or equal to the project life.

SUMMARY

A user-friendly, state-of-the-practice computer-aided design procedure has been developed. The program follows FHWA guidelines for the design of flexible channels, but incorporates more realistic, well-documented maximum shear stress and velocity values for user flexibility. This paper has documented the background, organization input parameters, channel computations, and selection of the most appropriate permanent geosynthetic reinforcement matting for the vegetated channel lining system.

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Design of Low-Flow Channels in Sand Bed Streams

J. C. Fischenich, Research Civil Engineer, USAE Waterways Experiment Station
3909 Halls Ferry Road, Vicksburg, MS 39180

Abstract: A procedure for the first-order approximation of a stable channel form based upon physical reasoning, empirical evidence, and constraints common to low-flow channel projects is presented. Simplifying assumptions, limitations and applicability are discussed.

INTRODUCTION

Restoration of physical aquatic habitat in streams during critical periods can often be accomplished with low-flow channels designed to concentrate flows and increase channel velocity and depth. Structural measures such as bank protection, flow deflectors and sills are generally required to stabilize the channel and to provide habitat diversity. In addition to the usual design requirements for these structures, a structural layout that allows the stream to adopt a form consistent with the project objectives is needed. Projects are all too often formulated with little or no thought given to the placement of structures and, as a consequence, structural and channel stability are compromised, and habitat benefits not realized (Fischenich *et al.* 1994). Designers must translate physical habitat objectives into hydraulic and geomorphic criteria, which, in turn are used to select structural measures and establish a stable project design. Unfortunately, no procedure to accomplish this has been proposed.

In this paper, a stable channel is defined as: a channel that, over an engineering time scale, retains its planform, cross-sectional geometry and grade with allowance for temporal variability about a mean that displays no trends. Existing techniques for predicting the hydraulic geometry of stable alluvial channels can be placed into two categories: 1) empirical regime equations derived from regression analyses of observed channel geometry, and 2) analytical models that attempt to model rivers on a rational basis using theoretical considerations. Knighton (1984), Chang (1988), and Mueller and Dardeau (1990), among others, present summaries of many of these techniques. The early regime equations generally relate some two-variable combination of velocity, width, depth, slope, area, hydraulic radius, perimeter or discharge as a power function and can be generalized in the form of Equations 1 through 3, where by continuity, ack=1 and b+e+m=1:

$$W = aQ^b \tag{1}$$

$$D = cQ^e \tag{2}$$

$$V = kQ^m \tag{3}$$

Analytical procedures include the method of maximum permissible velocity, the critical shear stress method, and the so called "rational" regime equations that use tractive force theory, resistance and sediment transport laws, minimum stream power, maximum sediment transport capacity, similitude, bank stability analyses and other concepts to overcome the fact that alluvial channels have more degrees of freedom than available relations for solution. Wargadalam (1993) showed that the existing body of

literature on the subject can be reduced to no fewer than 176 equations describing depth, width, velocity and slope in the form:

$$W, D, V, or S_0 = a_1 Q^{b_1} d_s^{c_1} Q_s^{e_1} \tag{4}$$

Despite the considerable effort expended in formulating these many approaches to stable channel geometry prediction, none are suitable for the development of stable low-flow channel designs. The reasons for this include: 1) no consideration of habitat requirements, 2) empiricism, 3) applicability only to straight channels, and 4) theoretical flaws. Perhaps the greatest limitation of many of these techniques is that they consider width, depth and slope to be the only degrees of freedom and do not account for the full dynamic nature and multi-dimensionality of fluvial systems. A minimal description of channel morphology requires that its grade, planform, cross-section geometry and some measure of the channel resistance be parametrically defined. Channel grade can be described by the bed slope, S_o, and resistance by the friction slope, S_f. Channel planform requires at least two parameters, sinuosity and meander arc length for example. To uniquely define the cross-sectional geometry of a channel requires that at least one variable in addition to the width and depth be specified. Maximum depth, bank slope and a channel shape factor are examples of a third variable that may be considered. Figures 1 and 2 show the parameters used in describing channel geometry and planform in this paper.

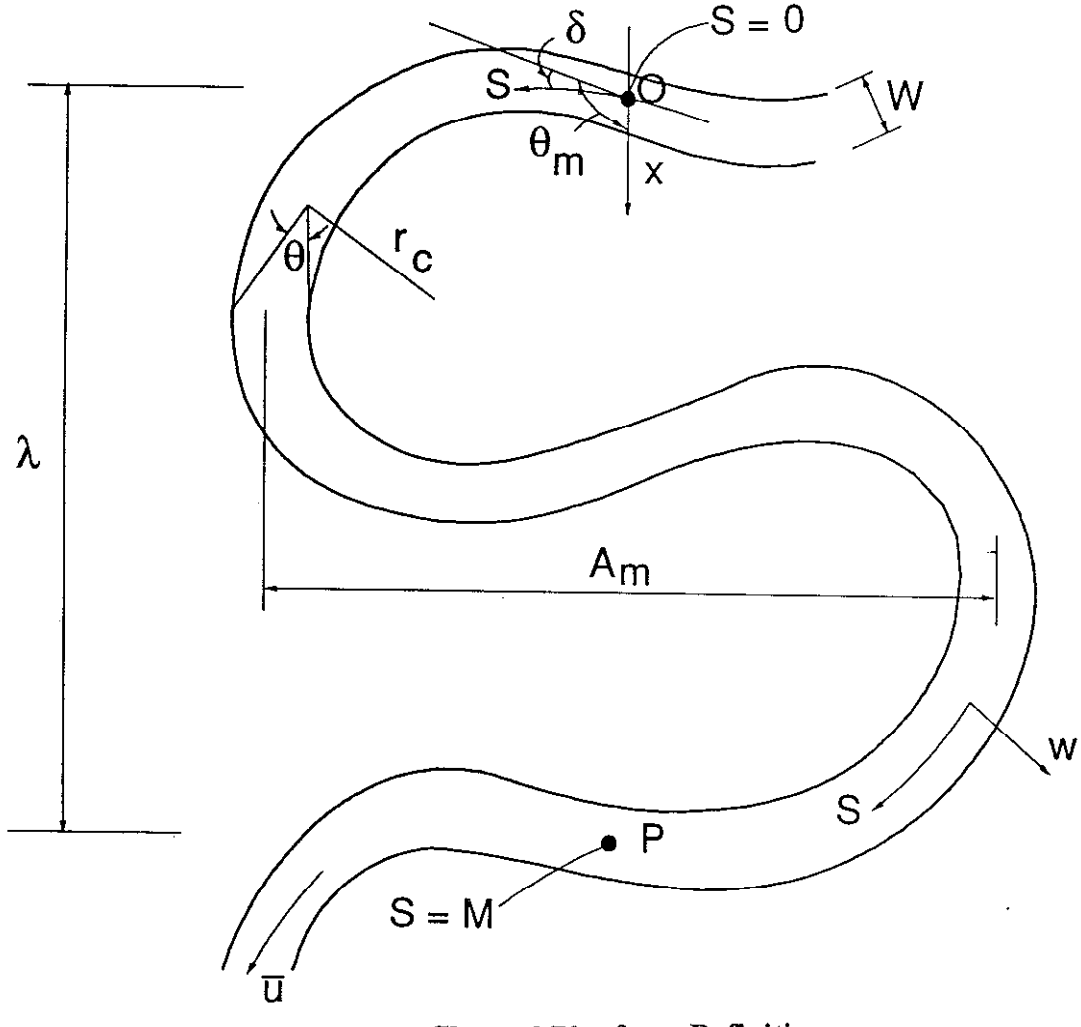

Figure 1 - Channel Planform Definition

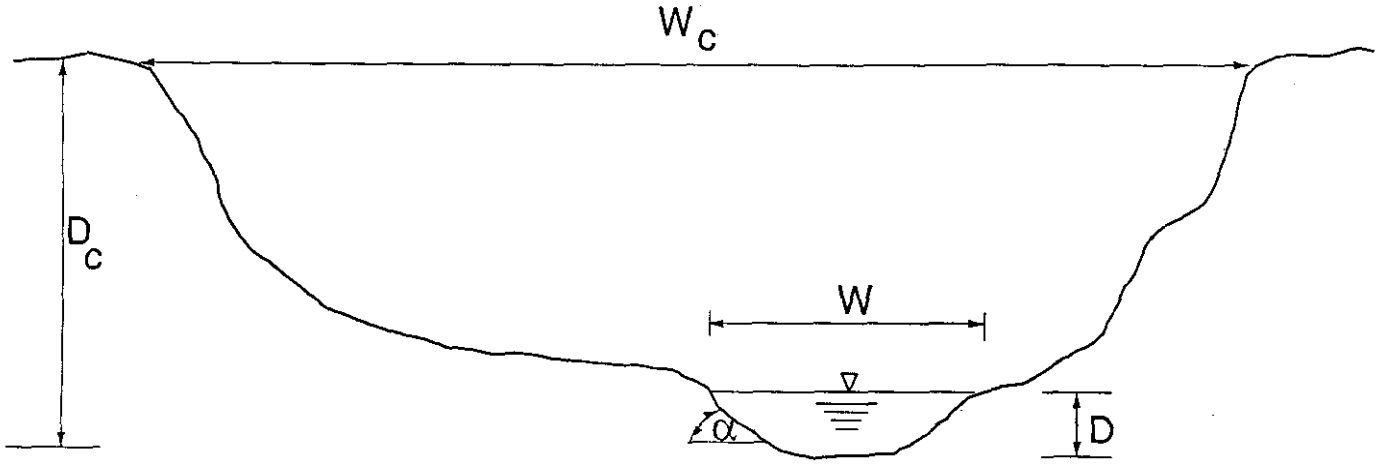

Figure 2 - Channel Cross Section Definition

PROCEDURE FORMULATION

Fundamental Considerations: Both cartesian and curvilinear coordinate systems are used in the formulation of the procedure. The principal axis in the cartesian system x defines the center line of the meandering pattern in the downstream direction for planform analysis and is aligned with the stream and horizontal for cross section and grade analyses. The y and z axis alignments are with the vertical and horizontal, both normal to the flow. In the curvilinear system, the sinuous axis s follows the centerline of the stream, and the transversal direction remains orthogonal to the principal axis in the horizontal.

Analytical determination of the hydraulic geometry for alluvial channels can be accomplished only if applicable physical and empirical relations are sufficient to describe the unknowns or degrees of freedom. Two characteristics of low-flow channels help satisfy this requirement. First, habitat analyses are used to establish a minimum flow depth and an optimum velocity for the channel based upon biological criteria. Second, instream and streambank structures are typically used to stabilize the channel, provide cover and substrate, and generate diversity in velocity. Given these constraints, physical reasoning, geometric analyses and a few semiempirical relations, a solution is possible.

A fundamental assumption in the application of this procedure is that sand bed channels tend toward a meandering planform that can be described by a sine-generated curve. Langbein and Leopold (1966) used the theory of minimum variance to show that the most probable path of a meandering stream was a sine curve. Julien (1985) showed that the fundamental shape of meandering planforms can be evaluated based on the separation of the boundary layer near the inner bank and the rate of energy dissipation. Julien demonstrated that among all possible geometries, with the exception of a straight channel, boundary separation and the rate of energy dissipation is minimized by a simple sine-generated function. Approaches based upon the Kinoshita equation, Von Schelling's curve and Fargue's spiral can also be reduced to a form of a sine-generated curve. The sine-generated function has thus been shown to fit the shape of meanders on stable rivers quite well and is the most convenient expression to use. The corresponding meandering pattern, with reference to Figure 1, can be written as:

$$\frac{d\theta}{dx} = -a_1 \sin \frac{2\pi s}{M} \tag{5}$$

A second fundamental assumption is that the flow is approximately uniform in the streamwise direction. This assumption allows the bed, water and friction slopes to be equated and permits the use of a uniform rating equation. The final assumption relates to sediment continuity, which must be preserved to meet the stable channel definition. The proper approach to assure that continuity is met is to perform both longitudinal and transversal sediment transport analyses. A simplified approach, wherein continuity is approximated by setting the bed shear stress at the critical value for the mean bed sediment size, is used.

Procedure: Because low-flow channels are developed for environmental purposes, the first step is the conduct of hydrologic analyses to establish the target or design discharge. Instream Flow Incremental Methodology (IFIM) analyses, habitat suitability curves, or other biological criteria are then used to establish target values of velocity and depth. Realizing the optimum values for both the depth and the velocity objectives is unlikely; thus, one must be established as a critical parameter. The most common case, in which a critical minimum depth is established and an optimum velocity target identified, is used here. Simple modifications to this procedure will accommodate other cases, such as a maximum velocity and an optimum depth.

Grade: The criterion of beginning of motion establishes a threshold corresponding to a condition of maximum allowable shear. The ratio of forces on an individual sediment grain on a sloping bed is evaluated to define the dimensionless shear stress called the Shields number:

$$\tau_* = \frac{\gamma R_h S_f}{(\gamma_s - \gamma) d_s} \tag{6}$$

The critical value of the Shields number, τ_{*c}, corresponding to the beginning of motion was determined by Shields (1936) and, for particle Reynolds numbers greater than 200, $\tau_{*c} \cong 0.05$. The widely accepted Shields diagram is used as a basis for many stable channel design techniques despite complexities encountered in its application to irregular channel geometries. Given the depth constraint and measurements of the sediment size, the maximum allowable friction slope for longitudinal stability can be computed from Equation 6 by setting the hydraulic radius equal to the minimum depth and setting $\tau_{*c} = 0.05$. For sands of specific gravity G = 2.65, this relation reduces to:

$$S_f = 0.0825 \frac{d_{50}}{D} \tag{7}$$

For uniform flow, channel grade can be equated to the friction slope, and either can be related to the sinuosity of the low-flow channel and measurement of the bed slope of the parent channel S_c:

$$S_o = S_f = \frac{S_c \lambda}{M} \tag{8}$$

While this must be the reach-wise average bed slope of the low-flow channel, variation about this average to accommodate pool-riffle sequences is possible provided the resultant reaches have the requisite transport capacities to ensure no aggradation or deposition. This first approximation of the channel grade ignores transversal stability, which can be incorporated into the analysis as discussed later.

Planform: With the allowable friction slope computed by Equation 7 and a measured thalweg slope of the parent channel, Equation 8 is used to establish the ratio of the meander wavelength to the arc length. Again, assuming a sine-generated curve for the planform shape and a constant velocity along the meander length, Equation 5 can be integrated for θ:

$$\theta = \frac{Ma_2}{2\pi V}\cos\frac{2\pi s}{M} + c_2 \tag{9}$$

in which the maximum angle of the streamline with the downstream valley θ_m corresponds to:

$$\theta_m = \frac{Ma_2}{2\pi V} \tag{10}$$

The meander wavelength is computed from the following relation:

$$\lambda = \int_0^M \cos\theta \, ds \tag{11}$$

Defining a nondimensional curvilinear distance $s' = s/M$ and rearranging with Eqs. 9 and 10 gives:

$$\frac{\lambda}{M} = \int_0^1 \cos\left[\theta_m \cos(2\pi s')\right] ds' \tag{12}$$

The sinuosity is defined as the ratio M/λ, and increases gradually with θ_m. Equation 12 has been integrated numerically and the results are shown in Figure 3. Figure 3 (or Equation 12) can be used to determine the maximum angle of the streamline θ_m. For a given friction slope and the assumption of a sine-generated curve, the shape, sinuosity, and maximum angle of the streamline are defined. Although the shape of the planform is known, the scale must still be determined.

The radius of curvature varies along the bend as a cosecant function, and the minimum radius of curvature r_m is reached when the cosecant function equals unity. The ratio λ/r_{cm} of the wavelength to the minimum radius of curvature is:

$$\frac{\lambda}{r_m} = \frac{2\pi\theta_m\lambda}{M} \tag{13}$$

The meander amplitude, A_m, as defined in Fig. 1 is evaluated analytically by the following integral:

$$A_m = 2\int_0^{M/4} \sin\theta \, ds \tag{14}$$

The ratio of wavelength λ to the meander amplitude A_m is:

$$\frac{\lambda}{A_m} = \frac{\int_0^1 \cos\left[\theta_m \cos(2\pi s')\right] ds'}{2\int_0^{\frac{1}{4}} \sin\left[\theta_m \cos(2\pi s')\right] ds'} \tag{15}$$

Equations 13 and 15 have been integrated numerically and are also shown in Figure 3. The ratio λ/A_m is a function of θ_m that increases rapidly when θ_m exceeds 90^0, and reaches a value of 3.25 at the meander cutoff ($\theta_m \cong 125^o$). The relations expressed in Equations 13 and 15 further define the planform shape, but an additional relation is needed for the size. One relation can be formulated on the basis of a limiting radius of curvature to ensure cross-sectional stability. This procedure is tedious and requires an analysis of the transversal shear stress distribution and sediment transport along the meander path. In the case of low-flow channel design, a simpler relation is available in that the meander amplitude is constrained by the width of the parent channel. The latter approach has been adopted.

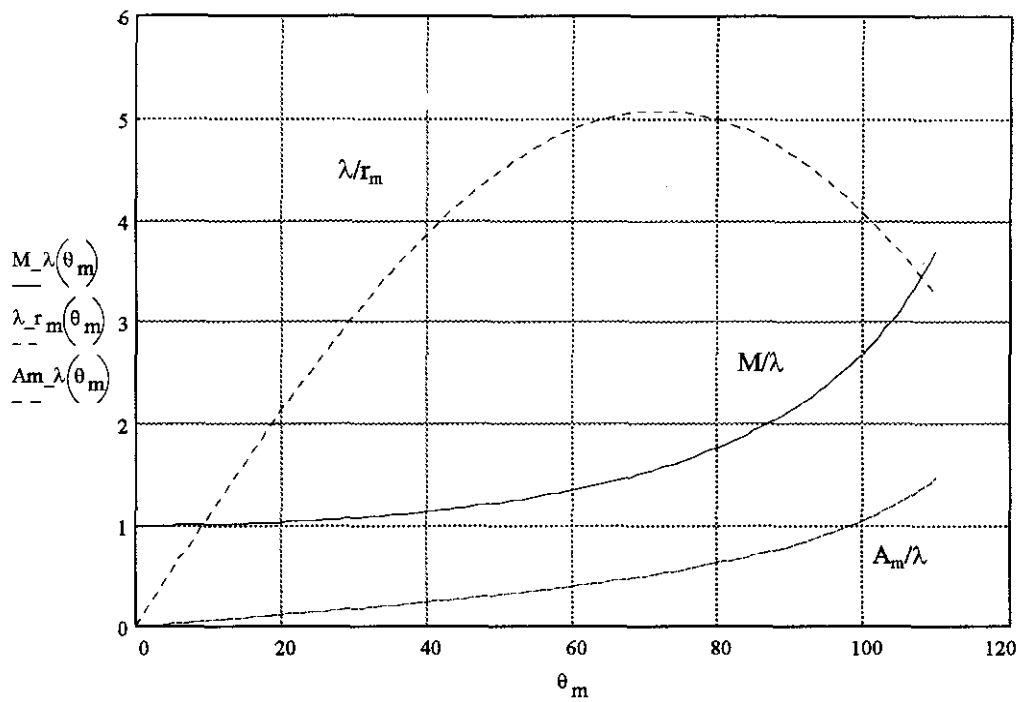

Figure 3 - Planform Relations

Low-flow channels are typically constructed by placing longitudinal bank protection, flow deflectors and sills within a parent channel. The width of the parent channel at the low-flow channel elevation defines a maximum meander amplitude with no erosion of the parent banks. Of course, this is only an upper limit and any $0 < A_m \leq W_c$ will meet the shape requirements. From a practical standpoint, however, constructability and habitat benefits are optimized with the larger amplitudes. The amplitude should be set at some fraction (perhaps 90 percent) of the parent channel width, minus the encroachment of longitudinal revetment structures. With this value of A_m, either Figure 3 or Equations 12 and 15 can be used to determine the meander arc length and wavelength. The channel planform can then be laid out and structures selected and located in the normal fashion.

Cross Section: Although habitat criteria were used to establish a target velocity, a rating equation is used to determine the mean velocity for the defined channel. If the Manning-Strickler Equation is used:

$$V = \frac{21.1}{d_{50}^{1/6}} D^{2/3} S_f^{1/2} \tag{16}$$

This mean velocity must be evaluated on the basis of habitat considerations. If it is unacceptable, then the depth must be adjusted and the entire procedure iterated until an acceptable value of velocity is obtained. When the velocity is within acceptable limits, the principle of continuity is applied to determine the cross-sectional area:

$$A = \frac{Q}{V} \tag{17}$$

An infinite number of cross-sectional shapes can be developed within the area and depth constraints given. However, naturally stable sand bed channels tend to have certain shapes with respect to their position, commonly trapezoidal in the crossings and a skewed parabola in the bendways. Investigators have described these shapes in terms of transversal sediment transport, force analyses and critical transversal shear stress distribution. From a practical standpoint, these complex cross sections would be difficult to construct and the width varies little from that for a trapezoidal section. As a first approximation, a trapezoidal section with one on three side slopes should be used to establish the channel bottom width:

$$W = \frac{A}{D} - 3D \tag{18}$$

DISCUSSION

A procedure for the first-order approximation of a stable low-flow channel form has been developed based upon physical reasoning, empirical evidence, and constraints common to low-flow channel projects. To design the channel in as near its stable form as possible, more sophisticated analyses would be warranted. However, this procedure yields a reasonable estimate of the planform and grade of the channel and that natural transport processes will quickly reform the cross section without impacts to the channel stability.

Many of the assumptions used in the formulation of the procedure are not always valid. For example, the Manning-Strickler Equation is assumed to apply, which further implies that the sediment is in the sand size range and that the flow is approximately uniform. In this simplified analysis, only the longitudinal shear stress is considered. A detailed analysis should also consider the transversal stress distribution, which will generally result in a decrease in the allowable friction slope. Finally, section stability under a range of discharges has not been established; such a determination would require that the friction slope decrease with increasing depth and that the transport capacity through the section match the inflowing load.

Despite these limitations, the method as presented yields a reasonable first-order approximation of the stable channel form. This "stable" channel configuration can then be further evaluated using more sophisticated techniques. The proposed procedure yields a close approximation of what occurs naturally and thus allows the stream to make minor adjustments that conform to its newly introduced geometry.

Acknowledgments: Information presented herein, unless otherwise noted, was obtained from research conducted under the Environmental Impacts Research Program. Permission was granted by the Chief of Engineers to publish this information. The author gratefully acknowledges technical reviews by Mr. E.A. Dardeau, Jr., and Ms. Trudy Olin. Discussions with Dr. Pierre Julien were used to formulate the more detailed procedure being developed. He is to be credited with the generation of several concepts and equations presented herein.

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NOTATION

A	cross-sectional area of the channel
Am	meander amplitude
$a, a_1, \ldots$	empirical coefficients
$b, b_1, \ldots$	empirical coefficients
$c, c_1, \ldots$	empirical coefficients
D	average flow depth
D_c	average depth in parent channel
d_{50}	representative bed sediment size
e, k, m	empirical coefficients
Q	total discharge
r_m	minimum radius of curvature
M	wavelength path distance
s	curvilinear longitudinal distance
s'	dimensionless curvilinear distance
S_f	friction slope
S_o	bed slope
V	cross-sect. ave. downstream velocity
W	channel width
W_c	parent channel width
w	curvilinear transversal coordinate
x	cartesian longitudinal coordinate
y	vertical coordinate
z	cartesian transversal coordinate
γ	unit weight of water
γ_s	unit weight of sediment
λ	meander wavelength
θ	angle of streamline with x
θ_m	maximum angle of streamline with x
τ_*	dimensionless Shields number
τ_{*c}	critical Shields number

SEDIMENT IMPACT ASSESSMENTS USING THE HYDRAULIC DESIGN PACKAGE - SAM

Ronald R. Copeland, Research Hydraulic Engineer, U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, Mississippi; and
William A. Thomas, Owner, Mobile Boundary Hydraulics, Clinton, Mississippi

Abstract: Several years ago a need was identified for a technique to make quick evaluations of channel performance for alternatives in the initial planning stages of flood control studies. Channel performance includes both conveyance potential and stability, especially as it relates to long-term maintenance. As a result, the U.S. Army Corps of Engineer Waterways Experiment Station, funded by the Flood Control Channels Research Program, developed a hydraulic design package for channels based on uniform flow assumptions using "typical" cross-sections. This package, SAM, allows for evaluating channel stability for a variety of channel geometries and sediment loadings using a sediment budget approach. The SAM hydraulic design package is especially useful for conducting sediment impact assessments during preliminary planning studies. This paper briefly describes the scope of a sediment impact assessment, the capabilities of the SAM hydraulic design package, and some general examples of applications.

INTRODUCTION

Sediment Impact Assessment: In the early stages of the planning process for water resource projects it is important to assess the proposed project's effect on the stability of the river system. In the US Army Corps of Engineers, these assessments are called sediment impact assessments and are usually conducted during the reconnaissance level planning study, where several alternatives are considered. The primary purpose at this stage of planning is to determine if there is an engineeringly feasible, economically justifiable, and environmentally acceptable project. The purpose of a sediment impact assessment is to identify the magnitude of possible sediment problems associated with proposed engineering projects. Sediment impacts are generally determined using a sediment budget approach where sediment transport and sediment yields for existing and project conditions are compared. Recommendations are made for appropriate sediment studies at the next level of planning study. In conducting a sediment impact assessment, one is more interested in magnitudes and comparisons than quantification.

SAM - Hydraulic Design Package: The SAM program (Thomas et al. 1995) consists of six modules. The three major modules that provide the calculating power of the package are the hydraulics module - *SAM.hyd*, the sediment transport module - *SAM.sed*, and the sediment yield module - *SAM.yld*. Three utility modules aid in the use of the major computational modules.

Hydraulics module: Hydraulic calculations are conducted in *SAM.hyd*. Normal depth and composite hydraulic parameters are calculated for complex channels with variable roughnesses, using one of four compositing options. Roughness can be assigned using Manning's roughness coefficients, relative roughness height, grain size, or grass type. The Manning, Strickler,

Keulegan (1938), Limerinos (1970), and Brownlie (1983) roughness equations are available, and different equations may be used for different sections of the cross-section. Effective hydraulic parameters for sediment transport calculations are written directly into an input file for the sediment module. The hydraulics module also has options for sizing riprap and for determining stable channel dimensions based on sediment continuity principles.

Sediment transport module: *SAM.sed* provides 16 sediment transport equations for calculating the bed-material sediment discharge. The calculations are made by particle size class.

Sediment yield module: *SAM.yld* calculates sediment yield using the flow-duration sediment-discharge rating curve method (USAEHQ 1989). This method consists of integrating the sediment-discharge rating curve and a flow-duration curve or flood hydrograph to calculate sediment yield on an average annual basis or for a flood event.

Utility modules: Three utility codes provide assistance to the user. *PSAM* is a series of input screens for coding input data files for the three major modules of the SAM package. It provides user-friendly input menus and writes the created data files into the format the computer code expects. *SAM.m95* reads the binary output file created by HEC-2, and uses that file to compute average hydraulic parameters of width, depth, velocity, slope and n-value. It writes those parameters into an input file for *SAM.sed*. *SAM.aid* provides guidance in the selection of sediment transport function(s). Using the *SAM.aid* approach, the investigator would prescribe a bed gradation and calculate hydraulic parameters using *SAM.hyd*. This establishes the screening parameters of velocity, depth, slope, width, and d_{50}, which are compared to a library of field data, compiled from the literature, and encoded into *SAM.aid*. Much of the library of test data was obtained from Brownlie (1981). Field data sets with screeening parameters similar to the project reach are identified. The three sediment transport functions that best reproduced the measured sediment transport in the matched field data sets are identified.

COMPONENTS OF A SEDIMENT IMPACT ASSESSMENT

Topics to Report: The following subject areas should be included in a sediment impact assessment:

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

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

- A site reconnaissance should be conducted to identify the stability of the existing channel as well as existing problems upstream and downstream from the proposed project area. The type of bed-material sediment load should be determined, and bed-material samples

collected. Aggradation and/or degradation in the project reach should be noted. The land use in the basin should be noted, especially if there have been any significant changes.

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

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

Sediment budget analysis: The sediment budget analysis is the analytical backbone of the sediment impact assessment. The sediment budget is a comparison between sediment yield leaving the project reach with the project in operation and sediment yield leaving the project reach without the project. A comparison of these two sediment yields, using the trapping efficiency equation, provides relative stability results for various alternatives. A positive trap efficiency indicates deposition; a negative value indicates erosion. The best balanced design is the one for which the trap efficiency is near zero. The level of confidence that can be assigned to the sediment budget approach is a function of the reliability of the available data.

EXAMPLE APPLICATION OF SAM

A small creek is to be "improved" for flood control by enlarging the cross section. The question is, "Will the proposed design channel be stable?" The answer is determined with a sediment impact assessment and calculations performed using SAM. A sediment budget analysis is conducted to determine if the proposed design is stable based on the trap efficiency of the proposed channel. If the channel design is determined to be unstable, then the stable-channel analytical method is applied to determine stable channel dimensions.

Sediment budget : First, hydraulic parameters are calculated for the existing channel upstream from the project reach using the *SAM.hyd* module. Input requirements are channel geometry, slope, hydraulic roughness, and a range of discharges. Results are shown in the first 9 columns of Table 1. Next, the bed material sediment load is calculated using the *SAM.sed* module. Required input includes gradation of the stream bed and a prescribed sediment transport function. Effective hydraulic parameters for sediment transport are read directly from a file produced by *SAM.hyd*. Calculated sand transport is tabulated in the last column of Table 1.

The proposed width, depth and slope of the Project Channel was furnished by its designer. The channel was sized to convey the one-percent chance exceedance flood. The design cross section is a trapezoid with a 100-ft bottom width and 1V:3H side slopes. The bottom slope is 0.0004 ft/ft. This proposed cross section was analyzed by SAM using the same procedure described above for the existing upstream channel. Results are shown in Table 2.

TABLE 1. Calculated Inflowing Sand Discharge to Project

N	Q cfs	Normal* Depth ft	Top Width ft	Area sqft	R ft	n-value	Velocity fps	Shear Stress lbs/sqft	Bed-material Transport Tons/Day
1	5	0.28	16.3	4	0.27	0.0217	1.16	0.028	5
2	10	0.43	17.0	7	0.42	0.0233	1.45	0.043	20
3	20	0.67	18.1	11	0.65	0.0249	1.80	0.066	50
4	25	0.77	18.6	13	0.75	0.0255	1.94	0.075	70
5	100	1.33	21.2	24	1.29	0.0171	4.14	0.131	700
6	500	3.41	30.9	78	3.16	0.0202	6.38	0.319	5,800
7	3680	9.82	123.5	381	8.55	0.0259	9.66	0.864	59,000

* The channel slope is 0.00166.

TABLE 2. Sand Transport through the Project Channel

N	Q cfs	Normal Depth ft	Top Width ft	Area sqft	R ft	n-value	Velocity fps	Shear Stress lbs/sqft	Bed-material Transport Tons/Day
1	5	0.35	70.8	13	0.23	0.0281	0.40	0.006	0
2	10	0.47	93.9	22	0.29	0.0286	0.45	0.007	0
3	20	0.60	100.6	35	0.39	0.0280	0.57	0.010	0
4	25	0.66	100.9	41	0.44	0.0279	0.62	0.011	1
5	100	1.20	104.2	97	0.96	0.0279	1.04	0.024	40
6	500	2.74	113.4	264	2.44	0.0284	1.90	0.061	600
7	3680	6.96	138.8	797	6.43	0.0223	4.62	0.160	14,000

Trap efficiency is expressed by the following equation.

$$E = \frac{100\ (Y_{s(in)} - Y_{s(out)})}{Y_{s(in)}}$$

Y_S is annual sediment yield. The subscript (in) is inflow and (out) is outflow. The two yields, $Y_{S(in)}$ and $Y_{S(out)}$, are calculated with the *SAM.yld* module. The sediment discharge rating curves, in this case, are the calculated bed material transport rates shown in Tables 1 and 2. The flow-duration data is shown in the following table. It is the same for both existing and project conditions.

TABLE 3. Annual Flow-Duration Table

Time*	1	5	10	15	20	30
Q, cfs	100	12	6	4	3	1

* The percent of time flow is greater than or equal to Q

In this example, the calculated average annual sand yield entering the project area is 2600 cubic yards per year. The calculated average annual sand yield leaving the project area is 85 cubic yards per year. That converts to a trap efficiency of 97% which indicates a potential channel stability problem, with this design, due to deposition. Long term maintenance would be required to remove those deposits. At this point in the analysis, the stable-channel analytical method may be employed to determine channel dimensions that would have a trap efficiency closer to zero.

Stable-channel analytical method: This method calculates channel dimensions of width, depth and slope that will transport an incoming sediment load without deposition. This computation assumes all bed material transport occurs over the bed of the cross section and that none occurs above the side slopes. Therefore, the portion of water conveyed above the side slopes expends energy but does not transport sediment, making flow distribution an extremely important calculation. The input parameters for the analytical method are bank angle, bank roughness, bed gradation, sediment inflow, and discharge. The maximum attainable slope (valley slope) should also be input for reference purposes.

Basic Equations: The method uses the sediment transport and resistance equations developed by Brownlie (1981), which automatically determines whether the flow-bed regime is upper or lower. A key point in this analytical method is the separation of channel roughness into bed and bank components. The composite roughness value is calculated using the equal velocity assumption proposed by Einstein (1950).

Inflowing Water Discharge: The design discharge is critical in determining appropriate dimensions for the channel. Investigators have proposed different methods for estimating that design discharge. The 50-percent chance exceedance flood is sometimes used for perennial streams. For ephemeral streams, the 10-percent chance exceedance flood is sometimes used. The 'bankfull' discharge is sometimes suggested; others prefer using the "effective" discharge. However, there is no generally accepted method for determining the channel forming discharge. Currently, it is recommended that a range of discharges be used in the analysis to test the sensitivity of the solution.

Inflowing Sediment Discharge: The recommended procedure is to calculate sediment inflow using the Brownlie equations in *SAM.hyd.* This ensures analytical consistency in the calculations of sediment transport upstream and in the project reach. The calculation of sediment concentration is based on hydraulic conditions in a user-specified sediment-supply reach. There is an option for directly specifying the concentration of the inflowing bed-material load.

Stable Channel Dimensions: The family of widths, depths, and slopes which satisfies the sediment transport and resistance equations for the example is shown in Table 4. Each line in the table satisfies the governing equations. It is important to be consistent in the selection of channel dimensions. That is, once a width is selected, the depth and slope are fixed. The designer may select specific project constraints, such as right-of-way or bank height or minimum bed slope, and then arrive at a consistent set of channel dimensions. It is also important to consider river morphology when interpreting these calculated values. That is, natural rivers tend toward a regime width that typically is determined empirically using existing channel widths or regression equations.

TABLE 4. Widths, Depths and Slopes Calculated by the Stable-Channel Analytical Method

```
INFLOWING WATER DISCHARGE, CFS          =   3680.000

                                          LEFT BANK  RIGHT BANK
SIDE SLOPE                              =      3.000       3.000
Ks, FT                                  =      1.149       1.149
n-VALUE                                 =     .03500      .03500

CALCULATE STABLE CHANNEL DIMENSIONS.
MEDIAN BED SIZE ON BED, MM              =     .34768
GRADATION COEFFICIENT                   =      1.783
VALLEY SLOPE, FT/FT                     =  .00040000

TABLE  4-1.  STABLE CHANNELS FOR Q=    3680.0 C,ppm=  5938.     D50=     .348

K: BOTTOM : DEPTH :  ENERGY :CMPOS"T:   HYD :   VEL : FROUDE:  SHEAR:   BED *
 :  WIDTH :       :   SLOPE :n-Value: RADIUS:       : NUMBER: STRESS:  REGIME
 :   FT   :   FT  :   FT/FT :       :   FT  :   FPS :       :   #/SF:

 1      12.   18.0   .003692   .0323    5.58    8.78     .49    2.30      UP
 2      24.    9.0   .002542   .0300    5.69    7.97     .47    1.43      UP
 3      36.    8.0   .002091   .0278    5.54    7.65     .48    1.04      UP
 4      48.    7.1   .001856   .0259    5.28    7.49     .50    0.82      UP
 5      61.    6.3   .001720   .0243    4.97    7.39     .52    0.67      UP
 6      73.    5.6   .001640   .0230    4.65    7.31     .54    0.58      UP
 7      85.    5.1   .001592   .0219    4.35    7.23     .57    0.51      UP
 8      97.    4.6   .001566   .0210    4.07    7.16     .59    0.45      UP
 9     109.    4.3   .001555   .0202    3.82    7.08     .60    0.41      UP
10     121.    4.0   .001553   .0196    3.60    7.01     .62    0.38      UP
11     133.    3.7   .001558   .0191    3.39    6.93     .64    0.36      UP
12     145.    3.4   .001569   .0187    3.21    6.86     .65    0.34      UP
13     157.    3.2   .001583   .0183    3.05    6.78     .66    0.32      UP
14     169.    3.1   .001599   .0180    2.90    6.71     .68    0.31      UP
15     182.    2.9   .001618   .0178    2.77    6.64     .69    0.29      UP
16     194.    2.8   .001638   .0175    2.65    6.57     .70    0.28      UP
17     206.    2.6   .001659   .0173    2.54    6.50     .70    0.27      UP
18     218.    2.5   .001681   .0172    2.44    6.44     .71    0.27      UP
19     230.    2.4   .001703   .0170    2.35    6.38     .72    0.26      UP
20     242.    2.3   .001726   .0169    2.27    6.32     .73    0.25      UP

          RESULTS AT MINIMUM STREAM POWER
21     117.    4.0   .001553   .0198    3.66    7.03     .62     .39      UP
>>>> MINIMUM SLOPE IS GREATER THAN VALLEY SLOPE - THIS IS A SEDIMENT TRAP <<<<
 * REGIMES:  LO=LOWER, TL=TRANSITIONAL-LOWER, TU=TRANSITIONAL-UPPER, UP=UPPER.
```

Channel dimensions corresponding to minimum stream power conditions are presented at the bottom of the table. In this example, even the minimum slope is larger than the valley slope,

indicating that none of the computed solutions are feasible. Deposition must be accounted for with a planned maintenance program or a sediment retention structure. This situation is common in valley streams where large sediment loads are delivered from upland watersheds. In this example, the sediment impact assessment has identified a significant sediment problem to be addressed with a more detailed study.

OTHER GENERAL APPLICATIONS

Carlsbad, New Mexico: A typical sediment impact assessment was conducted for a flood protection project for Dark Canyon Draw, a tributary of the Pecos River, in Carlsbad, New Mexico. One flood control alternative for Dark Canyon Draw was a diversion channel that would divert flow from Dark Canyon Draw upstream from Carlsbad and carry it to the Pecos River about five miles downstream from Carlsbad.

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

The criteria chosen for the diversion channel design were: 1) a composite channel geometry with a low flow channel designed to carry the effective discharge; and 2) the rest of the channel designed to carry the one-percent chance exceedance flood with a velocity of less than 6 ft/sec on the overbank. The effective discharge is the discharge that transports the largest percentage of the bed-material sediment load. The stable-channel analytical method in *SAM.hyd* was used to size the low-flow channel. The inflowing sediment concentration was determined for the effective discharge from the sediment transport rating curve developed for the typical reach of Dark Canyon Draw. Using the natural slope between the proposed Dark Canyon Draw diversion and the Pecos River, a unique solution for width and depth was read from the analytical channel method table of values. The width of the overbank portion of the channel was then calculated by trial and error using the maximum velocity criteria for the overbank. Having determined the dimensions of the composite channel, sediment yield was calculated using *SAM.yld*. Trap efficiency was then calculated for the flood hydrographs and for average annual conditions. At the next level of project formulation, it will be necessary to evaluate the temporal development of the diversion channel using a numerical sedimentation model.

Alabama River: The US Army Corps of Engineers maintains a nine-ft-deep navigation channel on the Alabama River between its confluence with the Mobile River and Claiborne Lock and Dam, 72 miles upstream. Due to increasing water demands upstream, reductions in minimum discharge releases are being considered, increasing the probability of losing required navigation

depths. The purpose of the sediment impact assessment was to identify the magnitude of changes in sediment deposition, and thereby dredging requirements, associated with alternative proposed navigation channel designs required to maintain the minimum draft. By comparing the calculated sediment yields for three proposed channel geometries, it was possible to estimate the increased deposition with the lower minimum flow releases. The average was found to be on the order of a 1 to 3 percent increase in dredging.

CONCLUSIONS

Early in project formulation engineers are pressured to answer questions about the impact of sediment on project performance and about the impact of the project on the stream system morphology. Data are scarce, and time and budget constraints prevent the use of design level techniques. The sediment impact assessment was developed for such situations, and the SAM package provides the computational power to quantify answers to those questions. Its extremely powerful hydraulic computations calculate channel dimensions directly. Channel stability is assessed, and riprap is designed automatically as needed. The 16 sediment transport functions are screened for the most appropriate one to use. Project trapping efficiency, using annual sediment yield calculations, allows preliminary estimates of long term maintenance. Input screens and graphical output make SAM easy to use.

ACKNOWLEDGMENTS

The work described and the resulting data presented herein, unless otherwise noted, were obtained from research conducted by the US Army Engineer Waterways Experiment Station, Vicksburg, MS. Permission was granted by the Chief of Engineers to publish this information.

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GRADIENT AND PLAN FORM STABILIZATION OF AN INCISING STREAM

R.J. Wittler[1], S.D. Keeney[2], B.W. Mefford[3], S.R. Abt[4], C.C. Watson[5]

INTRODUCTION

This paper describes the efforts of a partnership of Federal and local government agencies and a local citizen task force to solve the water quality problems associated with the incision of Muddy Creek near Great Falls, Montana. The United States Bureau of Reclamation and Greenfields Irrigation District are collaborating to reduce return flow to Muddy Creek. Reclamation, Cascade County Conservation District, and the Muddy Creek Task Force, are collaborating to stabilize the gradient and plan form of the stream.

Project Sponsors

The Sun River Watershed project sponsors include the Cascade County Conservation District (CCCD), Lewis & Clark County Conservation District (LCCCD), Teton County Conservation District (TCCD), Muddy Creek Task Force, Greenfields Irrigation District (GID), Fort Shaw Irrigation District (FSID), Broken O Ranch, Rocky Reef Ditch Company, and the Sun River Ditch Company. Federal, state, and local agencies include the US Bureau of Reclamation, US Fish & Wildlife Service (USF&W), US Bureau of Land Management (BLM), US Forest Service (USFS), Montana Department of Environmental Quality (DEQ), Montana Department of Natural Resources and Conservation (DNRC), Montana Department of Fish, Wildlife and Parks (DFWP), Montana State University Extension Service, Cascade County, City of Great Falls, Montana Power Company (MPC), Burlington Northern Railroad (BN), Medicine River Canoe Club (MRCC), Missouri River Flyfishers, Missouri Breaks Audubon Chapter, Great Falls Chamber of Commerce, Nilan Water Users, Tri-Area Water Users, Communities of Sun Prairie, Vaughn, Sun River, Fort Shaw, Simms, Augusta, Fairfield, and Power, and independent landowners.

BACKGROUND

Sun River Watershed is east of the continental divide and south of Glacier National Park. It covers an area of 2,200 square miles (1,408,000 acres), with roughly 356 square miles (228,096 acres) in northwest Cascade County, 1089 square miles (696,960 acres) in southern Teton County. The Sun River starts in the Bob Marshell Wilderness area and meanders out of the mountains through rolling grass-covered foothills and farmland to its confluence with the Missouri River at the City of Great Falls.

Muddy Creek, a tributary of the Sun River in the Upper Missouri River Basin, joins the Sun River 15 miles upstream from the Sun-Missouri confluence at Great Falls, Montana. The Creek drains approximately 314 square miles of agricultural land. Muddy Creek is a sinuous stream augmented by irrigation return flows resulting in an eight-fold increase from the mean historical flow. Irrigation return flow into Muddy Creek from developed agriculture is causing the stream to rapidly incise, accelerating the meandering and cutoff process. The stream has incised roughly thirty-three feet (10 meters) in glacially deposited silty soil. In 1993 Reclamation approved a low cost erosion control scheme as a demonstration research project. Success of the demonstration project may lead to expansion of the project to the headwaters.

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[1] Hydraulic Engineer, WRRL, US Bureau of Reclamation, D-8560, POB 25007, Denver, CO 80225
[2] Civil Engineer, US Bureau of Reclamation, MT-435, Billings, MT 59107-0137
[3] Technical Specialist & Hydraulic Engineer, WRRL, US Bureau of Reclamation, Denver, CO 80225
[4] Professor & Hydraulic Program Coordinator, Colorado State University, Fort Collins, CO 80523
[5] Asst. Professor, Dept. Of Civil Engineering, Colorado State University, Fort Collins, CO 80523

The erosion control scheme includes chevron weir rock ramps for grade control, barbs for plan form control, and willow post planting for bank stabilization. The demonstration project is on a four mile reach of stream experiencing the most active erosion. Reclamation is responsible for the design and placement of the rock ramps and barbs, evaluation of performance, and environmental and historic preservation compliance. Cascade County Conservation District is revegetating the area with native grasses and willows and surveying for the monitoring plan. The Task Force secured the necessary permits and provided overall project guidance. Greenfields Irrigation District, a task force member, provided all construction equipment and personnel as an in-kind contribution.

The increased creek flow from irrigation return flows has caused extensive erosion of the fine grained alluvial soils and has transported up to 200,000 tons of sediment annually into the Sun River. Much of this material comes from large scale bank slumping and vertical degrading of the stream bed in the Creek's lower reach. The sediment load transported by the Creek has severely impacted the water quality in both the Sun and Missouri Rivers. Figure 1 shows a reach of slumped bank along Muddy Creek.

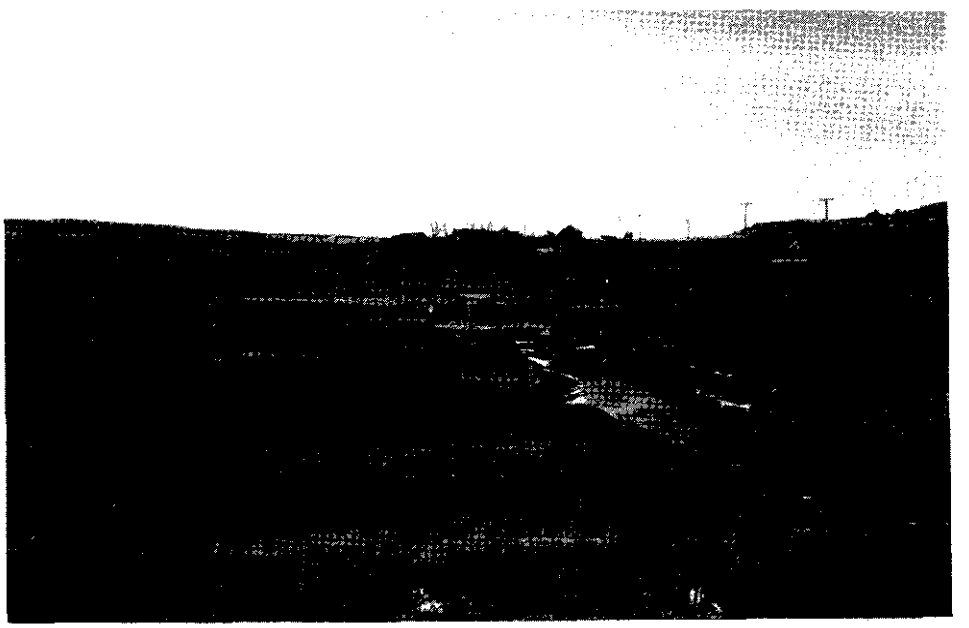

Figure 1. Destabilized bank and upland area due to incision, saturation, and slip failure.

Topography & Geology

The Sun River basin is 110 miles long and 30 miles wide, draining 2,200 square miles of the east slope of the Rocky Mountains. Elevation ranges from 9,000 feet to 3,350 feet. The watershed lies in two major land resource areas, 52 (Brown Glaciated Plain) and 46 (Northern Rocky Mountain Foothills). The majority of the irrigated areas are on distinct terrace formations consisting of shallow top soils overlaying a 15-20 foot thick gravel glacial deposit underlain by the impervious Colorado Shale formation. Soils range from gravelly and sandy loams to light clays. The geologic materials in the watershed include: 1) loamy to clayey deposits laid down by glaciers, 2) materials weathering from sedimentary shales and sandstones, and 3) alluvium on broad high benches, alluvial valleys, local footslopes, fans and small closed basins. Common soils groups are alluvial, brown, chestnut, chernozems, gray wooded, brown podsolic, regosols, lithosols and humic glei. The agriculture soils are largely developed from glacial till, sandstones, shales and alluvium. Clay loam to loam textures predominate. Principle problems associated with irrigated soils include 1) shallow depth over gravels and associated low moisture storage capacity, 2) poor drainage, and 3) a combination of poor drainage and high concentrations of toxic salts.

DEMONSTRATION STREAM RESTORATION PROJECT

The demonstration project is an evaluation of a low cost grade control and bank protection design on a four mile reach of the stream where down cutting and bank erosion is most active. Table 1 lists the structures installed through Fall of 1994. The Task Force installed additional barbs and one grade control structure, 1-G, in late 1994. The project currently includes 9 chevron weir rock ramps for grade control and 33 barbs for plan form control. Figure 2 shows the topography surrounding the demonstration reach.

Table 1. Partial list of demonstration project structures and actions.

Task	Type	Yards/ft.	Width	Volume (Yards)	Start Date	Complete Date	Supervisor
USACE	Sill	-			12/10/94	1/5/94	USACE
1A	1' Drop	2.5	80.0	200.0	2/21/94	2/25/94	RJW/TV/SK
1B	1' Drop	2.5	81.0	202.5	3/9/94	3/10/94	RJW
1C	1' Drop	2.5	51.5	128.8	3/2/94	3/8/94	SK/RJW
B-60	L-Bank Barb	3.0	30.0	90.0	2/28/94	3/1/94	SK
B-60.5	L-Bank Barb	2.5	25.0	62.5	2/28/94	3/1/94	SK
B-61	L-Bank Barb	2.0	20.0	40.0	2/28/94	3/1/94	SK
B-62	L-Bank Barb	1.5	15.0	22.5	2/28/94	3/1/94	SK
Cut #1	Cut & 2 Barbs	3.0	15.0	45.0	3/7/94	3/24/94	RJW/SK
1D	1' Drop	2.5	90.5	226.3	3/11/94	3/16/94	RJW/TV
1-E	1' Drop	2.5	93.0	232.5	4/11/13	4/13/94	SK
B-95	L-Bank Barb	3.0	30.0	90.0	3/15/94	3/17/94	TV
B-95.25	L-Bank Barb	3.0	30.0	90.0	3/15/94	3/17/94	TV
B-95.5	L-Bank Barb	2.5	25.0	62.5	3/15/94	3/17/94	TV
B-96	L-Bank Barb	2.0	20.0	40.0	3/15/94	3/17/94	TV
B-96.25	L-Bank Barb	2.0	20.0	40.0	3/15/94	3/17/94	TV
B-96.5	L-Bank Barb	1.5	15.0	22.5	3/15/94	3/17/94	TV
B-97	R-Bank Barb	2.5	15.0	37.5	4/13/94	4/14/94	SK
B-97.30	R-Bank Barb	2.0	15.0	30.0	4/13/94	4/14/94	SK
B-97.60	R-Bank Barb	1.5	15.0	22.5	4/13/94	4/14/94	SK
B-97.90	R-Bank Barb	1.5	15.0	22.5	4/13/94	4/14/94	SK
B-98.20	R-Bank Barb	1.5	15.0	22.5	4/13/94	4/14/94	SK
B-121	R-Bank Barb	3.0	28.0	84.0	4/14/94	4/18/94	SK
B-122	R-Bank Barb	2.5	26.0	65.0	4/14/94	4/18/94	SK
B-124.5	L-Bank Barb	2.0	30.0	60.0	4/22/94	4/22/94	RJW
B-125	L-Bank Barb	2.0	30.0	60.0	4/22/94	4/22/94	RJW
B-125.5	L-Bank Barb	2.0	30.0	60.0	4/22/94	4/22/94	RJW
2A	2' Drop	3.5	87.0	304.5	4/18/94	4/20/94	RJW/BH
2B	2' Drop	3.5	99.0	346.5	3/22/94	3/23/94	SK
2C	2' Drop	3.5	85.9	300.7	4/20/94	4/22/94	RJW/BH
2D	2' Drop	3.5	113.0	395.5	Fall 94	Postponed	
3A	Revetment			300.0	3/17/94	3/21/94	TV/BH/SK
Cleanup	Grading & pickup				4/11/94	4/22/94	BH
Seed Prep	33 acres				4/11/94	4/22/94	DB
Broadcast Seeding	8 acres				4/26/94	4/26/94	DB
Drill Seeding	25 acres				5/2/94	5/3/94	DB
Tree Planting	6000 willows				4/30/94	4/30/94	DB
				3,706			

Grade Control

The grade control is a hybrid of riprap, vortex weir, and drop structure design. Figure 3 shows the chevron shape of the structure. Figure 4 shows the ramp function in cross section. The combination of drop and chevron shape results in concentration of flow in the center of the channel at the toe of the ramp. The intent of the design is to keep high velocity flows away from the banks and to dissipate excess energy on the ramp rather than in a stilling basin or hydraulic jump, as Figure 5 shows. However, over concentrating the flow results in recirculating flow that causes bank degradation, as Figure 6 shows.

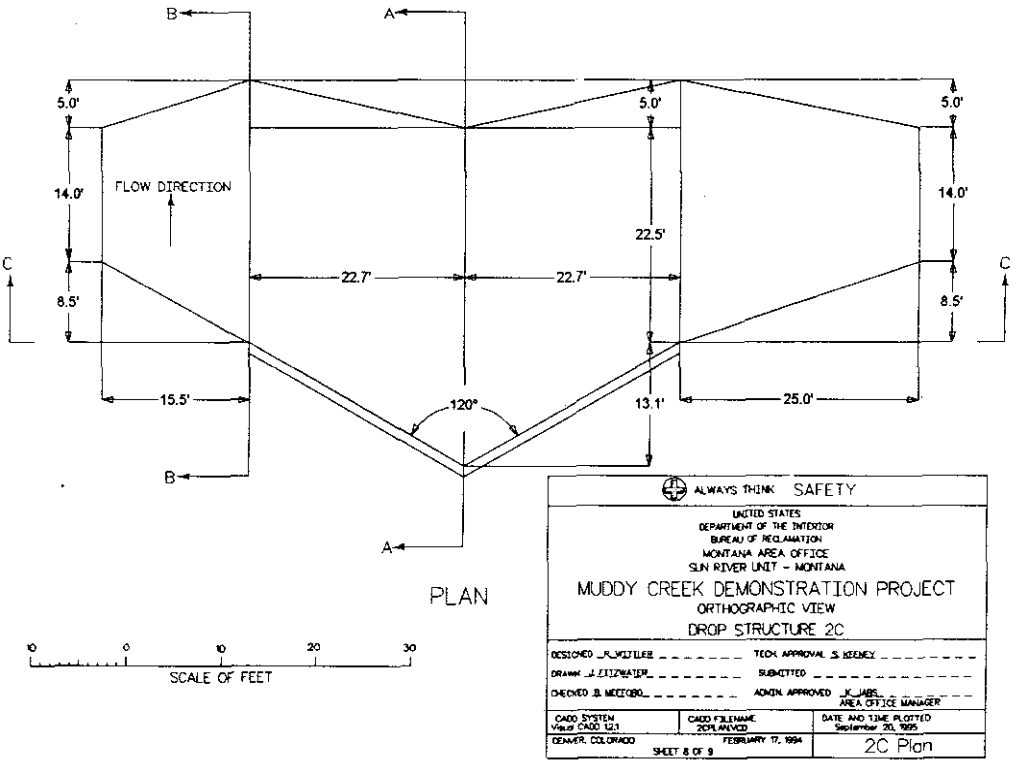

Figure 3. Plan view of chevron weir rock ramp grade control structure 2C.

Figure 3 shows the primary features of the chevron weir rock ramp. The two wings, or bank sections, the chevron weir section (center), and the sacrificial section at the downstream toe (top of picture). In the case of excessive erosion at the toe of the ramp, an additional five feet of sacrificial rock will launch into a developing scour hole, stabilizing the toe of the structure.

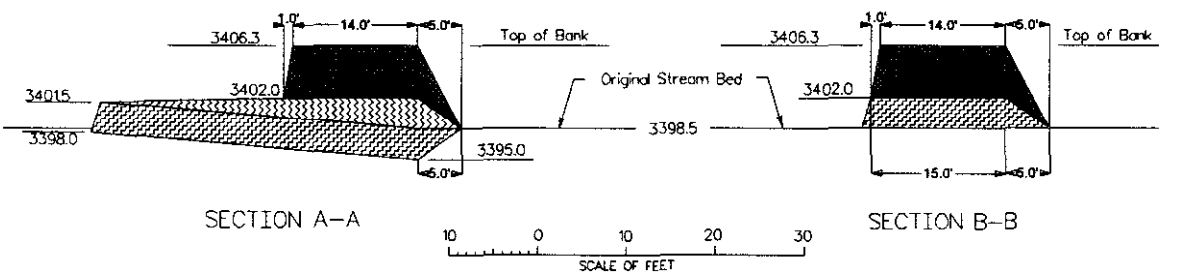

Figure 4. Sections A-A and B-B of structure 2C. Centerline slope is 10%, while groin is horizontal.

The riprap design provides the median rock size, D_{50}, as a function of the unit discharge, q, and the ramp slope, S, according to the following equation. The slope of all ramp structures is 10% while the unit discharge is a function of the hydrology.

$$D_{50} = 0.44q^{0.56}S^{0.43} \tag{1}$$

The wings elevate roughly ten feet above the invert. This was an arbitrary elevation, more a function of potential ice jam than flood stage analyses. Figure 7 shows an orthogonal view of the ramp. At Muddy Creek the median rock size was specified at thirty-three inches (0.84m). A geofabric (Mirafi 1100N) was placed between the excavated bed and the placed rock, with no gravel filter or other bedding. The key depth of the structure is 3.5 feet (1.07m). The weir height of a one-foot high structure is two feet above the

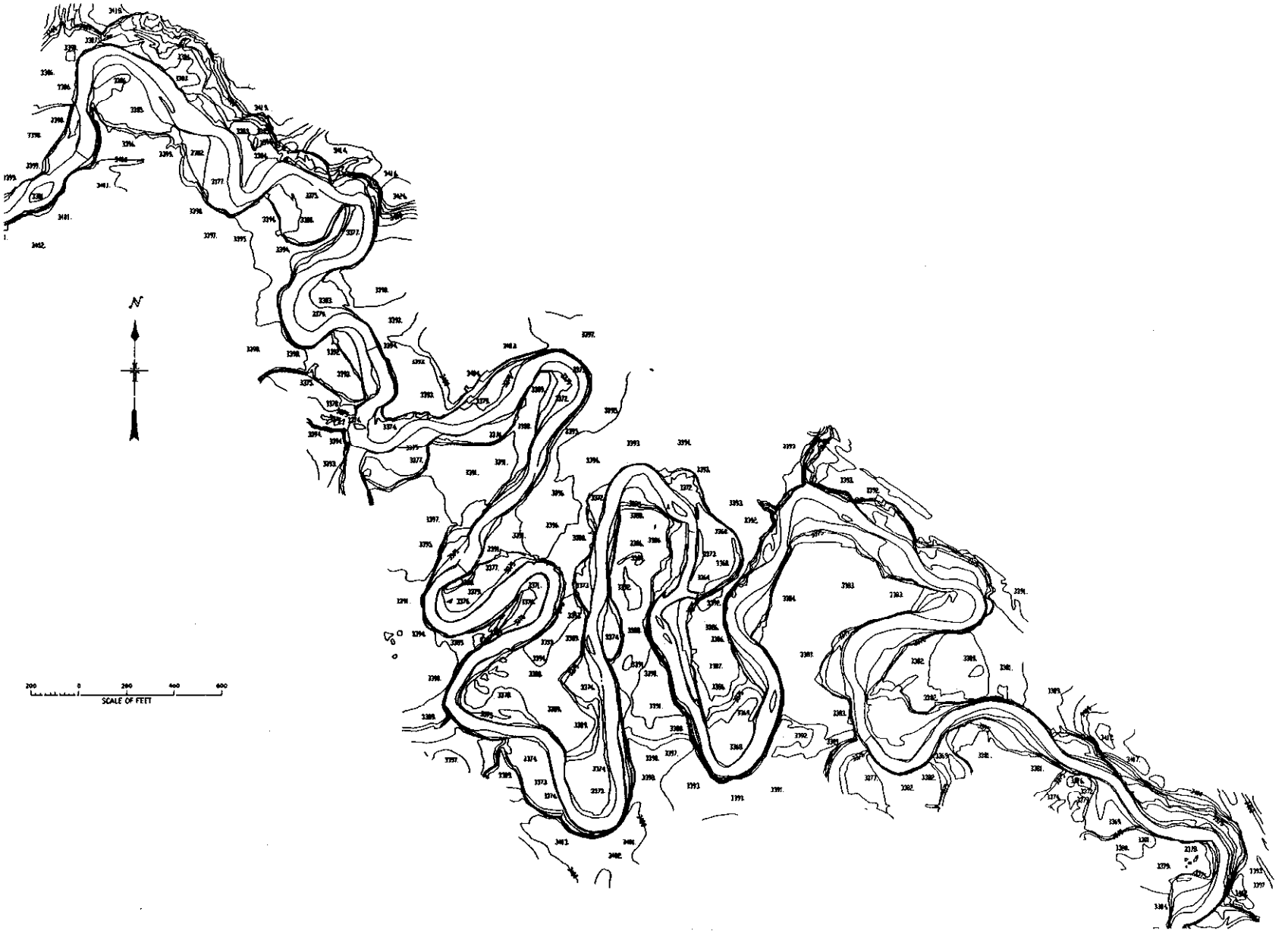

Figure 2. Topography surrounding roughly 4 mile reach of demonstration project.

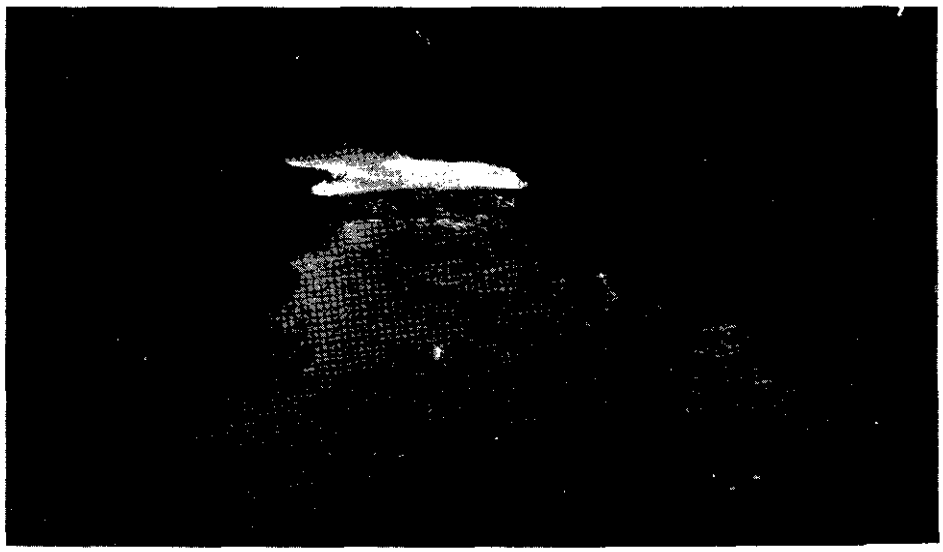

Figure 5. Upstream view of chevron weir rock ramp grade control structure.

existing bed, while the weir height of a two-foot high structure is three feet above the existing bed. The additional one foot in height compensates for the porosity of the rock that allows a significant portion of the flow to go through the rock rather than over the ramp. This design feature was judgment based and requires further research to properly specify.

Figure 6. Examples of operational problems with chevron weir rock ramp grade control structure.

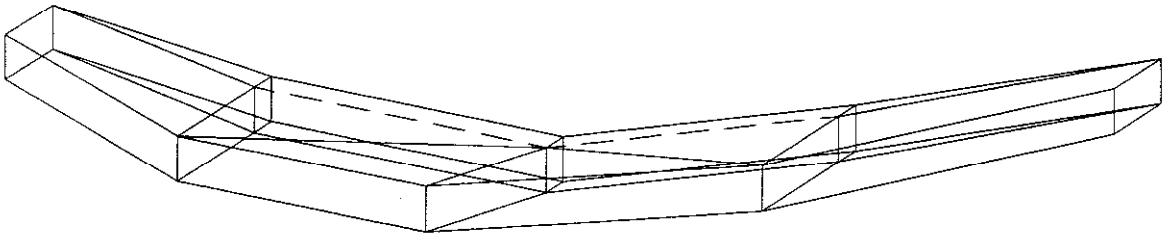

Figure 7. Orthogonal view of chevron weir rock ramp grade control structure.

Plan Form Control

Barbs are rock jetties, triangular in plan, placed on the outside bends of meanders, displacing the thalweg from the bank. The barbs slope from the bank to the stream and angle upstream to cause a slow back eddy next to the bank to create a protective flood plain bench. Seeding and willow post planting in combination with fencing and grazing management accelerates bank stabilization. Figure 8 shows the process of barbs displacing the thalweg and main current away from the toe of a nearly vertical bank. Barbs create a bench when bank material falls for vegetation to take hold and to stabilize the bank.

Figure 8. Deposition between barbs from siltation and bank degradation.

Monitoring

A primary task of the demonstration project is monitoring the response of the stream to the grade control and plan form control measures. Monitoring sub tasks fall into three categories: technical/surveying,

qualitative/aesthetic, and public acceptance. The technical/surveying category includes hydraulic analysis, surveying and land form tracking, geomorphological analysis, vegetative and wildlife census and density analysis, and water quality response. The qualitative/aesthetic category includes observational evidence of the response of the stream and habitat, and the relative amount of ground cover. The public acceptance category includes many aspects of aesthetics and the level of confidence of ultimate success. Specifically, the Task Force is planning to accomplish the following tasks:

1. Surveying/CADD
   a. Inventory existing surveys
   b. Redefine reaches and sub-reaches and rename structures
   c. Locate all structures within master AutoCAD file
   d. Update survey reduction to 1995 datum
   e. Integrate all survey data
      i. Reduce using NRCS proprietary program (BASIC)
      ii. Input ASCII triplets into Visual CADD or Generic CADD
      iii. Transfer VCADD or GCADD 2-D profiles (X-Sections or Profiles) to AutoCAD
      iv. Integrate all sections and topography on independent layers into master file
2. Implement GIS
   a. Current proposal for grant
   b. New proposal for GIS projects
3. Prepare FY 1996 maintenance plan based upon Fall 95 walk-through observations
   a. New structures below Corps project & maintenance of Corps project
   b. Additional structures from Corps project to large gully & maintenance in this reach
   c. Pipe drop and reformation of large gully
   d. Maintenance from large gully to 1-F & additional structures from large gully to 1-F
   e. Maintenance at 2-A, additional structures above 2-A, & maintenance above 2-A
   f. Hydraulic observations
   g. Geomorphological observations
   h. Hydrological observations (1995)
   i. Vegetation observations

PROJECT RESULTS

After two years the first assessments of the success of the Muddy Creek Demonstration Stream Restoration project are positive. The grade control structures functioned as expected during a five year event in the spring of 1995. Scour holes more than ten feet in depth formed below structures where backwater from the downstream structure fails to reach the toe of the upstream structure. Estimating the distance between structures, as a siting issue, requires more research before a reliable procedure comes into practice. Barbs are stabilizing the toe of hundreds of lineal feet of unstable banks while providing a vegetative base. The public is responding favorably, even while erring in their estimate of the time scale for restoration of the stream corridor. Irrigation and range issues are progressing with the Irrigation District working to reduce return flows, and local landowners reducing grazing pressure in riparian zones. Overall, the Task Force is of the opinion that the project is successful and plans to expand the project.

Acknowledgments
Mr. Alan Rollo, the Muddy Creek Task Force Coordinator, and Mr. Danny Burgett, Area Conservationist, US Natural Resources Conservation Service, produced much of the material for this paper. Both have been key proponents of the project. The Reclamation Montana Area Office, the Reclamation Water Technology and Environment Research program, and the State of Montana, are funding this project.

CHANNEL EVOLUTION IN THE LOESS AREA OF THE MIDWESTERN UNITED STATES

Andrew Simon, U.S. Geological Survey, Raleigh, NC; **Massimo Rinaldi**, Universita degli Studi di Firenze, Italy; and **Greg Hadish**, Golden Hills Resource Conservation and Development, Oakland, IA

INTRODUCTION

In the highly erodible loess area of the midwestern United States (fig. 1), human disturbances to channels, flood plains, and upland areas near the turn of the 20th century ultimately led to accelerated erosion and canyon-like stream channels that resulted in severe damage to highway structures, pipelines, fiber-optic lines, and land adjacent to the stream channels. Accelerated stream-channel degradation has resulted in an estimated $ 1.1 billion in damages to infrastructure and the loss of agricultural lands since the turn of the century in western Iowa (Baumel, 1994). A survey of 15 counties in northwestern Missouri identified 957 highway structures as damaged by channel degradation. Degradation and channel widening in the loess area led to the collapse of several bridges in West Tennessee (Robbins and Simon, 1983), southwest Mississippi (Wilson, 1979), Missouri (Emerson, 1971), and southeast Nebraska. The 1993 floods in the midwestern United States focused additional attention on channel-stability problems in the loess area because scores of bridges were either closed or failed during and immediately after the floods.

Channel degradation in some streams has resulted in a fourfold increase in channel depth (20 feet) and an almost fivefold increase in channel width (98 feet) since the middle of the 19th century (Piest and others, 1976; 1977). Erosion rates from two severely degraded silt-bed streams in western Iowa were calculated at 131,000 and 145,000 tons/year (Ruhe and Daniels, 1965; Piest and others, 1976). In the 20 years of channel adjustment following modification of about 90 miles of channels in the Obion River System, West Tennessee, an estimated 221 million ft^3 of channel sediments were eroded and transported out of the system (11 million tons/year; Simon, 1989a). About 10 million tons/year were discharged from the Forked Deer River System, West Tennessee between the early 1970's and 1987. On average, about 18% of this material was eroded from the channel bed, the remainder coming from the channel banks (Simon, 1989a).

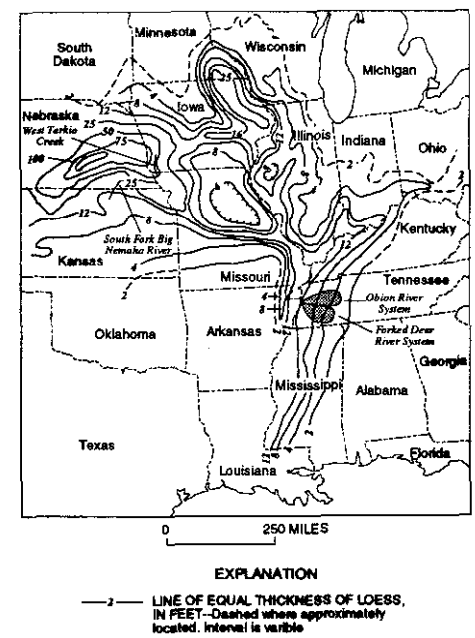

Figure 1: Location map of loess area of midwestern United States and thickness of loess (Modified from Lutenegger, 1987).

The purpose of this study, which is part of a larger one sponsored by the Federal Highway Administration, was to assess the general magnitude and extent of channel-stability problems in the loess area of the midwestern United States (fig. 1). This region represents a "worst-case" scenario where easily erodible soil has combined with extensive human disturbance to produce highly unstable stream-channel systems. Reconnaissance-level observations were made from a low-flying aircraft or by driving and noting stability problems along unstable streams in west, east-central, and southeastern Iowa; southeastern Nebraska; west-central Illinois; northeastern Kansas; and northwestern Missouri. Silt-bedded West Tarkio Creek, southwestern Iowa and northwestern Missouri, was investigated in detail to show typical channel changes. The sand-bedded Obion River System, West Tennessee, was studied previously (Simon, 1989b; 1989c; 1989c; Hupp, 1992; Simon and Hupp 1992; Simon, 1994).

FACTORS CONTROLLING CHANNEL STABILITY

Natural Factors: The predominant natural factors regarding channel-stability problems in the loess area of the midwestern United States are related to the relative resistance to erosion of the loess, and the distribution of loess

around and beneath the channel boundary. Although loess can stand in tall vertical cliffs when dry, it tends to be highly erodible when wetted by streamflow or by raindrop impact. Once disturbed, channels with beds of loess-derived alluvium generally incise rapidly and, without an ample supply of sand or gravel tend to be some of the deepest in the region. Because the fluvial transport of fine-grained sediments such as silt is not controlled by the hydraulic properties of the streamflow but by the supply of silt delivered to the flow from channel or upland sources, little silt is deposited in downstream reaches to aid in bed-level recovery. Channels that cut through the loess cap and entrain coarser sand and gravel deposits below will, on average, recover quicker from disturbances because there is a supply of hydraulically-controlled material that is likely to be deposited in downstream reaches, causing aggradation and the consequent decrease in bank heights.

The relatively low resistance (shear strength) of the loess bank material to erosion by mass failure is an important factor in channel evolution in the region. The mean values of cohesion and friction angle for 22 sites throughout the region are 203 lbs/ft^2 and 35.4^o, respectively (Lutenegger, 1987). The mean values for 14 shear-strength tests in Iowa bluff-line streams are 187 lbs/ft^2 and 24.9^o (Lohnes and Handy, 1968). Data indicate that, on average, loess in Kansas and Nebraska are considerably more cohesive, ranging from 720 to 1440 lbs/ft^2 (Turnbull, 1948; Lohnes and Handy, 1968). A study of channel banks in West Tennessee disclosed a mean cohesive strength of 182 lbs/ft^2, and a mean friction angle of 30.1^o (168 tests; Simon, 1989c). Following saturation and the generation of excess pore-water pressures, the shear-strength of the bank material is often insufficient to resist mass failure (Simon, 1989c). Along channels deepened by degradation, mass failure occurs on the recession of river stage as the banks lose the support afforded by the water in the channel. Established woody vegetation on streambanks can enhance bank stability but only if (1) degradation does not create bank heights and angles in excess of the critical shear-strength conditions of the bank material, and (2) rooting depths extend past the depth of the failure plane.

Human Factors: Large tracts of land were cleared for cultivation during settlement of the region prior to and after the Civil War (Ashley, 1910; Brice, 1966; Piest and others, 1977). The removal of grasses and woody vegetation resulted in reduced water interception and storage, increased rates of surface runoff, erosion of uplands, and gullying of flood plains and terraces. Surface runoff and peak-flow rates in western Iowa are estimated to have increased 2-3 times and 10-50 times, respectively, when compared to estimates of presettlement amounts (Piest and others, 1976; 1977). The removal of woody vegetation from streambanks resulted in decreased hydraulic roughness, increased flow velocities and stream power, and contributed to increased peak discharges. Much eroded material was deposited in channels resulting in a loss of channel capacity and frequent and prolonged flooding of agricultural lands (Morgan and McCrory, 1910; Moore, 1917; Piest and others, 1976). As a result, local drainage districts implemented programs to dredge, straighten, and shorten stream channels (channelize) to reduce flooding and thereby increase agricultural productivity (Hidinger and Morgan, 1912; Moore, 1917). By the 1930's most streams tributary to the Missouri and Mississippi Rivers in the loess area of the midwestern United States had been dredged and straightened (Speer and others, 1965; Piest and others, 1976; Lohnes and others, 1980; Simon, 1994). In many parts of the region these activities were conducted periodically throughout the 1960's and 1970's as additional tributaries were channelized and because some previously dredged channels filled with eroded sediment from upstream reaches (Lohnes and others, 1980; Simon, 1994).

STAGES OF CHANNEL EVOLUTION

In alluvial channels, disruption of the dynamic equilibrium often results in some amount of upstream channel degradation and downstream aggradation. If we consider the equilibrium channel as the initial stage (I) of channel evolution, and the disrupted channel as an instantaneous condition (stage II), rapid channel degradation would be stage III (fig. 2). Degradation flattens channel gradients and consequently reduces the available stream power for given discharges with time. Concurrently, bank heights are increased and bank angles are often steepened by fluvial undercutting and by pore-pressure induced bank failures near the base of the bank. Thus, the degradation stage (III) is directly related to destabilization of the channel banks and leads to channel widening by mass-wasting processes (stage IV) once bank heights and angles exceed the critical shear-strength conditions of the bank material. The aggradation stage (V) becomes the dominant trend in previously degraded downstream sites as degradation migrates further upstream because the flatter gradient at the degraded site cannot transport the increased sediment loads emanating from degrading reaches upstream. This secondary aggradation occurs at rates roughly 60% less than the

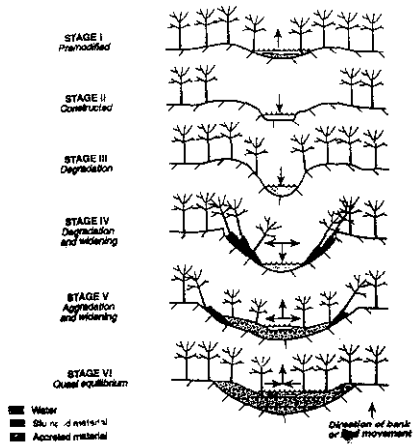

Figure 2: Stages of channel evolution.

associated degradation rate (Simon, 1992). These milder aggradation rates indicate that bed-level recovery will not be complete and that attainment of a new dynamic equilibrium (stage VI) will take place through further (1) bank widening and the consequent flattening of bank slopes, (2) the establishment and proliferation of riparian vegetation that adds roughness elements, enhances bank accrestion, and reduces the stream power for given discharges, and (3) further gradient reduction by meander extension and elongation. The lack of complete bed-level recovery results in a two-tiered channel configuration with the original flood-plain surface becoming a terrace. Stormflows are, therefore, constrained within this enlarged channel below the terrace level resulting in a given flow having greater erosive power than when flood flows could dissipate energy by spreading across the flood plain.

Channel Evolution in the Loess Area of the Midwestern United States: In western Iowa, stage of channel evolution of representative streams was identified by Hadish (1994) from aerial reconnaissance of 551 stream miles in 1993 and 989 miles in 1994. Results show that for both years, 56% of the stream lengths were classified as stage IV (widening and mild degradation). Bed-level recovery (stage V; aggradation and widening) is occurring along about 24% of the stream lengths, predominantly along the downstream-most reaches. This indicates that channel widening is currently the dominant adjustment process in the degraded streams of western Iowa, occurring along about 80% of the stream reaches observed. Only 6% were classified as either premodified (stage I) or restabilized (stage VI) (Hadish, 1994), indicating that about 94% of the stream lengths in western Iowa can be considered unstable. In West Tennessee, about 65% of the 1,645 studied sites are unstable, with channel widening occurring at about 60% of them (Bryan and others, 1995). In comparison, stage V conditions appear to dominate much of the larger sand-bed streams in southeast Nebraska. Stage IV conditions are common, however, along tributary streams. In contrast to western Iowa where stage III conditions can be found in the upstream reaches of most tributary streams, in southeast Nebraska only the smallest "finger-tip" tributaries near basin divides can be classified as stage III.

In West Tennessee, similar comparisons can be made between channel evolution in streams with silt beds (that dominate western Iowa) and those with sand beds (that dominate southeastern Nebraska). Tributaries of the largest, modified West Tennessee streams rise in unconsolidated sand-bearing formations that supply sand to the channels as bed material. Aggradation in downstream reaches occurs after 10-15 years of incision and channel widening occurs at moderate rates (Simon, 1989b). In contrast, smaller tributary streams near the Mississippi River bluff have cut only into loess materials, have silty beds, and no source of coarse-grained material. To reduce erosional forces and stream power for a given discharge without a coarse-grained sediment supply for downstream aggradation, channel widening (stage IV) may be the only mechanism for the silt-bed streams to recover (Simon, 1994). The silt-bed channels are the deepest and most rapidly-widening channels in West Tennessee and may take hundreds of years to recover. This is similar to the western Iowa silt-bed streams such as West Tarkio Creek where downcutting (stages III and IV) has lasted for as much as 70 years and channel widening is widespread. These findings indicate that:

(1) There is more coarse-grained sediment (sand and gravel) available for bed-material transport in southeastern Nebraska and West Tennessee than in western Iowa;
(2) Channel evolution and recovery in southeastern Nebraska and West Tennessee is further advanced than in western Iowa because of plentiful supplies of sand;
(3) Channel widening dominates all areas; and
(4) Degradation can be expected to continue to migrate upstream in western Iowa tributaries.

Reconnaisance in west-central Illinois and east-central and southeastern Iowa disclosed many simuous streams with bank failures and meander extension occurring on outside bends from the growth of point bars on the opposite, inside bank. This is typical of late stage V and stage VI channels. Bank heights did not appear to be nearly as high as in southeast Nebraska, or western Iowa where bank heights between 27 and 35 feet are common. Assuming that

channel modifications occurred in west-central Illinois and east-central and southeastern Iowa at about the same time as in western Iowa, it seems that many of the eastern loess-area streams have recovered more quickly probably because the thinner loess cap has been penetrated by the streams, exposing an ample supply of coarser sand and gravel in this part of the loess area.

The 1993 floods did not result in a rejuvenation of previously unstable channel systems or accelerated bed-level adjustments. Gully extension into agricultural fields occurred. The predominant effect of the 1993 floods on channel instability of tributary streams in the loess area of the midwestern United States was channel widening. This occurred because of removal of toe material and prolonged saturation of channel banks, resulting in reduced shear strength and failures upon recession of storm flows. In straight reaches, low-cohesion channel banks up to 45 feet-high fail readily at average rates of almost 7 feet per year. In reaches characterized by alternate bars, moderate flows erode low-bank surfaces on the opposite side of the channel resulting in steep bank angles, and accelerated rates of channel widening. The combination of alternate bar growth and accelerating widening on the opposite bank represent incipient meandering of the channel within the straightened alignment. Outside meander bends generally erode more rapidly than in straight reaches because the buttressing effect of failed bank material is removed from the bank toe by fluvial action. This process was exacerbated by the 1993 floods.

WEST TARKIO CREEK, IOWA AND MISSOURI

Trends of channel changes on West Tarkio Creek were estimated using historical bed-elevations, bank profiles, shear-strength tests, bed-material sizes, stages of channel evolution, and dendrochronologic information. Historical bed-level data were compiled from cross-section surveys and longitudinal profiles (fig. 3; Piest and others, 1977; Lohnes, R.A., Iowa State University, written commun., 1994). Bed-elevation data for 1994 and the additional field data were collected during the summer of 1994.

West Tarkio Creek is a typically unstable stream in the loess area of the midwestern United States that has undergone human disturbance. It has been undergoing renewed channel adjustment since being straightened in about 1920 and displays the systematic variation of stage of channel evolution with distance upstream; aggradation (stage V) in its downstream-most reaches merging to widening with mild degradation (stage IV), to rapid degradation (stage III), with increasing distance upstream (fig. 4). The dominant bed-material size class generally varies systematically with the stage of channel evolution and distance upstream; sand beds in aggrading reaches and silt beds in degrading reaches. Piest and others (1977) describe stable reaches of West Tarkio Creek in 1975 near its junction with the Tarkio River at a channel gradient of 0.0006 ft/ft. It is assumed that by stability the authors were referring to a lack of further downcutting. By 1994, aggrading conditions on West Tarkio Creek extended at least 5 miles into Iowa to the J52 bridge (fig. 4). Assuming that stable conditions in 1975 were at the confluence with the Tarkio River and were about 15.5 miles further upstream in 1994 than in 1975 (19 years), a rate of migration of 0.82 mi/yr is estimated.

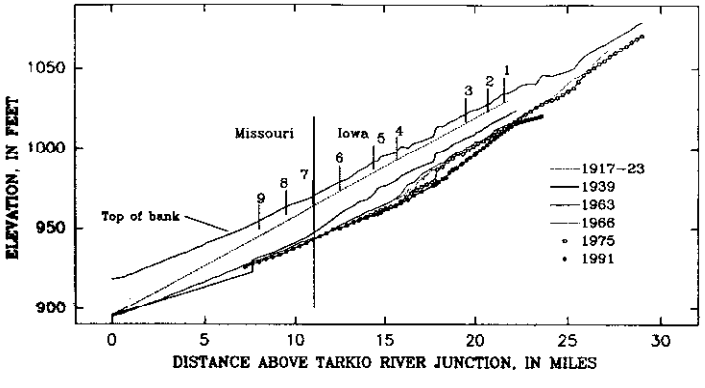

Figure 3: Channel profiles of West Tarkio Creek, Iowa and Missouri. Numbers refer to locations where trends of degradation were fit with equation 1.

The age of woody vegetation growing on streambanks is used as an indicator of bank conditions and the timing of renewed bank stability (Hupp and Simon, 1986; 1991; Hupp, 1992). It is probably coincidental, however, that the oldest riparian tree found at site T2 (fig. 4) is 19 years old. Still this indicates that sections of streambanks have been stable for some time at site T2. Ages and germination dates of the oldest streambank trees at other sites are shown in figure 4, indicating the timing of renewed bank stability along at least some low-bank surfaces. Note that

average, recent widening rates, as estimated from dendrochronologic evidence generally increase moving from stage V conditions in downstream reaches, to stage IV conditions further upstream (fig. 4).

Bed-Level Adjustments: Trends of bed-level changes were obtained by fitting historical data to an exponential decay equation (Simon, 1992):

$$z / z_0 = a + b\, e^{(-k\,t)} \qquad (1)$$

where z = the elevation of the channel bed (at time "t");

z_0 = the elevation of the channel bed at (t_0=0);

a = the dimensionless coefficient, determined by regression and equal to the dimensionless elevation (z/z_0) when eq. (1) becomes asymptotic, a>1 = aggradation, a<1 = degradation;

b = the dimensionless coefficient, determined by regression and equal to the total change in the dimensionless elevation (z/z_0) when eq. (1) becomes asymptotic, b>0 for degradation, b<0 for aggradation;

k = the coefficient determined by regression, indicative of the rate of change on the channel bed per unit time; and

t = the time since the year prior to the onset of the adjustment process, in years (t_0=0).

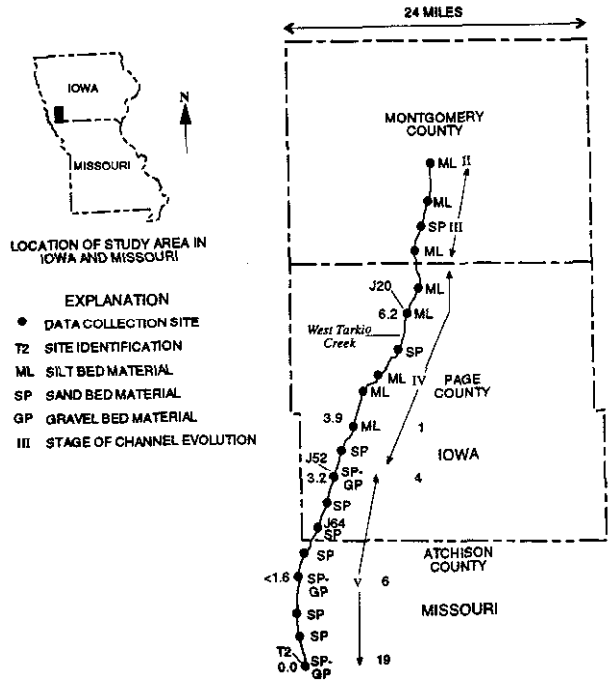

Figure 4: West Tarkio Creek showing stages of channel evolution, dominant bed-material size class, age of oldest riparian tree, in years (numbers on right), and widening rate, in feet per year (numbers on left).

Although aggradation rates along the downstream-most 8 miles of channel could not be estimated using this method because of a lack of recent survey data, accretion measured around trees on bank surfaces, particularly at the sites in Missouri confirmed that deposition was an ongoing process. Degradation trends at 9 sites are shown in figure 5. The 3 plots in the bottom row of the figure show that degradation along the downstream-most 11 miles of channel is virtually complete. Considerable degradation, continuing beyond the year 2000 can be expected along upstream reaches or until the curves become asymptotic (fig. 5). An empircal model of bed-level adjustment for West Tarkio Creek is obtained by plotting the a-value coefficient (from eq. 5) against distance from the junction with Tarkio River (fig. 6). The future elevation of the channel bed, when degradation has reached a minimum, can be obtained by multiplying the a-value at a site (fig. 6) by the initial bed elevation. Maximum degradation (minimum a-values) along West Tarkio Creek is estimated to occur near river mile 20 (fig. 6), in comparison to Piest and others (1977) observation that it would occur near the Iowa-Missouri stateline (river mile 11).

Channel-Width Adjustments: Application of analytic methods regarding bank instability are demonstrated for West Tarkio Creek in the vicinity of the J52 bridge, 5 miles north of the Iowa-Missouri stateline (fig. 4). In-situ shear-strength tests at 2.8- and 4.9-feet deep were taken about 150 feet downstream of the bridge. A mean cohesive strength (c) of 187 lbs/ft^2 and a mean friction angle (ϕ) of 29.8^o were obtained. The mean, bulk unit weight (γ) of two undisturbed samples of the bank material was 138 lbs/ft^3. A stability number (N_s) of 6.90 was calculated using ϕ = 29.8^o, assuming a vertical (90°) bank slope (i), and (Lohnes and Handy (1968):

$$N_s = (4 \sin i \cos \phi) / [1 - \cos (i - \phi)] \qquad (2)$$

Critical bank height (H_c; above which there would be mass failure) is obtained by substituting N_s into the following equation from Carson and Kirkby (1972):

$$H_c = N_s\, (c / \gamma) \qquad (3)$$

For i = 90°, H_c = 9.36 feet. Iterating for bank angles of 80, 70, 60, 50 and 40°, results in a bank-stability chart

for ambient field conditons at the J52 site (upper line; fig. 7). H_c at 40° is not plotted because the value (139 feet) is considerably greater than any channel bank on West Tarkio Creek. This entire procedure is then repeated assuming that the banks are saturated and that $\phi = 0.0$ (Lutton, 1974) to obtain H_c values under staurated conditions, resulting in the lower line of figure 7. The H_c values obtained range from 5.4 feet at a bank angle of 90° to 14.9 feet at a bank angle of 40° (fig. 7). The effect of staurated condtions on decreasing H_c can be seen by drawing a vertical line anywhere on figure 7 and comparing the difference in values at the intersection of the ambient- and saturated-condition lines.

Using the slope angles of the vertical face and upper bank to obtain an average bank angle of 62°, and using a surveyed bank height of 35 feet (vertical distance from bank top to thalweg), channel banks at J52 plot between the at-risk and unstable zones, fig. 7). If the angle of the "vertical face" (81°) is used to represent worst-case conditions with bank toes eroded, the banks are clearly unstable, even when relatively dry. This is probably an unrealistic scenario, but provides the end-member stability case for unstable banks. Because the shear-strength characteristics at the J52 site are similar to average values obtained in studies of western Iowa (Lohnes and Handy, 1968) and West Tennessee (Simon, 1989c; Simon and Hupp, 1992), it seems reasonable to use the bank-stability chart (fig. 7) to approximate stability conditions at other sites on West Tarkio Creek. Average and worst-case bank geometries are plotted for several other sites on West Tarkio Creek to show current (1994) bank-stability relations.

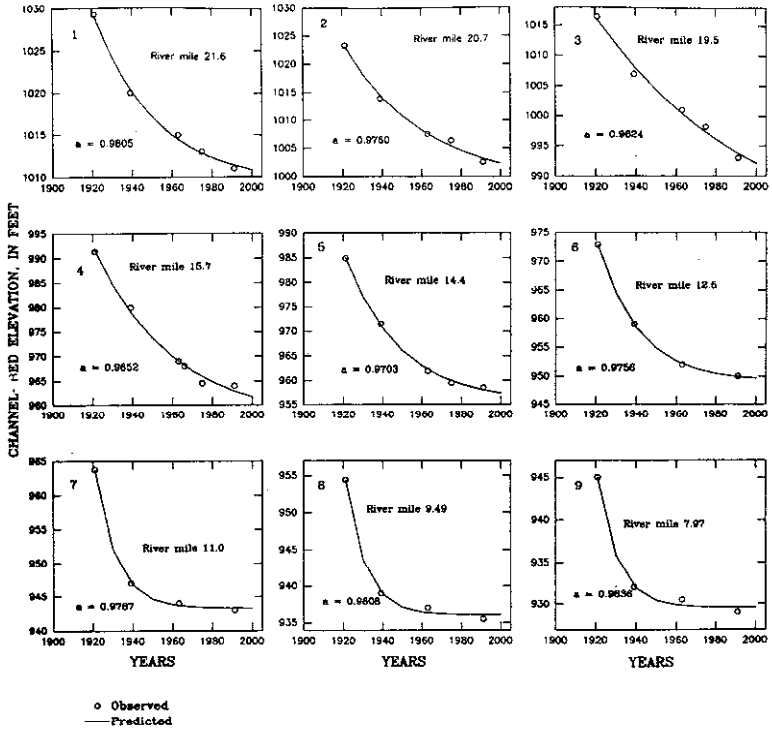

Figure 5: Degradation trends at 9 sites on West Tarkio Creek fit with equation 1. Numbers refer to location of cross sections shown in figure 3.

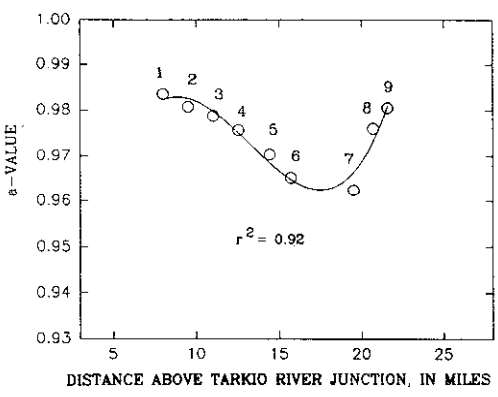

Figure 6: Longitudinal distribution of a-values (from equation 1) representing an empirical model of bed-level response for West Tarkio Creek. Numbers refer to location of cross sections in figure 3.

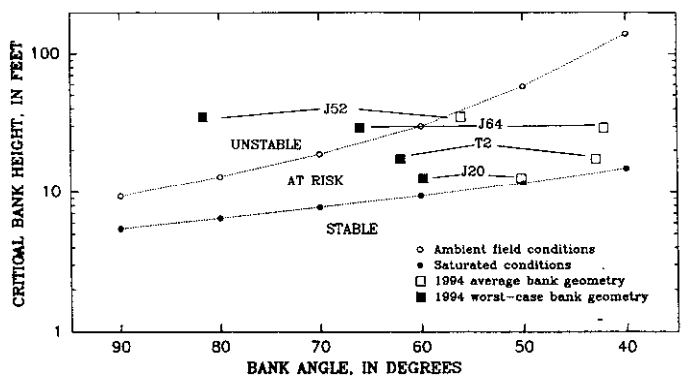

Figure 7: Bank-stability chart for conditions in the vicinity of the J52 bridge over West Tarkio Creek. Locations of sites J52, J64, T2, and J20 are shown in figure 4.

SUMMARY AND CONCLUSIONS

The loess area of the midwestern United States contains thousands of miles of unstable streams, predominantly responding to human modifications imposed near the turn of the 20th century. The dominant process appears to be channel widening and meander extension by mass failure. The most severe, widespread instabilities are in western Iowa where a thick loess cap and a limited supply of coarse-grained (sand and gravel) material restricts bed-level recovery by aggradation. Here relatively high streambanks combine with low cohesive strengths to sustain channel widening. Channel adjustments in West Tennessee and southeastern Nebraska are not as severe as in western Iowa. In general, channels in west-central Illinois and east-central Iowa appear to be closer to recovery than in other parts of the region.

Degradation rates for streams draining the loess hills of western Iowa and eastern Nebraska have decreased nonlinearly since 1920, approaching minimal values. In some downstream reaches, sandy alternate bars and fluvially deposited sand on low-bank surfaces indicate the beginning of bed-level recovery and the "aggradation" stage of channel evolution. This "natural" recovery process which often occurs in other unstable stream systems following 10-15 years of degradation, was apparently delayed for an additional 55 years in the loess area because of the lack of hydraulically-controlled (sand- or gravel-sized) source material. Finer-grained silts from eroding beds and banks were easily transported through tributary systems to the Missouri River. With loess thickness generally decreasing with distance upstream and with the degradation process migrating upstream with time, incision through the loess ultimately resulted in the exposure of coarser-grained glacial till. These deposits, thus, provided the coarse material necessary for downstream aggradation and the initial phase of channel recovery.

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MORPHOLOGICAL EVOLUTION PROCESS NEAR THE CHOU-SHUI RIVER MOUTH IN TAIWAN

By Hwung, H. H.; Professor, Department of Hydraulics and Ocean Engineering, N.C.K.U., Director, Tainan Hydraulics Laboratory, Tainan, Taiwan; Shieh, C. L. Associate Professor, Department of Hydraulics and Ocean Engineering, N.C.K.U., Vice Director, Tainan Hydraulics Laboratory, Tainan, Taiwan; Liou, J. Y., Associate Researcher, Tainan Hydraulics Laboratory, Tainan, Taiwan; Tseng, C. M., Ph.D. Candidate, Research Institute of Hydraulics and Ocean Engineering, N.C.K.U. Tainan, Taiwan

Abstract Chou-Shui Basin is located at the middle west of Taiwan. The main channel flows from east to west and then into Taiwan Strait. The annual sediment transport rate of Chou-Shui River is the highest among all rivers in Taiwan. Due to its abundant sand source, there is a wide alluvial fan in the downstream of Chou-Shui River. In this paper, the morphological evolution process around the Chou-Shui River mouth is analyzed based on past morphological information and recent continuously measured field data. Results show that, during the flood period, huge amount of sediments are carried by flood from upstream to the mouth of Chou-Shui River and cause significant topographical change. These depositions are then redistributed along the coast by wave and tide during the dry season. Their distributions range from Fang-Yuan to Hsin-Huwei River. Due to small grain size, the influence of suspended load from the Chou-Shui River on the coastal topographical changes can be neglected.

INTRODUCTION

Chou-Shui River is the largest river in Taiwan(Fig. 1). Before 1911, the main channel of Chou-Shui River migrated frequently on the alluvial fan to and fro in south-north direction. During that period, the sediment carried by the river deposited uniformly along the front of alluvial fan which was around 65 kilometers in length. This deposit became the sediment source which increased the coast growing. However, since 1911, Chou-Shui River was then constrained at the present location by artificial dike. Since then, the type of sediment supply from river to coast turned out to be a point source instead of the original line source. This kind of change resulted in a violent morphological evolution in the coastal area nearby. Therefore, the present paper is interested in the process and mechanism of the morphological evolution in this region. From long-term point of view, the past coastal morphological information and the recent continuously measured field data are analyzed and discussed in this paper .

year.month	H1/3 (m)	T1/3 (sec.)
1993.01	1.2	5.5
1993.02	0.9	5.2
1993.03	0.8	5.3
1993.04	0.7	4.8
1993.05	0.5	3.9
1993.06	0.4	3.9
1993.07	0.4	4.0
1993.08	0.6	5.0
1993.09	0.7	5.1
1993.10	1.3	5.8
1993.11	1.1	5.6
1993.12	1.4	5.9

Table 2. Measured wave data

Station:Tzyh-chyang B.		
Month	Observation Interval (Year)	Max. Daily Discharge (cms)
1	1986-1990	97.0
2	1986-1990	30.5
3	1986-1990	244.0
4	1986-1990	1330.0
5	1986-1990	618.0
6	1986-1990	2390.0
7	1986-1990	2660.0
8	1986-1990	3360.0
9	1986-1990	3880.0
10	1986-1990	1000.0
11	1986-1990	139.0
12	1986-1990	58.3

Table 3. The maximum daily discharge in station of Tzyh Chyang B.

Date:06/08 1993

Depth (m)	dm (μ m)
0.3	4.53
0.5	5.53
0.8	4.78
1.0	5.57
1.0	4.05
0.8	6.50
0.5	6.42
0.5	4.60
0.3	3.99

Table 4. The diameter analysis of the suspended load in Station of Tzyh Chyang B.

As to the grain size of sediment on the sea bed, Tainan Hydraulic Laboratory has performed a detailed measurement at the study area in March, 1992. The sampling locations are shown in Fig. 4. Results of investigation show that the medium grain size of sediment on the sea bed is between 0.01 mm and 0.57 mm. The averaged mean grain size is about 0.22 mm. However, since the effective grain sizes (D10) of sediments taken on the sea bed between Fang-Yuan and Hsin-Huwei River are all larger than 0.005 mm (see Fig. 5), the influence of suspended load from the Chou-Shui River on the coastal topographical changes should be neglected.

Bathymetry

Chou-Shui River is the largest river in Taiwan. During flood period, huge amount of sediments are carried to downstream and significantly affect the topographical changes around the river mouth. According to the past topography data (as shown in Fig. 6), the -6 m and -15 m bathymetry contour lines between Fang-Yuan and the mouth of Chou-Shui River move offshore 3 km and 2 km respectively during the 1961-1977 period. While the topographical changes in the place north of Fang-Yuan are insignificant, the occurrences of depositions and erosions mix together.

Fig. 7 and Fig. 8 show the comparisons of bathymetry contour lines during the period between 1972 to 1991. Results of -4m, -6m and -10m contour lines reveal that severe depositions occur in the area between Fang-Yuan and Hsin-Huwei River. In the area between Hsin-Huwei River and Taishi, topographical changes are trivial.

CONCLUSION

Results of field survey reveal that morphological evolution process near the Chou-Shui River mouth are dominated by three dynamic conditions, i.e. river flood, wave and tide. During the wet season , huge amount of sediments are carried by flood from upstream to the mouth of Chou-Shui River and cause significant topographical change. These depositions are then redistributed along the coast nearby by wave and tide during the dry season. Generally, most of the topographical changes around the mouth of Chou-Shui River are depositions. Their distributions range from Fang-Yuan to Hsin-Huwei River. While due to small grain size, the influence of suspended load from the Chou-Shui River on the coastal topographical changes can be neglected.

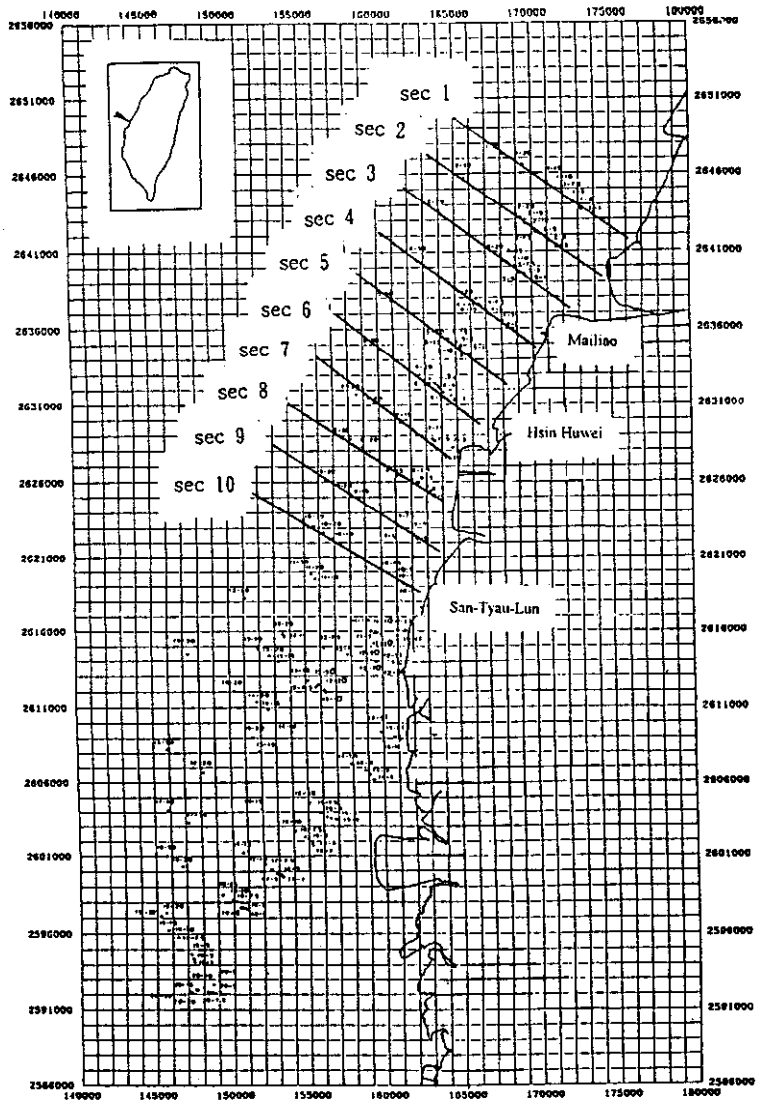

Figure 4. Location of the sea bottom material investigation

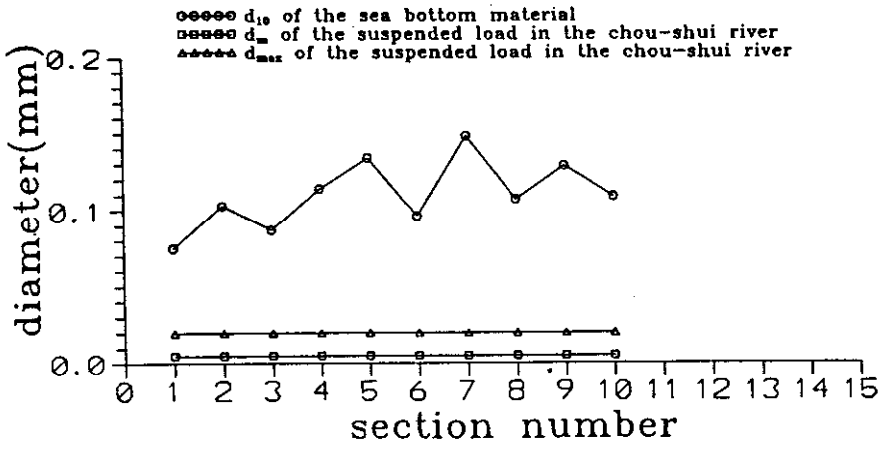

Figure 5. The distribution of the effective diameter d_{10} of the sea bottom material

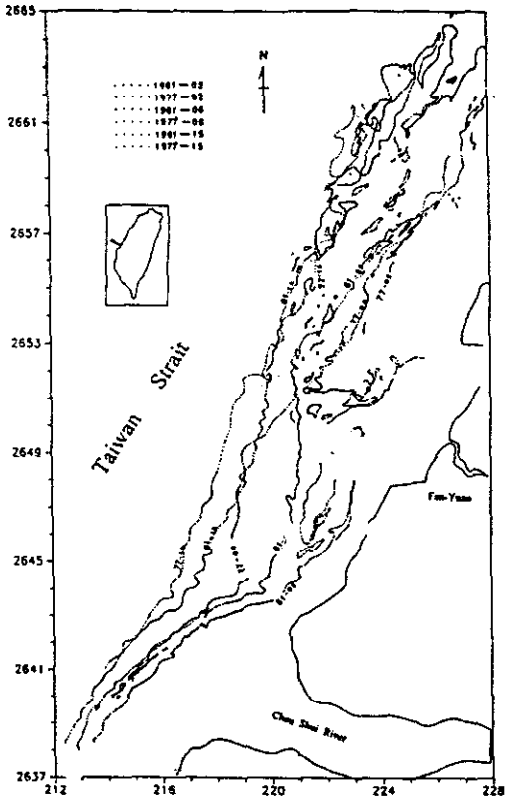

Figure 6. Topographical changes of Sea bottom near the Chou Shui River mouth from 1961 to 1977

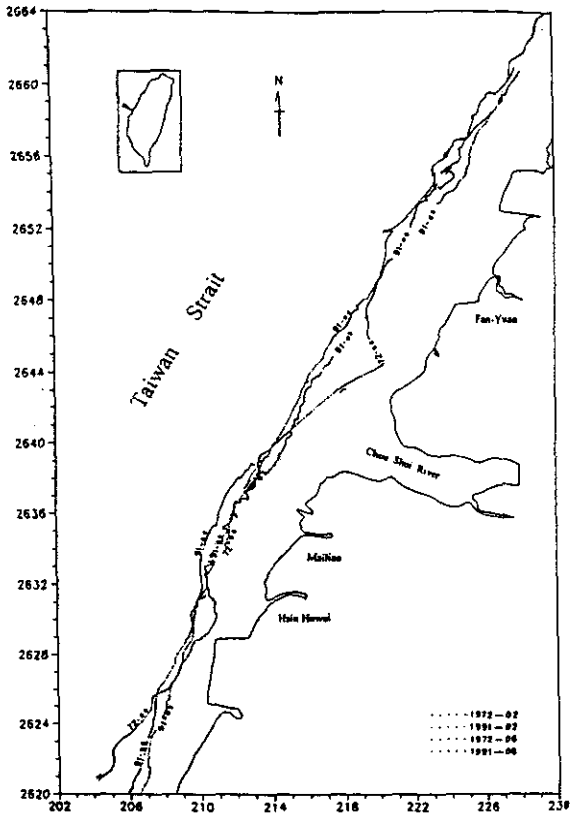

Figure 7. -2m and -6m contour lines changes near the Chou Shui River mouth from 1972 to 1991

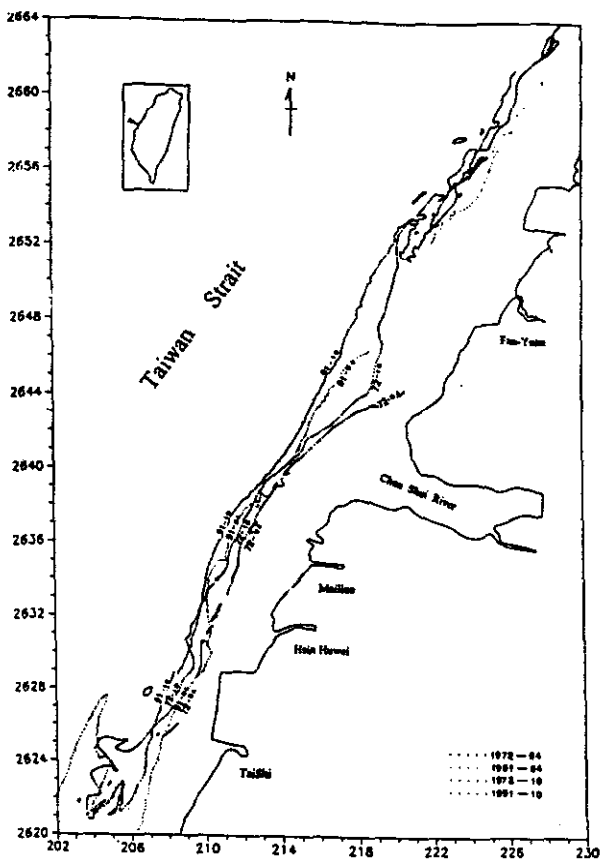

Figure 8. -4m and -10m contour lines changes near the Chou Shui River mouth from 1972 to 1991

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SIMULATED STREAM BANK EROSION OF SURFACE AND SUBSURFACE SAMPLES FROM THREE MEADOW PLANT COMMUNITIES

Sherman Swanson, Associate Professor, Department of Environmental and Resource Sciences, University of Nevada, Reno and Iradj Kamyab former Research Assistant[1]

Abstract Although the erosion resistance added by plant roots has often been noted, few experiments have compared plant communities in controlled simulations of stream erosion. To measure erosion resistance of surface (0-.1 m) and subsurface (.3-.4 m) soils in three meadow communities, intact soil samples were placed in the wall of a flume used to simulate stream bank tractive stress at three levels. Soil loss differed among three levels of tractive stress and was greater for subsurface soils than for the more densely rooted surface soils. The dry, Kentucky bluegrass, *Poa pratensis*, dominated, community lost more surface soil than wetter communities dominated by Baltic rush, *Juncus balticus*, and Nebraska sedge, *Carex Nebraskensis*. Many soil and root variables differed among community types, especially those that reflect the great root mass of plant communities living in saturated soils. Many of these variables similarly differed between depths. These results suggest the importance of wetter riparian vegetation types for the stability of certain streams.

INTRODUCTION

Soil and vegetation control streambank stability in alluvium (Smerdon and Beasley 1959, Moore and Frank 1961, Grissinger 1962, Smith 1976, Enger 1964, Wolman 1959). The importance of soil properties could be expected from the work of Hjulstrom (1935), Sundborg (1956), Smerdon and Beasley (1959), Dunn (1959), Schumm (1969), Schumm and Lichty (1963), Wolman (1959), Wolman and Bush (1961) and Grissinger et al. (1981). Hjulstrom (1935) and Sundborg (1956) documented an increase in critical tractive stress or water velocity needed for erosion of soils with particle size decreasing into the clay range. However, Sundborg (1956) also recognized the importance of interparticle phisico-chemical forces. Smerdon and Beasley (1959) correlated the tractive force necessary to erode cohesive soil to the plasticity index, dispersion ratio, mean particle size and percent clay. Grissinger et al. (1981) found that stream bank erosion measured in the field with a portable flume correlated with increased dispersion and decreased clay. However, Partheniades (1971) concluded that erodibility is complex and that indices such as percent clay, plasticity index, and dispersion ratio are in effect secondary indices of the internal force system in soil.

Mackin (1956), Hadley (1961), Grissinger (1962), Brice (1964), and others show the effects of vegetation on channel stability. Smith (1976) showed that bank material with 16 to 18 percent by volume of roots had 20,000 times more resistance to erosion than comparable soil without vegetation. Zimmerman et al. (1967), Hey and Thorn (1986) and Murgatroyd and Ternan (1983) found differences in channel form coincident with riparian vegetation. Zimmerman et al. (1967) and Murgatroyd and Ternan (1983) explained these data by greater roughness associated with forests and greater sheer strength of bed and banks from herbaceous vegetation. Garofalo (1980) attributed differences in channel pattern and migration rate to differences in vegetation types and consequent soil holding capacity between saline and freshwater tidal environments. Manning et al. (1989) concluded that root length density varies among meadow-plant communities and recommended research to determine how rooting patterns influence bank stability. Kleinfelder et al. (1992) and Dunaway et al. (1994) concluded that both soil and root properties were significant in bank erosion. They suggested that studying the interaction of soil and vegetation is essential for understanding streambank erosion.

Channel shape and erosion resistance controls its reaction to the regimen of water and sediment discharge. For some stream types (Rosgen 1994) channel shape and pattern maintenance requires vegetation. Watershed or riparian management depends on understanding the effect of vegetation and vegetation

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[1]Research was funded by Nevada Department of Wildlife and the University of Nevada, Agricultural Experiment Station project 682.

influencing activities on erosion processes (DeBano and Schmidt 1989). Engineering to replace an overly straight channel with a meandering channel, requires knowing the erosion resistance of new channel materials. Many watersheds have lost water storage capacity and many meadows lost productivity because of weakened stream bank vegetation and a lowering water table (Van Havern and Jackson 1986, Swanson 1988, and Swanson and Myers 1994). However, the comparative streambank erosion resistance of the roots of various plant communities has not often been measured (Smith 1976, Garofalo (1980) and Dunaway et al. 1994). Further research is needed on the effect of bank materials and vegetation on river meander initiation (Rhoads and Welford 1991) and many other aspects of streambank stability and stream response to riparian vegetation (Swanson and Myers 1994).

This study applied consistent planned levels of tractive stress, simulating different stream flows or bank positions, to profile samples of surface sod and undisturbed subsoil from three plant communities. The plant communities were selected to represent the wide range of soil water (and therefore vegetation) and narrow range of other soil properties in one meadow. It focuses on measures of plant roots and on soil properties that others have correlated to erosion.

STUDY AREA

The study area is in Slinkard Valley, Mono County, California, approximately 58 km south-southwest of the obtuse angle of the Nevada-California border. Riparian vegetation stabilizes existing channel reaches and could help stabilize a redesigned stream channel. A spring feeds the stream in meadows formed in extensive Quaternary alluvium (Moore 1969) that was eroded and transported from the peripheral mountains and deposited as an outwash train. Soils classified according to Soil Taxonomy (USDA 1975), are Histic Haplaqualls for wet sites and Cumulic Haplaqualls for semi-wet and dry sites. Precipitation records in this area are meager. Only one long-term record exists in the California portion of the basin. It shows an average precipitation of 277 mm per year, mostly as snow (Kamyab 1991). Summer temperatures are mild, rarely exceeding 32° C in the valley or 21° C at higher elevations. The winters are severe and below-freezing temperatures often occur. We selected wet, semi-wet and dry meadow communities, consisting mainly of the dominant species Nebraska sedge (*Carex nebraskensis*), Baltic rush, (*Juncus balticus*), and Kentucky bluegrass, (*Poa pratensis*), respectively. The meadows were heavily grazed by cattle for many decades but had been only moderately grazed in recent years. Gopher activities are obvious at all dry sites in the study area.

METHODS AND MATERIALS

Upon general completion of stand mapping in spring, 1989 and with the aid of aerial photographs, we selected five stands of three plant communities, (total = 15) in one meadow. Each study plot was located near the center of a 100 m^2 minimum size stand of relatively homogeneous vegetation, thus avoiding ecotones. Three sampling locations per plot were randomly selected for soil coring from October through December 1989. For the flume test, we collected .07 m diameter soil core samples to a depth of .1 m and in the interval between .3 and .4 m with a metric volume core soil sampler (Arts Manufacturing, American Falls, ID). These intervals were selected to contrast the most densely rooted depth increment with a lower depth that was considerably less well rooted (Manning et al. 1989). Extra samples were collected for root and soil analyses at the same times and locations as erosion samples.

Soil core sleeves split longitudinally allowed sample extraction from the corer without damage. Wrapping samples in similar PVC pipe sleeves and plastic bags prevented damage in transport to the lab. Erosion samples were kept frozen until cut vertically in half and trimmed with a hacksaw. Cut and trimmed samples were then fixed in a holder with nails 100x3 mm through the rear of the sample. The soil and holder were then oven dried at 30° C and weighed.

We tested samples with a flume to simulate streambank erosion tractive stress. The laboratory flume for this study was 26.8 m long, 0.9 m wide, and 0.9 m deep with slope adjustable to 1.8 percent. A narrower test section, where the soil samples were placed was 0.3 m wide, 0.3 m deep and extended 3.0 m upstream

and 1.5 m downstream. These dimensions were selected to approximate the channel size near the Slinkard Creek study area and the one-to-one width depth ratio sometimes observed for stable E4 channels (Rosgen 1994) in Sierra Nevada Meadows. Two pumps recirculated up to 15 m^3 of water per minute to the flume through a constant-head tank. At the downstream end of the flume, vertically-sliding pieces of plywood, known collectively as the tailgate, achieved conditions of steady uniform turbulent flow.

The samples were placed into the wall of flume. The sample holding device was bounded top and bottom with removable wall parts and backed by a stilling well. This minimized eddy currents, pressure differences, and net flow past the sample through the flume wall. The .1 X .054 cm cut face became part of the vertical wall plane parallel to the general direction of flow. Presoaking samples for 24 hours eliminated variation in strength due to variation in wetness or wetability and simulated the wetness of banks during spring erosive events.

When flow conditions stabilized, we removed a protective shield. Slight adjustments brought the sample face into the plane of the flume wall. Then soil erosion was allowed for thirty minutes. This time was selected to exceed the time required for some near-surface (high root-length density) samples to stop eroding in preliminary studies. Following the test, we replaced the shield, stopped the flow, and removed the sample and holder for oven drying to measure soil loss.

The three levels of tractive stress selected for simulation were: 1. The minimum force required to initiate a general movement of detached sand particles 2 mm in diameter (0.59 N/m^2, according to Schoklitsch 1950); 2. The average tractive stress per unit of bankfull wetted perimeter (6.2 N/m^2), based on the Du Boys (1879) equation; and 3. The maximum tractive stress expected along parts of a meandering channel (15.7 N/m^2) in relation to average tractive stress. We measured the existing E4 (Rosgen 1994) channel upstream from the study site to figure out bankfull channel dimensions ($R = 0.12$ m and $S = 0.00513$). Ippen and Drinker (1962) and Yen (1970) used a channel with 60° curves at a width/depth ratio of eight and width/radius (relative curvature) of 0.8 to learn that maximum tractive stress was 2.4 times the average stress. Because this channel differs from the natural E4 channel shape, this tractive stress may not represent natural maxima.

Because tractive stress in turbulent flow is proportional to velocity squared, we measured average velocity for each flume configuration. The average velocities (.47 and 1.47 m $second^{-1}$) we created to simulate low and moderate tractive stress over the range of average velocities reported for similar E3 channels by Rosgen (1994) (.48 at low flow to 1.49 m $second^{-1}$ at bank full flow). These velocity values appear consistent with those reported for similar slope and width to depth ratio streams in Barnes (1967). Therefore, tractive stress varied within a range sufficient to evaluate flow conditions close to those needed (Table 1). To avoid confusion, we will describe them as low, medium, and high. Additional hydraulic data (presented in Table 1) show that flow conditions ranged from subcritical flow for the low and mid levels of tractive stress to critical for the high level. For all three levels of tractive stress, the flow was turbulent ($R_e > 500$) as shown by the size of the Reynolds number.

To obtain a direct measure of tractive stress, we constructed a device to place in the space normally taken up by samples in the flume. The device became deflected slightly by moving water and registered deflection with a signal from a strain gauge (SR4) in a meter (Baldwin Lima Hamilton, SR4). Then we measured tractive stress with weights hung below a pulley to duplicate the deflection and strain gauge signal of each tractive stress level. The face plate of the device was smooth steel, like the painted steel flume wall, (.1 m X .054 m) during one set of runs. It was coated with coarse open grained sandpaper (Adalox A211, 36-D), simulating the roughness of our samples for a duplicate set of runs. The measured tractive stresses are presented in Table 1 as a range (smooth to sandpaper covered) of values.

Table 1. Summary of hydraulic conditions at the three levels of tractive stress applied as steady turbulent flow and a width/depth ratio = 1.

Relative tractive stress	Du Boys tractive stress N/m^2	Average velocity m/sec.	Strain[1] gauge N/m^2	Froude number tractive stress	Reynolds number	Discharge m^3/sec. (F)	Slope of flume (R_e)
low	0.59	.47	1.1-1.2	0.28	4.8×10^4	.042	$6. \times 10^{-4}$
medium	6.21	1.47	9.7-10.3	0.86	14.7×10^4		.133
6.3×10^{-3}							
high	15.70	2.43	17.0-17.6	1.42	24.1×10^4		.219
1.6×10^{-2}							

1. The range represents the difference between a smooth surface and a sandpaper covered surface of the strain gauge.

Temperature for each run was measured after reaching equilibrium flow conditions. Temperature varied between 19° C and 22.5° C and therefore viscosity varied between 0.01034 cm^2/sec and 0.0095 cm^2/sec. This was judged not important and was disregarded in data analysis.

Roots were separated from the soil in duplicate samples using a hydropneumatic illutriator and 0.5 mm mesh screens (Gillison's Inc., Benzonia, Mich.). Only functional roots, those with an intact stele, were measured. Most big nonfunctional roots and debris were separated from functional roots by floating the washed sample in water. Remaining roots and debris, which floated or stayed in suspension, we then sorted into four categories. We selected the categories: rhizomes, coarse roots (diameter 1 mm or greater), fine roots (diameter 0.5 to 1 mm), and very fine roots (diameter less than 0.5 mm) to compare erosion resistance by different structure, size (and presumably strength), and surface per unit mass fractions. The lengths of rhizomes, coarse and fine roots we measured manually to the nearest mm. The length of very fine roots we measured with a root length scanner (Commonwealth Air Craft Corp., Melbourne, Australia) that counted intersections and used the approach of Newman (1966). Scanned samples were oven dried at 75° C for 24 hours and weighed. To remove the contribution of debris and nonfunctional roots from the very fine roots, a subsample was counted manually for functional versus debris plus nonfunctional root intersections using a 10 power binocular microscope. Root classes for each sample were oven dried and weighed to 0.01 grams. Because very fine roots comprised more than 95% (usually >99%) of total root length in all samples, it was replaced by total root and\or rhizome length density (RLD)in analyses. Very fine (< .5 mm diameter) roots always dominate total RLD.

Soil cohesion was indirectly represented by many parameters including, organic carbon (Medius 1960), plasticity index (American Society for Testing Materials Committee D-18 1950), dispersion ratio (Middleton 1930), particle size distribution (phi index) and clay content (Krumbein 1963, Kilmer and Alexander 1949), and void ratio. To identify the significance of differences among variable and treatment means we used analysis of variance with a factorial experiment using community type, depth, and stress level as the factors. For the significant ($p<05$) main or simple effects, we compared among community types and depths with least significant difference.

RESULTS

Soil loss differed by plant community, depth, and tractive stress level as indicated by analysis of variance ($p<.001$). However the interaction between community type and depth was also significant ($p<.001$). Therefore, we assessed differences among community types at each depth and differences among depths within each community type using multiple comparison of means (Table 2).

At the high stress level, all subsurface sample material eroded. This we predicted from the high erosion rate of subsurface samples subjected to medium stress. Overall, soil loss varied significantly at different stress levels (low=24.7 g, medium=70.6 g, high=91.9 g) even though measured subsurface erosion was less than it might have been because erosion stopped when the sample material was gone at high stress.

Table 2. Mean values of root and soil properties among three meadow communities.

Variable		Plant community			
		Dry	Semi-wet	Wet	P
Soil loss, grams	surface	45.4^{bA}	7.5^{aA}	4.3^{aA}	<.001
	subsurface	$117.^{aB}$	$93.^{aB}$	$107.^{aB}$	.10
Root length density:					
Total RLD mm mm^{-3}	surface	1.5^{aB}	2.5^{bB}	4.9^{cB}	<.001
	subsurface	$.2^{aA}$	$.4^{bA}$	$.6^{cA}$	<.001
Fine RLD, mm mm^{-3}	surface	$.003^{aB}$	$.01^{bB}$	$.01^{bB}$	<.001
	subsurface	$.002^{aA}$	$.003^{abA}$	$.003^{bA}$	.05
Coarse RLD mm mm^{-3}	surface	$.0007^{aA}$	$.0014^{bA}$	$.002^{cA}$	.004
	subsurface	$.0005^{aA}$	$.0015^{abA}$	$.002^{bA}$	.004
Rhizomes, mm mm^{-3}	surface	$.0007^{aB}$	$.0017^{bB}$	$.002^{bB}$	.003
	subsurface	$.000003^{aA}$	$.00001^{aA}$	$.000^{aA}$	.35
Dry root biomass, mg mm^{-3}	surface	$.01^{aB}$	$.03^{bB}$	$.04^{cB}$	<.001
	subsurface	$.0016^{aA}$	$.0026^{abA}$	$.004^{bA}$	<.001
Void ratio	surface	1.4^{aB}	2.4^{bB}	3.3^{cB}	<.001
	subsurface	1.2^{aA}	1.3^{aA}	1.2^{aA}	.32
Organic carbon, %	surface	4.1^{aB}	6.7^{bB}	7.3^{bB}	.007
	subsurface	3.0^{aA}	3.8^{abA}	3.6^{bA}	.03
Clay content, %	surface	21.3^{aB}	23.2^{aA}	19.8^{aA}	.29
	subsurface	16.4^{aA}	21.7^{bA}	18.6^{aA}	.001
Mean grain size, Phi	surface	4.5^{aB}	5.2^{aA}	5.0^{aB}	.57
	subsurface	4.3^{aA}	4.9^{aA}	4.7^{aA}	.06
Dispersion ratio	surface	8.1^{aA}	12.9^{bA}	16.3^{bA}	.006
	subsurface	19.5^{aB}	20.1^{aB}	23.0^{aB}	.47
Plasticity index	surface	8.3^{bA}	5.6^{bA}	4.3^{bA}	.06
	subsurface	8.3^{aA}	9.9^{aB}	10.6^{aB}	.08

Means within rows followed by the same small letter and means between depths followed by the same capital letter do not differ significantly ($P<.05$) tested by analysis of variance and least significant difference.

At the surface, all root and soil properties except clay and mean particle size showed significant variation among plant communities (Table 2). Five of the subsurface variables did not differ among community types. Total, fine, and coarse RLD as well as root biomass, and organic carbon, were lower in the dry community at both depths. Amounts and differences were smaller in subsurface samples. Dry community surface and all subsurface samples often eroded substantially at higher tractive stress. Coarse root and rhizome length density, root biomass, and void ratio were higher and in the surface soil of the wet community. These samples often eroded negligibly. Although many soil properties differed among community types, the differences were small for those not strongly affected by vegetation.

Several surface soil (void ratio and organic carbon) and root (total, fine, coarse, and rhizome RLD, and root mass) properties were lower for the dry community type and might explain its greater erodibility. All but coarse RLD, which did not differ by depth were also lower for subsurface samples and might explain their greater erodibility. Although surface soil dispersion ratio was lower for the dry community type, the fact that it was higher in subsurface soils may also explain their greater erodibility. The low variation in soil properties reflected homogeneity within the study area except for soil water and vegetation differences.

DISCUSSION AND CONCLUSIONS

Intentionally eroding replications of massive streambanks was not possible. This study used a flume to control water flows and therefore tractive stresses. Samples were small and extracted from meadows rather than remaining a part of a natural, or excavated channel with intact sod. Presumably both the small size and any disturbance would increase soil loss. We chose this approach to test sufficient samples for statistical analysis of a range of environmental and experimental conditions.

The smooth walls of the flume may cause the velocity gradient in the near-wall or near-sample region to exceed normal for streams. However, streams of this size, shape, slope and velocity come closer to the flume conditions than most. As on most streambanks, a local discontinuity caused by sample erosion probably caused eddy currents of unknown importance. This effect may be greater for erodible versus nonerodible samples.

Experiments varying tractive stress along various existing banks would avoid complications caused by small samples, extraction disturbance, and flume effects. However, such work would be difficult to control. Careful measurement (Zonge and Swanson 1994) of natural experiments in channel shape and vegetation monitoring will avoid these difficulties, but add much natural variation and confusion. Even so, knowledge so gained will continue to support enlightened management (Swanson and Myers 1994).

This study suggests a high level of erosion resistance provided by the roots of riparian vegetation for stability of E4 and potentially other stream types. Factors that weaken this vegetation or decrease root-length density could destabilize such stream banks. Such factors could include desiccation (represented by the differences between wet and dry meadows), herbicidal treatment, excessive grazing, or an important change in soil nutrient status. Bank changes also degrade salmonid habitat such as deep channels and pools bounded by steep or overhanging banks (Platts 1991).

The differences between depths observed for root density and soil erodibility suggest part of the cause for overhanging banks along small streams bounded by wet meadow vegetation. The differences in erosion among levels of tractive stress suggest the complementary cause. Although in most streams, differences in cumulative tractive stress may be caused more by differences in the duration of inundation.

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