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SECTION 3

EROSION AND ITS CONTROL

THE ARTICULATED CONCRETE MATTRESS: HISTORY AND USE

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ABSTRACT

Bank erosion poses a serious threat to both flood control and navigation on the lower Mississippi River. Since being assigned responsibility for "taming" this river in 1850 (Humphreys and Abbot 1876), the Corps of Engineers has used a variety of materials and techniques to protect the banks from erosion. Years of trial and error have resulted in a cost effective technique that consists of a mattress of concrete blocks held together with corrosion-resistant wire fabric and cable. The assembly is known as an articulated concrete mattress (ACM).

The area of coverage of the ACM basic component, the square, has been 100 sq ft since this method was first used in 1915 (Robinson and Ethridge 1985). However, many changes have taken place in the design, materials, and placement practices. A square has nominal dimensions of 4 by 25 ft. Presently, it consists of 16 blocks, each 46 1/4 by 17 3/4 by 3 in., within which a wire fabric is embedded. The wire fabric acts as a flexible skeleton for the square, allowing the blocks to adjust (i.e., articulate) to bank geometry. Squares are placed on the sloping deck of an assembly barge and joined to adjacent squares and steel launching cables with corrosion-resistant wire to form a mattress. The launching cables are used to lower the mattress to the riverbed as the barge is moved away from the bank.

Documentation of the many changes in ACM design, materials, and placement practices is sparse, and much of the corporate memory has been lost. The objective of this paper and its parent study (Collins and Dardeau, in preparation) is to document the evolution of ACM using not only published data but also extant file and archival information plus that gained from interviews with knowledgeable individuals.

INTRODUCTION

Background

Since being assigned responsibility for "taming" the lower Mississippi River in 1850, the U.S. Army Corps of Engineers has used various materials and techniques to protect the banks from the erosive action of the river. The most common method used in the early days was the willow mattress, which was an effective but very labor-intensive means of addressing the problem. Other experimental methods have been tried, but none has proved to be as effective as ACM, which was introduced in 1915. Since 1915, many changes have taken place in design, materials and placement practices.

Purpose and Scope

This paper and its parent study are the results of an effort to examine and document the evolution and use of ACM on the lower Mississippi River by the Corps. Among the aspects considered are block and square design; concrete;

casting; and sinking. Full use is made of published data plus information available from extant file and archival sources, and interviews with knowledgeable individuals.

BLOCK AND SQUARE DESIGN

The basic unit of ACM coverage is the square, a section of concrete blocks designed to cover a 4- by 25-ft area, within which a wire fabric has been embedded (Figure 1). The number, shape and, size of the blocks has varied

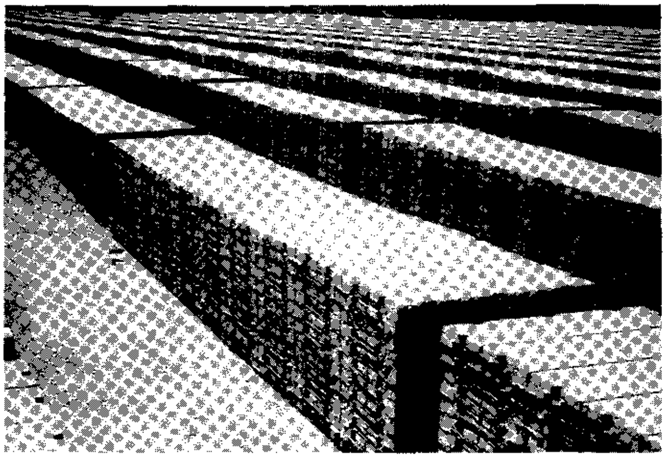

Figure 1. Thirteen-square stacks ready for loading

over the years. What began as a concept of MAJ E. M. Markham in 1914 to develop a concrete mat to replace the willow mattress has evolved into today's ACM. The first experimental reinforced concrete mattresses were placed in 1915 (Mississippi River Commission 1922). By 1930, Vicksburg and New Orleans Districts were using 25-block squares consisting of blocks 46 by 11 by 3 in. in size (Gilliand 1930). In 1932, there were three separate "types" of ACM squares, as follows (Dobson 1932):

Type	No. of Blocks	Dimensions, in.
Memphis	20	46 1/4 x 14 x 3
Vicksburg	25	44 x 11 x 3
New Orleans	18	44 x 16 x 3

The various ACM configurations necessitated unique casting, handling, and placement requirements. Later in 1932, the 20-block Memphis mat was adopted as a standard by the Mississippi River Commission (Elliott 1932, Jackson 1935). In 1969, the Memphis District began experimentation on the use of a 16-block square, with the first section being sunk in September 1970 at Flower Lake Revetment (mile 666) (Gray 1970).

In 1984, the 16-block array was adopted as standard with the following dimensions: 46 1/4 by 17 3/4 by 3 in. Reducing the number of blocks in a square from 20 to 16 decreased the number of side ties by four, saving both time and money. The newer design had fewer openings, which resulted in greater riverbank coverage and less opportunity for the leaching of materials beneath the in-place revetment (Robinson and Ethridge 1985).

CONCRETE

According to Vinzant (1930), prior to 1924, the most economical mix was considered to be that "which would give the concrete just enough strength to sink it in average current without excessive breakage after a curing period of two or three weeks." An approximate mix of 1:2:3 (i.e., cement:sand:gravel) was used.

Over the years, the acceptance of concrete for mattress has been judged on the basis of two factors, mix and strength. Until 1966, emphasis was placed on strength. The Corps provided guidelines and limits on concrete mterials; each manufacturer was then responsibile for designing the specific mix(es) that would meet very stringent Corps-specified strength requirements. A shift in emphasis occurred in 1966. The manufacturers supplied the Corps with samples of their materials so that the Corps could design mixes to meet the strength requirements. It was then each manufacturer's responsibility to meet the Corps' exacting mix specifications.

In 1916, MAJ Markham noted that "the concrete has no especial beam function to perform" (Mississippi River Commission 1922). By 1929, however, the Vicksburg District implemented a requirement that a 40-in. span of concrete be able to withstand a load of 500 lb at the center of the beam at the age of 10 days (U.S. Army Engineer District, Vicksburg 1929). In the same year, the New Orleans District established a compressive strength requirement at placement of 1500 psi (U.S. Army Engineer District, New Orleans 1929). Compressive and flexural strengths were adjusted over the years, with requirements for flexural strength finally being dropped in 1986 (U.S. Army Engineer Division, Lower Mississippi Valley 1986). Presently, mix design and concrete strength are rigidly controlled through seven-day cylinder breakage tests.

CASTING

In the early days, ACM was cast on the river. The manufacturing plant consisted of a quarterboat, a gravel and sand dredge, a floating mixing barge, and a series of material and casting barges. In this procedure, six to ten casting barges were moved end to end, and the floating concrete mixing barge traveled alongside providing concrete. Approximately 150 squares were cast each day in special forms using wire reinforcement. A locomotive crane then lifted the cured squares from the casting barges to the deck of the mat-sinking unit where they were assembled into mattresses and launched into the river (Robinson and Ethridge 1985). For economic reasons, mattress transport distance was minimized. As a result, sand and gravel used was "run of the bar" secured from deposits near the mat-laying site (Vinzant 1930).

As the volume of work increased, the floating plant casting procedure became infeasible. Since 1938, mattress casting has been carried out on land (Figure 2) at casting sites strategically located along the river. The fields offered storage and production capacities that made possible the large quantities of ACM required to meet seasonal demands (Robinson and Ethridge 1985). Presently, mats are cast and stored at seven Government-owned,

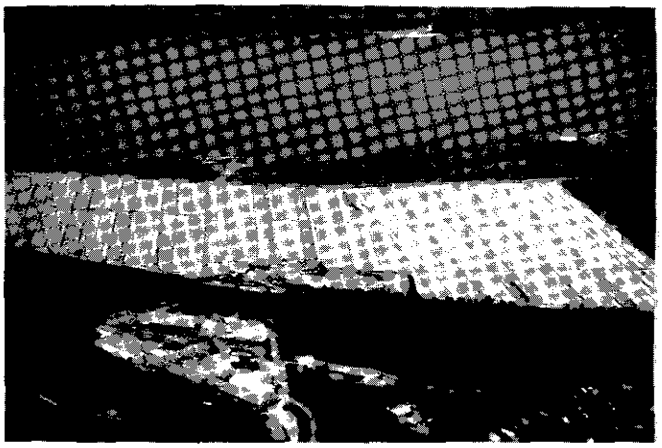

Figure 2. Casting field

contractor-operated casting fields. These fields include Gates and Richardson, TN; Helena, AR; Greenville, MS; and Delta, Vidalia, and St. Francisville, LA.

The casting of concrete mattress has become a highly automated procedure. Casting contractors have developed specialized equipment to place, strike, vibrate, and finish the concrete; to clean and lubricate forms; and to spread Kraft paper between lifts. The Corps has developed additional specialized equipment to load the squares on trucks and barges. From the 150 or so squares per day that could be cast on the floating plant 60 years ago, new technology now affords an average daily production rate of 1500 squares per day on a single casting field.

SINKING

For each sinking season (generally, the low-water season from August through November), the Corps prioritizes those reaches that require revetting. Robinson and Ethridge (1985) point out that the earliest mat-sinking plant consisted of a barge on which launching ways were constructed. A 2.5-ton locomotive crane traveled the full length of the barge lifting squares from a supply barge to the sinking plant. When a 35-square-wide mattress was assembled with hand ties and connected to galvanized cables, it was then ready for launching. Additional sections (called launches) were added so that the mattresses extended from the water's edge to "maximum depths." After the cables were cut, the next section was placed upstream with a 10-ft overlap. This process continued until the eroding bank was revetted.

For a time, the Memphis, Vicksburg, and New Orleans Districts each had their own sinking plants and revetted their own eroding reaches. Now, however, only Memphis and Vicksburg have sinking units (Figure 3); these units are essentially identical in operation, the main difference being that Memphis launches a 140-ft wide mattress, while the width of Vicksburg's mattress is 156 ft. Memphis operates one 10-hr shift daily, and Vicksburg, two 10-hr shifts, with 4 hr set aside for maintenance. Four gantry cranes have replaced

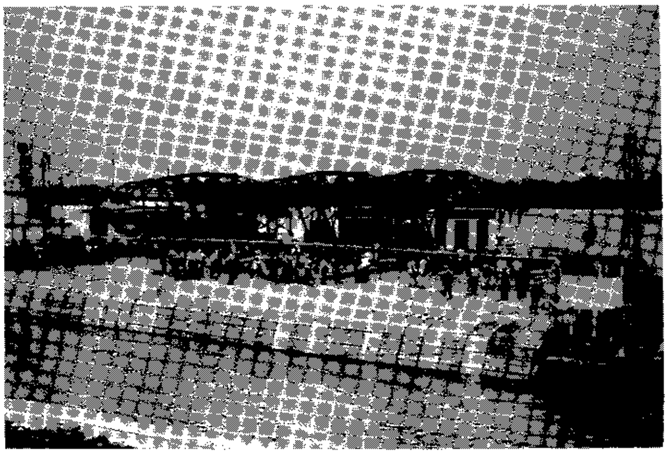

Figure 3. Sinking unit

the single locomotive crane, and automatic tying tools have resulted in an efficiency not possible with the hand tools. Normally, mattress is laid from the water's edge to the thalweg beginning at the downstream end of the eroding bank. Each succeeding upstream mattress is placed with a 5- to 15-ft overlap with the previous mattress (depending on depth at time of sinking) until the revetment is complete (Figure 4).

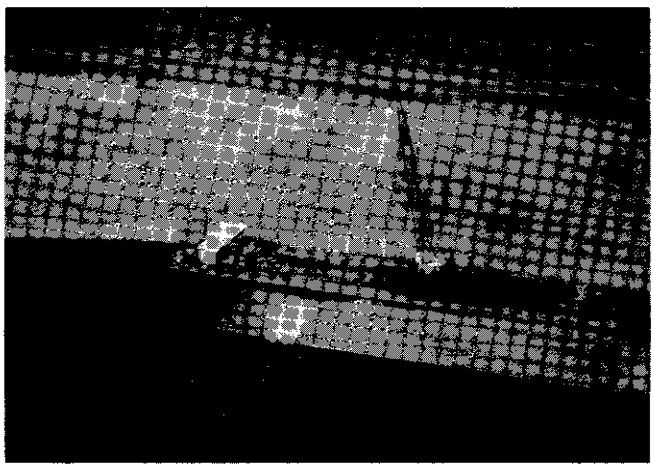

Figure 4. In-place revetment

CONCLUDING REMARKS

From its point of beginning in 1915, the ACM revetment program has grown into a large and complex operation. The Corps now has approximately 200 employees involved on a continuous basis. There are also approximately 1100 seasonal workers (400 in Memphis and 700 in Vicksburg), which includes those engaged in bank grading and mat loading. The Corps has a considerable investment in its mat-sinking plants, which includes trucks, bulldozers, graders, mattress barges, sinking units, quarterboats, crew boats, mooring barges, spar barges, towboats, etc.

An enormous growth in the ACM program has taken place over the past 75 years. In 1927, 67,000 squares were placed (DeBerard 1928); in 1989, this seasonal quantity reached 651,417 squares, an average rate of 8,042 squares per day. The cost of in-place mattress is now $150 per square, as compared with $14.65 in 1927 (DeBerard 1928). In constant dollars, however, the in-place cost has actually decreased about 55 percent. Today's mattress contains fewer blocks, less cement, less fabric wire, and fewer ties and takes fewer hours to construct; yet, it is more effective and lasts longer than the earlier ACM designs. Product cost reductions and improvements have not happened by chance, rather through the consistent test and evaluation of suggestions and recommendations made by observant Corps and contract employees.

Thus far, 937 miles of ACM have been placed along the lower Mississippi River. Approximately 148 miles of bank remain to be revetted before the completion of construction now scheduled for 2010. In addition, the Corps repairs about 8 miles of existing revetments each season. Following the conclusion of construction, the future Corps ACM program will be limited to maintenance of the existing revetments. In a sense, a status quo will have been reached but only with reference to revetted areas. The ACM itself will continue to evolve as innovative product improvements and emplacement practices continue to be made.

ACKNOWLEDGMENTS

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SYSTEMS DESIGN APPROACH IN NORTHERN MISSISSIPPI

by

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ABSTRACT

A design approach for the streams within the Demonstration Erosion Control (DEC) Project in northern Mississippi has been developed. This approach requires a detailed geomorphic study of the entire watershed, from which hydraulic and geotechnical stability criteria are developed. The criteria are used for the planning of rehabilitative structures, and predicting the channel response to the implementation of these features. Although developed specifically for the DEC watersheds, the general concepts of this approach can be applied to streams in other parts of the country.

INTRODUCTION

The U.S. Army Corps of Engineers, Vicksburg District and the Mississippi Soil Conservation Service have been involved since 1984 in the Demonstration Erosion Control (DEC) Project in selected watersheds of the Yazoo Basin in northern Mississippi. The DEC watersheds are experiencing a wide variety of problems ranging from severe channel instability to sedimentation and flooding. One of the primary objectives of the DEC project is to demonstrate how these complex and inter-related problems can be solved using a systems approach.

This paper presents the methodology used by the Vicksburg District to assess the stability of the DEC watersheds and develop plans to solve the identified problems. Since most of the DEC streams are directly or indirectly affected by existing channel improvement projects the methodology discussed herein primarily focuses on the rehabilitation of existing, unstable channel systems.

As with the explanation of any methodology, application of the procedure requires trial and error, and good judgement. The procedure presented herein should not be viewed as a sequence of independent steps since most projects require several loops back to check initial assumptions and to refine and re-examine conclusions.

Data Assemblage. The first step in the design approach is to assemble all pertinent data for the subject watershed. The types of data which are typically gathered include: stream gage data, historical mapping and surveys, aerial photography, land use data, sediment data, soil borings, geologic information, inventory of existing structural features and projects, and neighboring watershed information that might influence the subject watershed.

Channel Stability Assessment. The purpose of the channel stability assessment is to identify reaches of relative stability or instability within the channel. This requires developing an understanding of the total system dynamics on a reach basis. This understanding will then allow the engineer to discriminate between channel reaches that are degradational, aggradational, or in a state of equilibrium, and also to categorize channel banks as stable or unstable. Through study of field conditions and available data, system responses, both past and present, can be determined, and a picture of the overall system stability can be assimilated.

Various tools are available to the engineer for conducting the stability assessment. A list of those commonly used in the DEC watersheds includes: specific gage analysis; comparison of surveys (both thalweg and cross section), aerial photography or other mapping; detailed field investigations and the Channel Evolution Model (Watson et al 1988); hydrologic and hydraulic analyses (HEC-1, HEC-2); hydraulic geometry analyses; sediment transport modeling (HEC-6); regional hydraulic and geotechnical stability curves developed for the area from previous investigations; and detailed geotechnical investigations. Some or all of these tools may be utilized depending upon the level of study and project objectives.

The stability assessment of the system is accomplished by integrating the information from all available tools previously mentioned. Analysis of each individual tool will yield a verdict of aggradational, degradational, or equilibrium with respect to the channel bed, and stable or unstable with respect to the channel banks. After the individual components of the stability assessment are completed, the overall stability of the system is determined. Often individual components of the assessment may produce conflicting results. For this reason the engineer must assign a level of confidence to the various components when weighing one against the other. An expert system has recently been developed which assimilates all available information and assists the engineer in making the channel stability assessment (Peterson et al 1990).

Development of Hydraulic and Geotechnical Stability Criteria. Once the stability assessment is complete the next step is to quantify the hydraulic and geotechnical characteristics of the stable reaches. This provides the engineer with criteria to assess the remaining reaches in the watershed where there may have been insufficient data to conduct a reliable stability assessment. It also provides a quantitative basis for the planning of rehabilitative structures, and predicting the channel response to the implementation of these features.

(1) <u>Hydraulic Stability.</u> Hydraulic stability is defined as a balance between sediment transport capacity and sediment supply. There are many ways to express hydraulic stability. These range from very simple procedures to more complex modeling techniques. One simple method of quantifying the hydraulic characteristics of the stable reaches is to define the morphometric (width, depth, and slope) parameters. In the incised channels of north Mississippi channel slope is one of the best indicators of stability and can easily be measured directly from the thalweg survey. If sufficient data exists then a regional stability curve relating stable slope to drainage area can be developed. A typical slope-drainage area curve is shown in Figure 1. This curve was developed for sand bed streams in three of the DEC watersheds. With this curve the stable slope for any given reach in these watersheds can be determined. As with any empirical relationship the reliability of these curves becomes questionable when extrapolating to other watersheds, or even within the same watershed if channel characteristics are significantly different. For this reason it is advisable to develop specific curves for the watershed being studied.

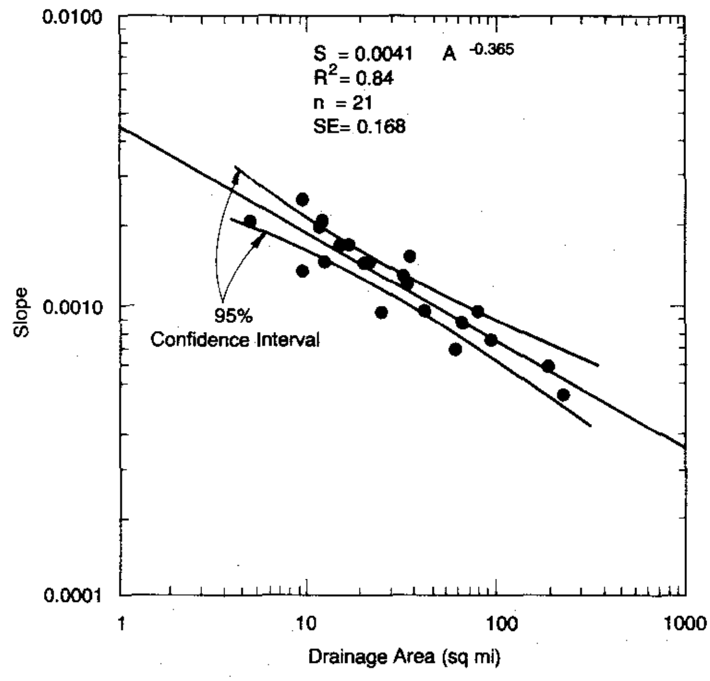

Figure 1. Typical equilibrium slope vs. drainage area relationship for north Mississippi streams

The hydraulic characteristics of the stable reaches can also be defined using more sophisticated procedures which may require some amount of modeling effort (HEC-1 and HEC-2). Parameters such as shear stress, unit stream power, shear intensity, or others can be defined for the stable reaches. These methods address the conveyance of the channel by including slope and cross sectional characteristics (width, depth, hydraulic radius, etc.). Consequently, these parameters are better suited for sizing a channel than the slope criteria. It is important that these parameters be defined within the range of the dominant discharge which for the DEC streams has been shown to be slightly less than the 2 year recurrence interval (Water Engineering and Technology, Inc., 1987). Although these parameters can provide considerably more detail than a simple measurement of bed slope, it must be recognized that the results are based on hydrologic models that have inherent uncertainties.

In most DEC applications it is not necessary to utilize sediment transport modeling techniques to assess the channel stability since this can be accomplished more effectively using the previously mentioned procedures. However, there are some situations where sediment transport modeling is required to predict the long and short term sedimentation trends resulting from implementation of project features. In these instances the results of the stability assessment are used to calibrate and interpret results from the sediment transport model.

(2) <u>Geotechnical Stability.</u> Thorne (1988) has investigated and tested several procedures for determining the geotechnical stability of stream banks within the DEC watersheds. In the DEC watersheds severe system-wide mass failures of the banks can exist due to extensive bed degradation. Unlike local bendway erosion which is easily treated with conventional bank stabilization techniques, system-wide instability may exist along both banks of the channel and requires a more comprehensive treatment plan which usually includes stabilization of the bed. Geotechnical stability criteria allows the engineer to assess the existing stability of the banks, recognize potential instability should future degradation occur, and to predict the response to construction of rehabilitation structures.

In the DEC streams the bank stability is usually assessed on a reach average rather than a site specific basis. The geotechnical stability criteria are generally related to a critical bank height and bank angle which will depend on the geotechnical properties of the bank materials. Osman and Thorne (1988) developed stability curves for the DEC streams based on earlier studies of bank stability of incised north Mississippi streams (Thorne et al. 1981). These curves may be altered for each stream based on the bank material characteristics. The geotechnical stability of four streams within the Hickahala Creek watershed is illustrated in Figure 2. Banks that locate in the unstable zone might be expected to fail at any time, while banks plotting in the stable zone should remain stable under almost all situations. Banks in the unreliable zone may appear stable for extended periods but may fail under worst-case conditions. With these curves the engineer can determine the stability of the banks under existing as well as future conditions.

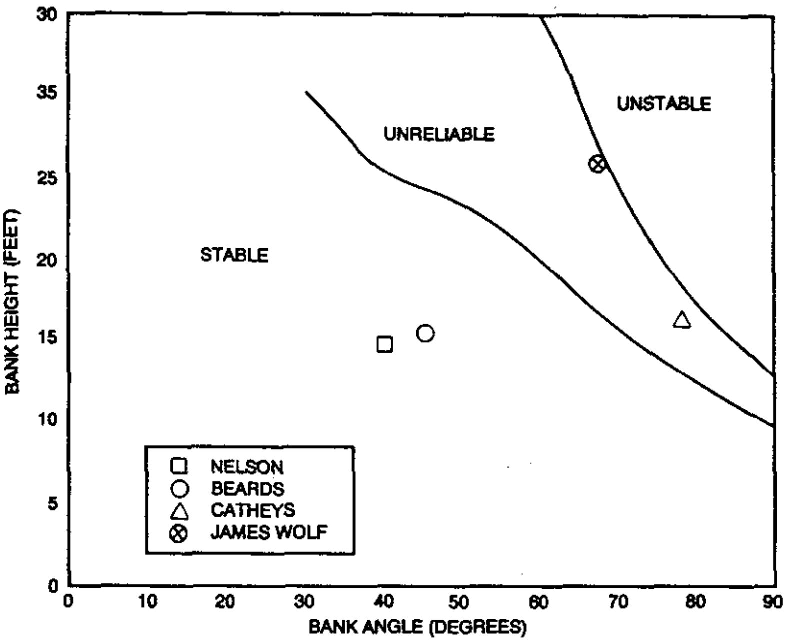

Figure 2. Geotechnical stability curves for Hickahala Creek watershed, MS

(3) <u>Dimensionless Stability Numbers.</u> Assessing channels with respect to both the hydraulic and geotechnical stability criteria provides the basis for design of rehabilitative structures. It is sometimes helpful to express the hydraulic and geotechnical stability in terms of two dimensionless stability numbers: Ng which is the geotechnical stability number, and Nh which is the hydraulic stability number.

Ng provides a measure of the system-wide geotechnical stability and is defined as the ratio between the existing bank height and the critical bank height at a given bank angle. Bank stability is attained when Ng is less than 1.0. Therefore, the reaches plotting in the stable zone of Figure 2 would have an Ng less than 1.0, while those plotting in either the unreliable or unstable zones would have an Ng greater than 1.0.

The hydraulic stability number, Nh, is defined as the ratio of the sediment transport capacity to the sediment supply. This ratio may be expressed by computed sediment transport, shear stress, channel slope, or other energy relationships. Hydraulic stability is attained when Nh approximates 1.0. When Nh is much greater than or less than 1.0, then the channel will degrade or aggrade, respectively. Nh can be quantified by any of the hydraulic procedures previously mentioned depending upon the situation and experience of the engineer. For instance, assume that the stability assessment shows that hydraulic stability is attained when the bed slope is 0.0015. Using this as the hydraulic stability criteria a reach with a similar drainage area having a bed slope of 0.0025 would have an Nh of 1.67 (0.0025/0.0015), and therefore would be considered degradational.

The Ng/Nh diagram (Figure 3) is a very useful tool for illustrating the existing hydraulic and geotechnical stability of a channel and how this stability may be altered if further degradation or aggradation should occur, or if improvement features such as grade control structures are implemented.

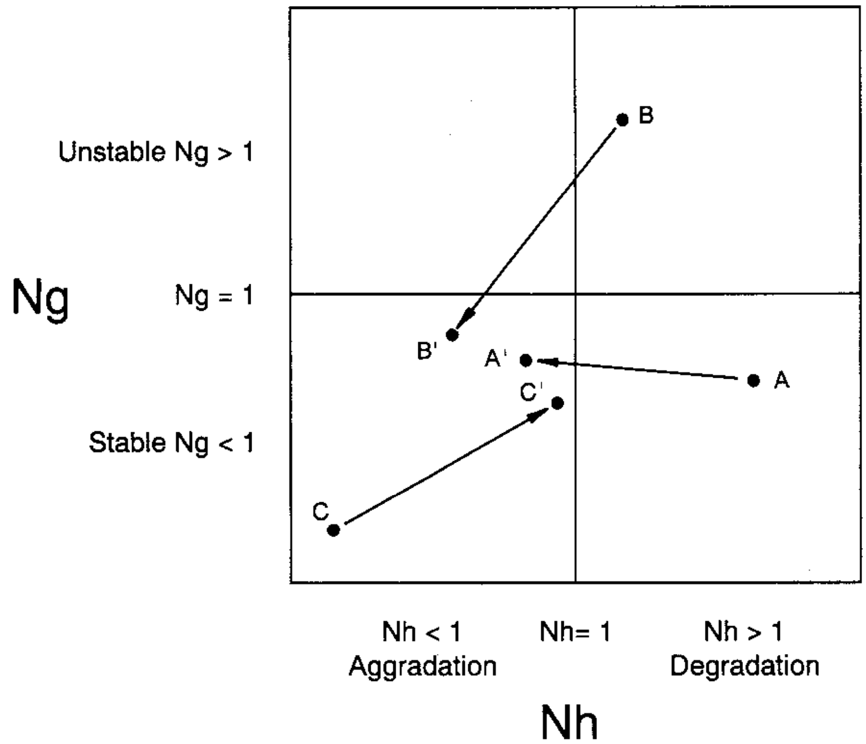

Figure 3. Dimensionless Stability Number diagram

APPLICATION

Rehabilitation of incised streams in the DEC watersheds usually involves the implementation of grade control structures. The siting (location and elevation) of these structures is accomplished by satisfying both the hydraulic (Nh) and the geotechnical (Ng) requirements for stability. This is illustrated in the following scenarios.

In the first situation the hydraulic stability analysis indicates that the reach is degradational (Nh greater than 1.0). The geotechnical stability analysis reveals that the banks are stable (Ng less than 1.0); therefore system-wide bank instability does not yet exist in this reach. On the Ng/Nh diagram this reach would plot in the lower right quadrant and is depicted by point A in Figure 3. In this situation the preferred solution would be to correct the hydraulic instability by constructing grade control structures. By stabilizing the bed the existing bank stability would also be conserved by preventing the bank heights from increasing beyond the critical condition. This would effectively move the reach from point A to A′ on the Ng/Nh diagram (Figure 3).

If in the previous example grade control was not implemented, then the bed would continue to degrade and ultimately result in severe bank instability as Ng became greater than 1.0. This reach would then be represented by point B in Figure 3. In this situation stabilization of the bed at or near its existing elevation would satisfy the hydraulic requirement for stability, but severe bank instability could still exist if Ng is not reduced. This could be accomplished by raising the bed of the channel with a system of grade control structures. These structures would be designed to induce sediment deposition in the bed and thereby reduce the bank heights to an acceptable level (Ng less than 1.0). By reducing both Ng and Nh the reach would be moved from point B to B′ in Figure 3.

The design criteria can also be applied to the design of new flood control channels as well as to the rehabilitation of existing ones. For example, consider an aggrading channel with a severe flooding problem represented by point C on the Ng/Nh diagram (Figure 3). In this case the hydraulic stability criteria is defined by a shear stress value of 0.5 lbs/ft^2 at the dominant discharge. Therefore the new channel should be designed by adjusting the bed slope and cross sectional area to produce a shear stress of 0.5 lbs/ft^2. Using the geotechnical stability criteria the channel bank slopes would be designed to ensure stability. This design would shift the channel from point C to C′ on the Ng/Nh diagram (Figure 3). This should produce a channel design that minimizes instability and long term maintenance problems.

SUMMARY

The described systems design approach has been utilized in developing plans for incised channels in the DEC watersheds. This is not a "black box" approach to a unique solution, but rather it requires the engineer to recognize the interrelationships between channel morphology and hydraulic and geotechnical stability. This allows the engineer to determine the critical needs and to develop a systematic basis for design of the plan. The end result of such an effort is a plan that encompasses all aspects of the existing watershed and describes the measures that will best serve and compliment the entire system.

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RED RIVER WATERWAY SEDIMENTATION

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ABSTRACT

The Red River is a large alluvial river system with the lower 280 miles currently being developed for navigation. This development includes the construction of locks and dams as well as various channel realignment and bank stabilization works. The Red River is a heavily sediment laden stream with one of the highest sediment concentrations of all major navigable rivers within the United States. Given the high sediment load on the Red River, the design, construction, and operation of project features can have a dramatic impact on system and on local sedimentation tendencies.

This paper will identify the sediment characteristics of the lower Red River and investigate changes in or the severity of sediment trends which can be attributed to the construction and subsequent operation of waterway project structures. The construction of bank stabilization measures, channel contraction works, and bendway cutoffs can all alter pre-project sedimentation patterns. However, this paper will concentrate primarily on the assessment of the local and the system sedimentation impacts resulting from the construction and operation of Locks and Dams No. 1 and No. 2 on Red River Waterway and on the analysis of the effectiveness of the sediment management features being utilized at these structures. The paper will also concentrate on an analysis of the design of the sediment management features and operational procedures proposed for Locks and Dams No. 3, 4, and 5. The identification of sediment problems and the development of ways in which to most efficiently manage sediment deposition at each of these locks and dams is crucial in the effective maintenance of the waterway project.

INTRODUCTION

Red River Waterway Project

The Red River Waterway Project (Figure 1) was authorized in 1968 with the primary purpose of providing a 9-foot deep by 200-foot wide navigation channel from the Mississippi River to Shreveport, Louisiana. The construction of a system of five locks and dams in conjunction with an intense program of channel realignment and bank stabilization work is required to provide and maintain the authorized project channel. At present, Locks and Dams No. 1 and No. 2 are complete (fall of 1984 and fall of 1987, respectively) and in operation. Lock and Dam No. 3 is under construction with a scheduled completion during the spring of 1992. Locks and Dams No. 4 and 5 are in the detail design phase with initiation of construction dependent upon available funding. The channel realignment and bank stabilization program has progressed with practically all work complete and developing through Pool 3 with work continuing in Pools 4 and 5.

General Sediment Conditions

The Red River is a heavily sediment laden stream with average annual suspended sediment loads of approximately 32 million tons at Shreveport, Louisiana and

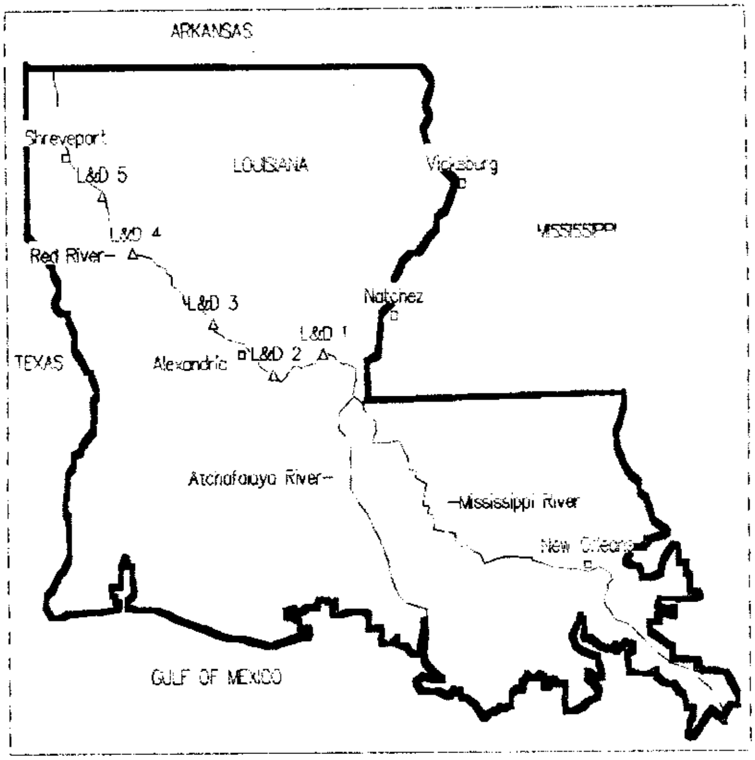

Figure 1. Red River Waterway Location Map

37 million tons at Alexandria, Louisiana. This suspended sediment load consists generally of 25% fine sand and 75% silt. Previous studies have indicated that the bed load on the lower Red River is less than 10% of the total load. The bed is primarily composed of fine to medium sand with a grain size distribution at Alexandria of approximately 2% coarse sand, 18% medium sand, 65% fine sand, and 15% very fine sand. At Shreveport, the grain size distribution includes approximately 5% coarse sand, 52% medium sand, 39% fine sand, and 4% very fine sand. The banks of the lower Red River consists primarily of fine sand and silt with very little clay. The sediment contribution from the tributaries is minimal with the majority of the transported sediment coming from the erosion of unrevetted banks, especially upstream of Shreveport.

LOCK AND DAM NO. 1

Structure Description

Lock and Dam No. 1 is located in east central Louisiana at pre-project (1967) River Mile 45. The structure consists of a 84-foot by 685-foot (usable length) lock and a gated dam. The dam consists of 11 60-foot wide tainter gates. Separation is provided between the lock and gated dam to provide structural stability for the lock. The structure is designed to pass the project design flow through the gates without an overflow section and with a maximum swellhead of 1 foot. Lock and Dam No. 1 holds a permanent minimum pool elevation of 40.0 feet, NGVD with a minimum tailwater of 4.0 feet, NGVD. The tailwater at Lock and Dam No. 1 is controlled by backwater from the

Mississippi River. Therefore, tailwater stages at Lock and Dam No. 1 can vary as much as 55+ feet. Due to this extreme variation in tailwater stages, the structure includes floating guidewalls. The minimum pool elevation of 40.0 feet, NGVD is frequently exceeded with Pool 1 experiencing open river conditions annually.

<u>Sediment Problems</u>

Shortly after Lock and Dam No. 1 was opened in the fall of 1984, normal high water was experienced on the Red River. During the highwater period when the sediment load transported by the river is highest, Lock and Dam No. 1 experienced significant sediment deposition in four primary locations. These locations as identified on Figure 2 are (1) in the upstream lock approach channel, (2) along the riverside lockwall, (3) in the downstream lock approach channel, and (4) in the lock chamber.

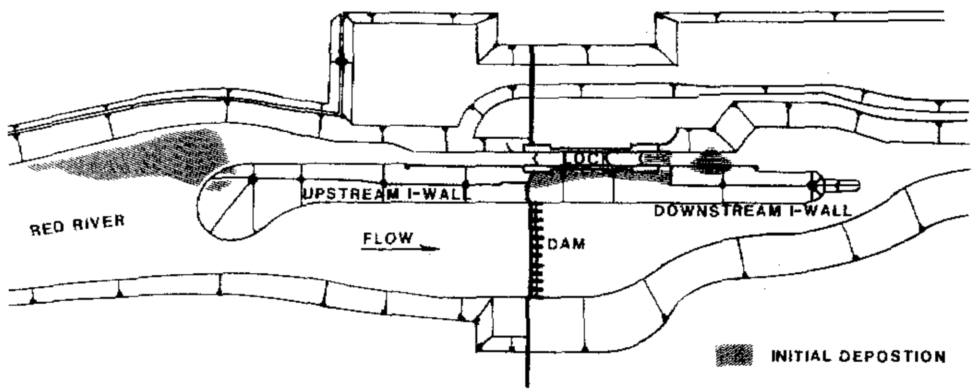

Figure 2. Initial Deposition Problem Areas at Lock and Dam No. 1

The eddy formation and slack water in the upstream lock approach channel created by the designed I-wall separation between the lock approach and dam inlet channels resulted in sediment deposition that restricted navigation as the highwater receded. The structural separation between the lock and gated dam provided a slack water area along the riverside lockwall. During the 1984 - 1985 highwater season, as much as 16 feet of sediment deposited along the lockwall. This deposition could have compromised the structural integrity of the lock since the lockwall was designed for a maximum of 2.5 feet of sediment load. During this initial high water season, the downstream lock approach channel experienced as much as 20 feet of sediment deposition along the floating guidewall and 8 to 10 feet of deposition in the immediate vicinity of the lower miter gates. Sediment deposition in the downstream lock approach channel was the result of a large eddy that formed when the I-wall separating the lock approach channel and the dam outlet channel overtopped. Continued operation of the lock during the high water season resulted in 8 to 10 feet of sediment deposition within the lock chamber and in damage to the lower miter gates. The repair of this damage required the dewatering of the lock which closed the river to navigation for approximately three months.

Analysis and Problem Solutions

During the design of Lock and Dam No. 1, model studies indicated that sediment deposition would occur in each of the four areas that experienced major deposition, but not to the extent that actually occurred. Originally, the fine grain sediment transported in suspension by the Red River was expected to remain in suspension and be carried through the waterway system. However, the areas of channel expansion and flow separation provided at Lock and Dam No. 1 created sediment deposition inducing slack water areas and eddies in both the upstream and downstream lock approach channels. Subsequent to the initial sediment problems experienced at Lock and Dam No. 1, the District initiated 2-dimensional numerical flow and sediment model studies to evaluate the problems and aid in the determination of appropriate solutions. The results of these studies indicated structural modifications were required to either reduce the amount of sediment deposition or at least relocate the deposition into more manageable portions of the channel.

In the upstream approach channel, the model studies indicated that the sediment deposition would be greatly reduced by the construction of four stone dikes extending from the right descending bank. These dikes increased velocities through the approach channel and also reduced the eddy by shifting the flow toward the left bank. Construction of these four dikes was completed in December 1985. During water year 1984 - 1985 (November 1984 - October 1985), in excess of 1,000,000 cubic yards of material was dredged from the upstream approach channel. Subsequent to construction of the dikes, following water years, as indicated by the model results, experienced as much as a 75 percent reduction in required dredging. Therefore, these dikes have performed well, resulting in savings of several million dredging dollars.

Model results of the downstream approach indicated that the sediment deposition that occurred within the approach channel as a result of overtopping the I-wall could be shifted downstream away from the lock miter gates by raising the I-wall. As a temporary solution, a 17-foot high timber wall (to 10-year frequency elevation) was constructed on top of the upstream most 400 feet of I-wall. Since completion of the timber wall in February 1986, sediment deposition has been greatly reduced at the miter gates with no additional damage to the gates or closure of the lock to river traffic due to sediment deposition. This lack of significant sediment deposition in the vicinity of the miter gates agrees favorably with the results of the model studies. However, model testing indicated that an additional 500 feet of timber wall would further reduce the sediment deposition within the structure proper. The District concluded that the prudent decision on this additional extension would be to delay construction until the effectiveness of the original 400-foot timber wall could be evaluated in the prototype. Subsequent evaluation of the original timber wall performance during a couple of high water seasons verified the validity of the model results. Therefore, the additional 500-foot extension and associated realignment of the right descending bank to improve flow patterns and navigation conditions were constructed during the summer of 1988. Since that time, the downstream lock approach channel has required periodic dredging. However, sedimentation problems have not developed in the vicinity of the miter gates.

Structural alternative features are being investigated to alleviate the sediment deposition along the riverside lockwall. These features include the construction of some type of wall located on the riverside of and aligned parallel to the lockwall to provide a barrier that prevents sediment from reaching the lockwall. At this time, various type walls are being evaluated in order to determine the best alternative design. A stability analysis of the lockwall indicated that continued removal of this sediment by mechanical means provided an acceptable short term solution to this sediment deposition problem as long as the sediment is removed expeditiously while stages are still high.

LOCK AND DAM NO. 2

Structure Description

Lock and Dam No. 2 is located at pre-project River Mile 88, approximately 14 miles downstream of Alexandria. The structure consists of an 84-foot by 685-foot (usable length) lock and a gated dam with a 250 foot uncontrolled overflow section. The gated dam consists of 5 60-foot wide tainter gates. Unlike Lock and Dam No. 1, Lock and Dam No. 2 includes fixed guidewalls. Pool 2 was initially raised during the fall of 1987. At that time, the structure held an interim pool elevation of 58 feet, NGVD to allow for continued development of channel realignment and bank stabilization works. Pool 2 was raised to its permanent minimum pool elevation of 64 feet, NGVD during February 1989. This minimum pool level of 64 feet, NGVD is exceeded annually at the lock and dam, thus resulting in open river conditions.

Sediment Control Features

As a result of the sediment problems experienced at Lock and Dam No. 1, the District anticipated some sediment deposition at Lock and Dam No. 2. However, several design features at Lock and Dam No. 2 which differ from Lock and Dam No. 1 indicated a reduction in deposition problems could be expected (Figure 3). These features included a cross section at the structure more representative of the natural river section (5 tainter gates versus 11 at Lock and Dam No. 1), no separation between the lock and dam, and fixed guidewalls instead of floating guidewalls. However, the potential for deposition within the approach channels remained. As a result of this concern, both physical and numerical model studies were initiated to help identify potential problems and the impact structural modifications could have on reducing the severity of sediment deposition.

Results of the model studies indicated that several structural modifications were required to reduce sediment deposition at Lock and Dam No. 2. These modifications include (1) a stone sediment control dike extending downstream from the riverside lockwall, (2) three reverse angle dikes extending from the right descending bank immediately downstream of the dam, and (3) narrowing of the approach channels in order to increase velocities. Since the downstream guidewall is located on the landside of the lock, the sediment control dike was constructed to prevent the flow in the dam outlet channel from breaking over into the downstream lock approach channel. This flow situation would have resulted in the formation of a sediment inducing eddy in the vicinity of the miter gates. This dike extends 700 feet downstream from the lock with the upper 300 feet at a crest elevation equal to the elevation of the top of the lock walls (10-year frequency elevation plus 2 feet). The lower 400 feet of the dike was constructed to overtop periodically in order to allow relatively sediment free flow to enter the lower lock approach channel and thereby re-suspend and transport deposited sediment. Model studies indicated that the three reverse angle dikes would further reduce the tendency for sediment deposition in the lock approach channel by inducing the sediment to the opposite side of the channel.

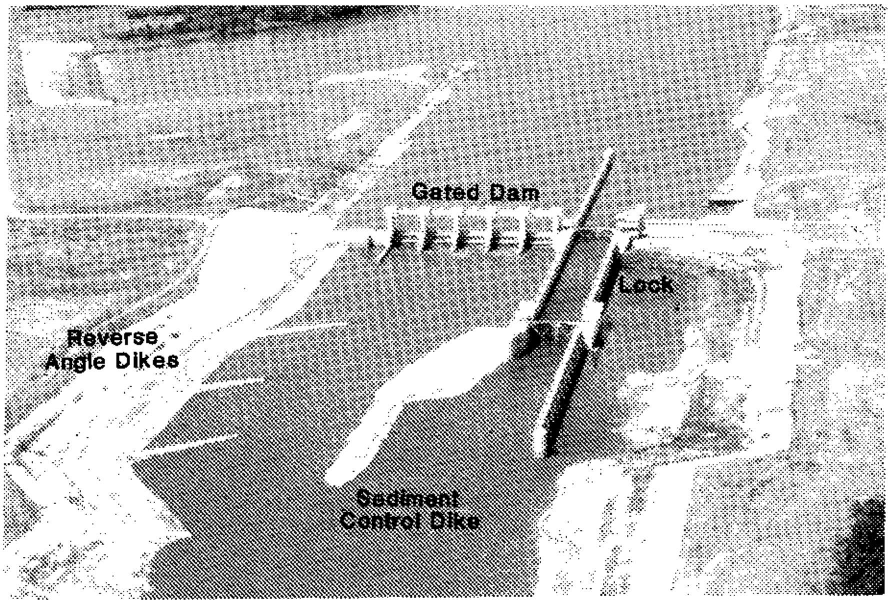

Figure 3. Lock and Dam No. 2

Sediment Control Performance

Immediately subsequent to opening Lock and Dam No. 2 in November 1987, sediment deposition occurred in the vicinity of the upstream miter gates and the downstream lock approach channel. The sediment deposition that occurred downstream was due to overtopping of the sediment control dike. Lock and Dam No. 2 was opened prior to the completion of the sediment control dike to final grade. Unfortunately, unusually high water was experienced on the Red River during late 1987 before the sediment control dike was completed. Since that high water event, the sediment control dike has been completed and while some dredging has been required in the lower approach since, it has been limited to an acceptable level. The sediment deposition experienced at the upper miter gates has resulted in difficulty in operating the gates. The sediment entering the upstream lock approach channel that doesn't exit into the dam inlet channel through the ported guidewall gets caught in an eddy which forms at the miter gates and deposits. The District, after much investigation and study, designed and installed a high velocity jet system to re-suspend deposited sediment at the miter gates. The lock is operated so that this re-suspended sediment is transported into the lock chamber and out through the emptying system. This system since its installation in July 1988, has been very successful in removing sediment deposition at the upstream miter gates, thus keeping the lock operable. This jet system is serving as a pilot system to determine if more elaborate, permanent systems could be utilized to provide long term sediment management at Locks and Dams No. 3, 4, and 5 on the Red River. This jet system has provided successful sediment removal at the upper miter gates for a very minimal cost. The first year total costs for the jet system included less than $2200 for labor and materials associated with initial installation and approximately $350 for operation.

LOCKS AND DAMS NO. 3, 4, AND 5

Since the sediment deposition problems have occurred at Locks and Dams No. 1 and 2, detailed investigation of the sediment management features of Locks and Dams No. 3, 4, and 5 have been initiated. These investigations include the use of both physical and numerical model studies to aid in the design of the most effective sediment management structural features as well as in the identification of the most effective operational procedures for the reduction of sediment deposition both at the structures and within the pools.

Pool Sedimentation Potential

Pools 1 and 2 hold low frequency minimum pool levels which result in open river conditions on an annual basis. That is, the discharge is sufficiently high that the tainter gates in the fully opened position can no longer pass enough flow to maintain minimum pool stage. Pools 3, 4, and 5 will hold a much higher frequency minimum pool level, each in excess of the 15-year frequency elevation. The potential for sediment deposition within a pool is obviously reduced once the flow capacity of the structure is exceeded and open river conditions develop. To date, full 9-foot navigation is provided to Alexandria, Louisiana. This channel includes all of Pool 1 and approximately one third of Pool 2. Since Locks and Dams No. 1 and 2 have been in operation, only limited dredging has been required within the pools (excluding that at the locks and dams) to maintain the authorized channel dimensions. However, with much less frequent periods of open river for Pools 3, 4, and 5, additional sediment deposition is expected within these three pools. Hinge pool operation has been approved for Pool 3 and studies are being conducted to determine the value of hinge pool operation for Pools 4 and 5. Hinge pool operation involves opening of the tainter gates to draw the lower end of the pool below minimum pool level on the rising limb of the hydrograph in order to increase channel slope. This increase in channel slope results in increased channel velocities which in turn enhance sediment transport. Until these hinge pool studies are complete, the full extent of the various impacts of hinge pool operation for Pools 4 and 5 will not be known. However, detailed design of Locks and Dams No. 4 and 5 are proceeding to allow for hinge pool operation.

Structural Sediment Control Features

The design of Locks and Dams No. 3, 4, and 5 include structural features aimed at reducing similar sediment deposition to that experienced at Locks and Dams No. 1 and 2. These features include (1) downstream guidewall moved from landside of the lock to the riverside, (2) permanent, more elaborate jet system for either the upper miter gates or both sets of miter gates, (3) uncontrolled pipes provided through the downstream lock sill, and (4) channel cross section which like the one provided at Lock and Dam No. 2 closely approximates the natural river section. Relocating the downstream guidewall to the riverside of the lock will provide a barrier between the lock approach channel and the dam outlet channel and thus prevent the sediment transported through the structure from entering the lock approach channel. Studies are now under way to design the jet systems for reduction of sediment deposition at the miter gates. Various alternatives are being studied which vary from a simple system like the one being used at Lock and Dam No. 2 to very elaborate systems which provide jets on both the upstream and downstream sides of both the upper and lower sets of miter gates. The uncontrolled pipes through the downstream sill of the lock are intended to utilize the head between the upper and lower pools to reduce sediment deposition against the downstream face of the miter gates. Each of these features will be incorporated into the design of Locks and Dams No. 3, 4, and 5 in order to help reduce sediment deposition at each structure.

SUMMARY

Due to the nature of the sediment carried by the Red River, both Locks and Dams No. 1 and 2 have experienced varying degrees of troublesome sediment deposition since their opening to navigation. In order to reduce costly removal of this deposition and in order to insure year-round use of the locks, sediment management modifications have been developed and constructed at these locks and dams. The District has utilized lessons learned at Locks and Dams No. 1 and 2 and both physical and numerical models to develop sediment management features that are being incorporated into the design of Locks and Dams No. 3, 4, and 5. Through the use of these sediment management features as well as through operational procedures such as hinge pool operation, the Red River Waterway will provide reliable navigation from the Mississippi River to Shreveport, Louisiana for decades to come.

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MISSISSIPPI RIVER DIVERSIONS FOR MARSH CREATION

by Cecil W. Soileau, Chief, Hydraulics and Hydrologic Branch, US Army Corps of Engineers, New Orleans District.

ABSTRACT

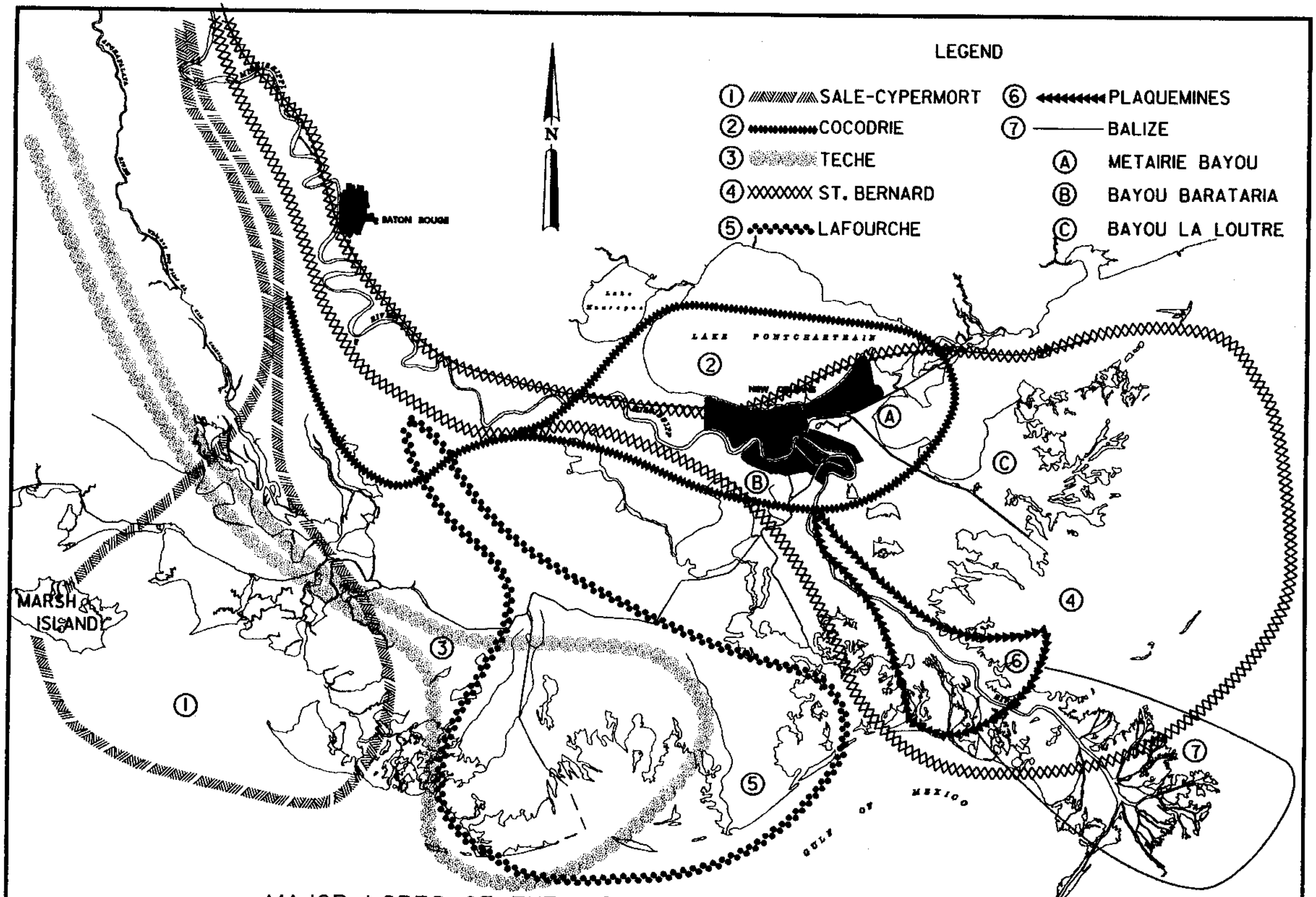
Large scale, uncontrolled sediment diversions from the Mississippi River are proposed by the Corps of Engineers as a means of creating new marsh and reducing the loss of wetlands. These uncontrolled diversions are intended to simulate the natural bifurcations and crevasses of a delta system and would consist of a cut in the bank and a conveyance channel to convey sediment laden waters to shallow open water areas. Various combinations of design discharge and sill elevations were evaluated. In order to enhance the development of marsh within the receiving waters, earthen dikes would be constructed to assist in retaining discharged sediment. To assist in extending delta growth, additional bifurcations would be dredged in the new delta.

INTRODUCTION

The loss of Louisiana's coastal wetlands is a national crisis. Forty percent of the Nation's coastal wetlands lie in Louisiana; but, Louisiana experiences eighty percent of the nation's annual loss. The magnitude of both current and future economic and environmental losses is great.

Louisiana's coastal wetlands were formed by the Mississippi River. Over the past 6000 to 8000 years, the Mississippi River has shifted its course several times in its search for a more direct route to the Gulf of Mexico. This natural process resulted in the development of the major delta complexes as sediments carried by the river were deposited. (Figure 1).

Four Mississippi River subdeltas have been active since the first accurate survey of the Mississippi River delta in 1838. Three of these subdeltas lie above Head of Passes (AHP) and take approximately 18.4 percent of the flow that reaches the lower delta. (Figure 2).

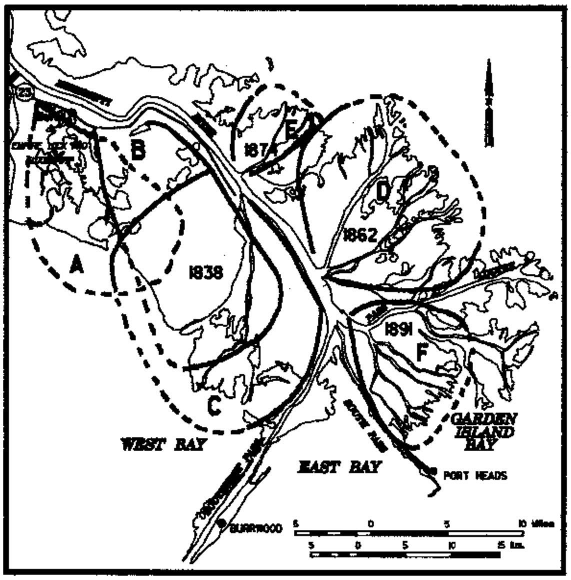


MAJOR LOBES OF THE MISSISSIPPI RIVER DELTAIC PLAIN

FIG I

The smallest Mississippi River subdelta, the Baptiste Collette, began its growth in 1874 after a crevasse broke through the small Baptiste Collette Bayou canal. Sediments were carried northeast into the sheltered Bird Island Sound and deposited in water less than two meters deep. The pattern

MODERN MISSISSIPPI RIVER SUBDELTAS



A DRY CYPRESS BAYOU COMPLEX
B GRAND LIARD COMPLEX
C WEST BAY COMPLEX
D CUBITS GAP COMPLEX
E BAPTISTE COLLETTE COMPLEX
F GARDEN ISLAND BAY COMPLEX

FIGURE 2

of growth was relatively uniform and lasted 72 years (1874-1946). The growth period was followed by a period of stabilization (1946-1958) and then rapid deterioration.

A ditch dug in 1862 by the daughters of Cubit, an oyster fisherman, was responsible for the formation of the Cubits Gap subdelta. With the flood of 1862, the ditch enlarged into a natural crevasse opening which rapidly increased in size. However, initial subaerial delta growth was rather slow, as the deep (9m) receiving basin was filling subaqueously with sediment. A period of rapid growth took place from 1891 to 1905, and then a modest growth rate continued until 1946. Unlike the Baptiste Collette and Garden Island Bay subdeltas, the Cubits Gap subdelta never stabilized; but,entered a rapid destructional phase in 1946.

Prior to 1839, West Bay was connected to the Mississippi River by Wilder's Bayou. A small lock allowed passage of shallow-draft vessels between the river and the bay. During the flood of 1839, the lock was destroyed and Wilder's Bayou became a crevasse known as The Jump. The West Bay complex rapidly developed into the largest subaerial delta in the Mississippi River system. Approximately 86 percent of the total subaerial delta developed between 1845 and 1875. In 1875, the first signs of deterioration appeared and the amount of subaerial land diminished until 1905, when a new pulse of growth began. Deterioration resumed in 1932 and continued until another growth phase occurred between 1971 and 1978.

The Garden Island Bay subdelta, utilizing only approximately four percent of the Mississippi River flow, developed a subaerial land mass that was twice the size of Baptiste Collette. Building to the southeast between Pass a Loutre and South Pass, it filled the shallow Garden Island Bay between 1891 and 1922. However, the life cycle of this subdelta has been the shortest of the four Mississippi River subdeltas; and unless man intervenes, the Garden Island Bay subdelta will revert to open water within 25 years.

PROJECT PLAN

We hope to emulate these four subdeltas with our proposed large scale sediment diversions on the mainstem of the Mississippi River. Cuts will be made in the banks and conveyance channels will be constructed to convey sediments to shallow open water areas. Diversions will be constructed on the left descending bank at mile 7.5 AHP and on the right descending bank at mile 4.7 AHP. Figure 3 shows the location of these diversions.

The selection of the diversion site is very important in maximizing sediment diversion. In general, the inside of a bend is the best location because the bed load is swept to the inside of the curve by the spiral action of the flow. The angle of the diversion channel is also important. The diversion of flow at an angle to the main stream creates a curve. The higher velocity surface water tends to continue with the main stream, while the slower moving water near the bed tends to flow into the diversion channel. Previous model studies have indicated that the best angle for sediment diversion varies with the discharge ratio and with the location of the outflow channel in the bend (Vanoni, 1975). A model study will be used to determine optimum angle and location for the diversions. Figure 4 shows a schematic for a typical diversion.

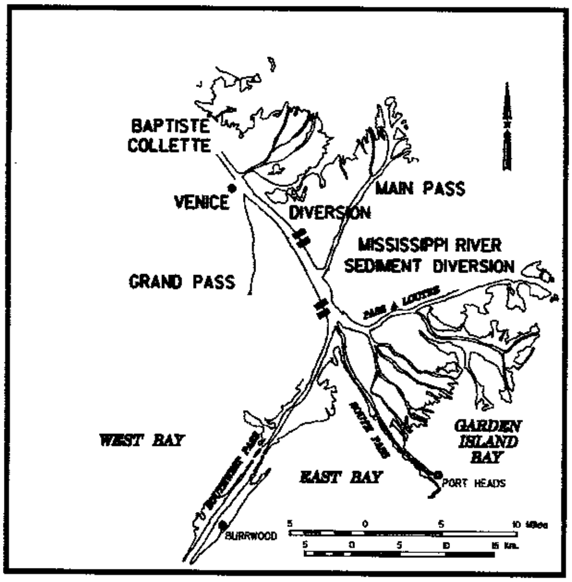


FIGURE 3

Four design flows and three sill depths, for a total of 12 alternatives, were analyzed at each diversion location. A discharge rating curve was computed for each design flow using a broad-crested weir equation. Using this rating curve and an average stage hydrograph for the Mississippi River at each location, an average flow diversion hydrograph was generated. A suspended sediment concentration of 200 parts per million (PPM) was assumed in the diversion. For the deeper cut, the suspended sediment concentration was increased by 15 percent to include the unmeasured bedload. For the middle depth cut, it was assumed that half as much bedload would be diverted; and, for the shallow cut, the suspended sediment was not increased.

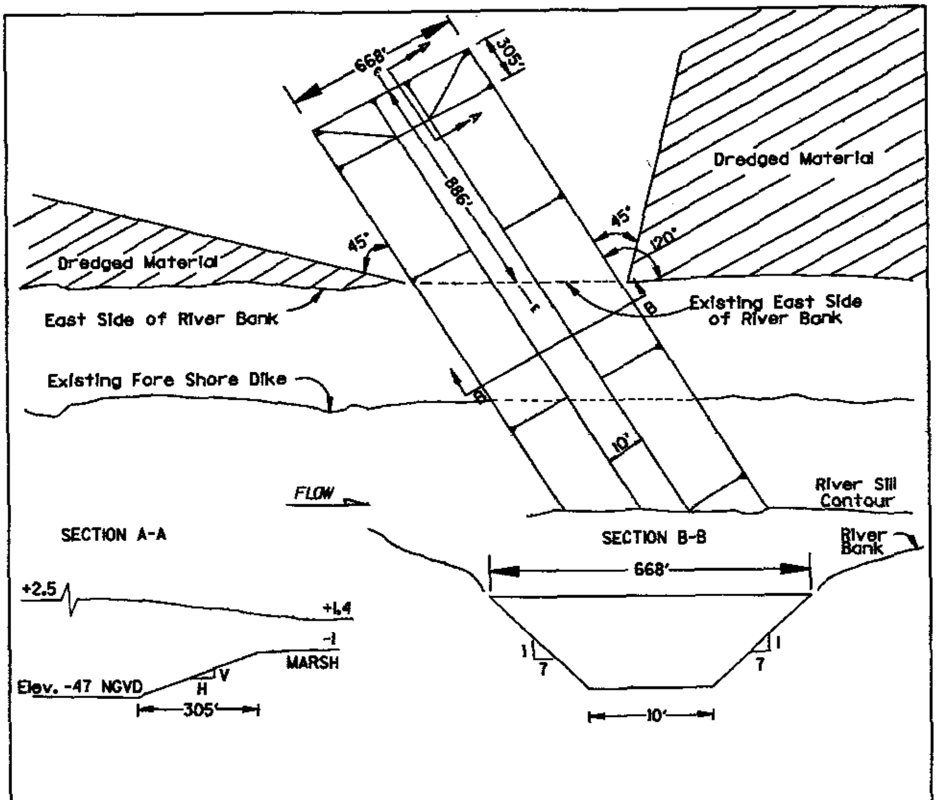


FIGURE 4

The sediment load diverted, Qs (tons per day), was then computed by

$$Qs = Q \times PPM \times 0.0027$$

where Q = flow diverted (cubic feet per second) and PPM = sediment concentration (parts per million). The sediment load was computed for each day of the flow diversion hydrograph and summed to get an annual volume of sediment. Several assumptions were required to convert this volume of sediment into an area of marsh. It was assumed that 50 percent of the suspended load and all of the bedload diverted would deposit. Using a bottom elevation of -2 ft National Geodetic Vertical Datum (NGVD) for the shallow open water areas and allowing for compaction and consolidation, we estimated that six cubic feet of sediment are required to produce one square foot of marsh. Observed land loss rates were applied annually to the available marsh. Thus, an acreage of marsh created over the project life could be determined.

In order to increase the amount of marsh created, the use of Sediment Retention Enhancement Devices (SREDs) was proposed. Several types were evaluated. These consisted of various combinations of piling, wire netting, old tires, etc.; but, the most economically feasible SRED consists of a low earthen dam with 50 foot openings every 1000 feet to permit migration of sealife. The use of SREDs is estimated to increase the retention rate of suspended sediments from 50 percent to 80 percent, with a corresponding increase in the acres of marsh created.

Both large scale diversions will create an estimated 50,000 acres of vegetated wetland over 50 years. After applying current loss rates, we are left with an estimated 22,000 productive acres at the end of the period. The projected total cost per acre is approximately $4000.

As a comparison, we have studied marsh creation using dredged material. The largest of these plans would create approximately 2800 acres over 50 years; with an estimated 1300 productive acres remaining at the end. The projected cost is $6500 per acre.

CONCLUSION

The uncontrolled large scale sediment diversions are not a total solution to the problem of loss of coastal wetlands because a total solution is not achievable. However, measures of this type can be a factor in maintenance of this important national resource.

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EROSION STUDIES IN BURNED FOREST SITES OF GEORGIA

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ABSTRACT

Runoff and sediment was measured from 1x5 m plots established on a cut-over and burned mixed pine and hardwood site in the Georgia Piedmont. Trees on the study site were cut and removed without mechanical intrusion. Slash was left on the plots and burned prior to the first simulated rain event. Three slopes, two rainfall intensities and three initial soil moisture conditions were used. To study the effects of temporal changes in surface conditions and particularly root mat and residual forest floor decomposition, the experiment was repeated four times during the period July 1989-July 1990. The results indicate that the runoff and sediment production was generally low. However, under a combination of 30 percent slope, rainfall intensity of 100 mm/hr and dry initial soil conditions, the runoff hydrograph peaked over 40 mm/hr and the sedigraph peaked over 3.5 t/ha/hr in the early part of the run. These initial peaks were caused by hydrophobic conditions at the plot surface. Under the above conditions, sediment concentration was above 8 g/l at the beginning and gradually dropped to less than 1 g/l near the end of the simulated rain.

INTRODUCTION

Soil erosion from disturbed forestlands is of great concern to forest managers, soil scientists and hydrologists. The problem arises not only from detrimental effects of erosion on soil productivity but also by the adverse effects on water quality.

Site preparation techniques are most frequent causes of disturbance to forestlands. Burning is a common practice used to control understory hardwood, reduce fuel hazards, improve wildlife habitat and prepare seedbeds and sites for planting (Van Lear 1985). Burning, however, can increase the erosion rate by two different mechanisms. First, by destroying the surface litter layer and possibly the underlying fibrous root layer, the mineral soil is exposed and the forces resisting erosion are reduced. Second, burning can decrease the infiltration rate by creating a hydrophobic condition (DeBano 1981), thus, surface runoff will increase and that increases the driving forces for erosion. Arend (1941), in a study of infiltration rates on seven types of soil in Missouri, found that burning reduced infiltration rate 38 percent compared to 18 percent when forest floor was removed by raking.

In an extensive review, Ahlgren and Ahlgren (1960) point out that the role of fire in increasing erosion and surface runoff and in changing soil moisture characteristics has been a subject of much concern in studies of effects of burning on soils. Durgin (1985) suggests that fire's effect on the erodibility of forest soils has been debated for years. He refers to some studies that indicate an increase in erosion with fire and some which fire had little effect on soil loss. Ralston and Hatchell (1971) reported that soil losses caused by burning in a site in North Carolina were over 7.4 t/ha/year. However erosion from five other burned woodlands in the South were about 1.2 t/ha/year or less. Danielovich (1986) found that site preparation burns in the mountains of upstate South Carolina had no impact on infiltration, erosion or soil exposure.

Reliable and consistent data on the rate of runoff and sediment production from burned forest sites is not available in the South. This research is presented with two main objectives: (1) to assess the rate of erosion and runoff from a burned forest site in the Georgia Piedmont and how it changes with time for different levels of slope steepness, rainfall intensity, and antecedent moisture conditions, and (2) to present data on observations of the hydrophobicity phenomenon and discuss its significance on runoff and erosion production.

MATERIALS AND METHODS

The study was conducted at the University of Georgia, Whitehall Forest located in the southern Piedmont region, between July 1989-July 1990. The soil is shallow with a sandy loam texture in the top 50 cm horizon. From 50 cm to 1 m depth, it is mostly clay and below 1 m it is clay loam with mica flakes. Bulk densities determined by the compliant cavity method range from 0.81 g/cm^3 in the top 5 cm depth to a maximum of 1.6 g/cm^3 at 15 cm depth. A fibrous layer of fine roots, typical of hardwood forests, with a thickness varying between 1 to 3 cm covered the mineral soil in an undisturbed condition.

Three sites with average slopes of 10, 20, and 30 percent were selected in a mixed pine-hardwood stand. On each site, two pairs of 1x5 m uniform slope plots were located. Each pair corresponded to one intensity for application of simulated rain. Trees were cut and removed without mechanical intrusion. Slash was removed, kiln dried and replaced on each site and burned prior to plot setup. No measurement of temperature during burning was made, however, similar burning on nearby plots during summer 1990 indicated temperatures as high as 380° C occurred at and just below the surface. Following the burning and before the application of the first simulation rain event, a pair of 1x5 m plots with 15-cm metal sidewalls were established on each of the two locations within each slope class.

Two simulated rainfall intensities, 71.1 mm/hr and 101.6 mm/hr were used. The first rainfall intensity has a recurrence interval of 2 years for a 30 min. duration and a recurrence interval of 25 years for a 60 min. duration. The second rainfall intensity has a recurrence of 10 years for a 30 min. duration and 100 years for a 60 min. duration (Hershfield 1961).

Initial soil moisture conditions were created by using three rainfall applications, dry, wet and very wet runs. The wet run was applied about 24 hours after the dry run and the very wet run was applied about 30 minutes after the end of the wet run.

The oscillating nozzle rainfall simulator used for this study had the nozzle spacing, water supply and recirculating system of the Purdue-type simulator (Foster et al. 1982), with the nozzle opening at the axis of oscillation (the Kentucky-type of Moor and others, 1983). A Veejet 80150 nozzle (Spraying System Company) was used for rainfall application. The nozzle pressure was 41

KPa and the fall height was 3 m. A network of 20 sprinklers spaced 1.52 m apart applied rain uniformly on each pair of plots. The frame supporting the sprinklers was wide enough to permit setup of the two 1x5 m plots with a buffer zone of 50 cm between them.

Runoff samples were collected manually in 1000 ml bottles over timed intervals to define the runoff hydrograph. The sampling time ranged from 15 seconds to several minutes depending upon flow rate and its variation. At the end of each run, the sediment deposited on the covered apron and runoff collector was washed into a bottle. All samples were oven dried and weighed to determine sediment loss.

Effects of temporal changes in surface conditions, particularly the root mat and the residual forest floor, on runoff and erosion rate were studied by repeating the experiment four times (July 24-August 21, August 22-September 8, November 7-25, in 1989, and July 16-26, 1990). Due to time limitations and based on the analysis of results of the first three trials, the fourth trial was conducted only for the high intensity rainfall plots. All vegetative regrowth was prevented during the entire study period by repeated application of Glyphosate as needed.

RESULTS AND DISCUSSIONS

Runoff The total depth of runoff per 30-minute run per plot throughout the period of experiment ranged from a maximum of 5.97 mm or about 12 percent of applied rain (for steep slope high intensity plot, third trial, dry run) to a minimum of a 0.02 mm (for low slope high intensity plot, trial 2, very wet run). The mean depth of runoff per 30 minute period for high intensity rainfall runs was 1.11 mm compared with 0.78 mm for low intensity rainfall runs. These runoff values are about 2 percent of the depth of their respective applied rain. Thus, it was observed that the runoff production potential of these burned sites is generally low.

The low runoff depth was explained by the high infiltration capacity associated with the 1 to 3 cm thick root mat layer on the surface, not affected much by the burning, and the existence of numerous macropores and root holes in the surface organic and mineral soil horizons. Burning removed the litter layer and underlying decomposed organic layer, however it left much of the root mat covering the mineral soil intact. The existence of the root mat layer after burning and subsequent observation of low runoff is consistent with the results of Metz et al. (1961) and Moehring et al. (1966) who suggested changes in infiltration may be too small when surface organic horizons are not completely burned.

Temporal variation of runoff during the period July 89 to July 90 is presented in Table 1. There are no significant differences between the mean of trial 3 compared to trial 4 and between the mean of trial 1 compared to trial 2. However, the means of trials 1 and 2 were significantly different from those of trials 3 and 4. The lack of significant difference in runoff depth between trial 3 and trial 4 indicates that gradual changes in the thickness and spatial coverage of residual root mat during the period Nov. 89 to July 90 was not appreciable enough to increase runoff.

Variation of runoff depth with slope steepness can also be evaluated using Table 1. For high intensity rainfall runs it is shown that increasing the slope steepness from 10 to 30 percent increased runoff nearly four-fold. The increase in slope from 10 to 20 percent increased mean runoff by about 70%. There are no significant differences in mean runoff among slope classes for low intensity runs. These results indicate that under low intensity rain (71.1 mm/hr), slope gradient does not affect runoff production.

Table 1 also indicates the effect of antecedent moisture condition on runoff. For low intensity rainfall, there is no significant difference in runoff depth of the three runs (dry, wet, very wet); however, mean runoff from run 1 was significantly greater than runs 2 and 3, when high intensity rainfall runs were considered.

The fact that dry runs produced significantly higher depth of runoff than wet and very wet runs deserves more detailed explanation. Figure 1 presents the hydrographs of runoff for the three runs on steep slope high intensity rain, plot 2, trials 1-4. It can be observed that during the dry run (run 1), the runoff hydrograph of trial 1 rises sharply to a peak over 20 mm/hr at about 4 minutes into the run and then the hydrograph recedes gradually to a value about 5 mm/hr in the middle and about 2 mm/hr at the end. During the wet and very wet runs (runs 2 and 3), the runoff hydrograph did not show similar peaks and the runoff rates are constantly low. This observation is contrary to the generally known increase in runoff with increasing soil moisture content.

Table 1. Duncan test of significance of runoff means.

All Slopes and Runs		All Trials and Runs		All Trials and Slopes	
Trial	Mean Runoff	Slope	Mean Runoff	Run	Mean Runoff
	mm	%	mm		mm
		High intensity rainfall runs			
1	0.82 b*	10	0.51 b	1	1.47 a
2	0.62 b	20	0.87 b	2	0.88 b
3	1.55 a	30	1.96 a	3	0.99 b
4	1.46 a				
		Low intensity rainfall runs			
1	0.52 b	10	0.80 a	1	0.94 a
2	0.60 b	20	0.79 a	2	0.67 a
3	1.16 a	30	0.76 a	3	0.74 a

* Means with the same letter are not significantly different at 0.05

Similar peaks can be observed in the dry run hydrographs of the same plot in other trials except that peaks had different magnitudes, although the time to peaks were the same. Referring to Fig.1, the magnitude of peak in the second trial of the same plot was about 13 mm/hr as compared to 43 mm/hr in the third trial and 10 mm/hr in the fourth trial. The occurence of the peaks in the hydrographs of dry runs indicate that, at the beginning of rainfall application, there is a resistance to infiltration which gradually decreases as the surface layer is wetted. Differences in the magnitude of the peaks in the hydrographs of dry runs for the four trials is explained by the differences in resistance to wetting during the early parts of these runs. These differences are related to surface moisture and depend upon the depth and time of antecedent rainfall preceeding the runs. The dryer the surface, the higher the peak. The relatively low peak in the dry run of trial four appears to be the result of 76 mm of rain prior to initiation of the simulation run.

These observations of reduced infiltration are explained by temporary hydrophobic conditions at the surface. The water drop penetration time test, (DeBano 1981), which tests the existence of hydrophobic conditions, conducted on the surface of these plots during summer 1990 showed that more than 50 percent of the points randomly selected for this test did not absorb the water drop during the first 5 seconds, the criteria for slightly hydrophobic conditions. Once the surface was wetted any additional drop added was absorbed immediately. DeBano and Rice (1973) explain that when soil is dry, water repellency severely restricts or even stops intake rate, however as the water content increases, the resistance to wetting diminishes, and when wet it will transmit water as rapidly as a wettable soil. Peaks similar to those in the dry run hydrographs of steep plots under high intensity were observed in the dry runs of other treatments, although they were relatively lower in magnitude. This indicates similar resistance to wetting during the dry runs of other plots, however due to lower slope gradient and lower rainfall intensity, not much runoff is moved down the slope during the repellency period, therefore the hydrographs were not as peaked.

Sediment production The total weight of sediment per 30-min. run per plot throughout the period of experiment ranged from a maximum of 189.7 g (high slope high intensity plot, third trial, dry run) to a minimum of 3.0 g (low slope low intensity plot, first trial, dry run). The mean weight of sediment production for high intensity rainfall runs was 30.24 g per plot or about 60 Kg/Ha compared with 21.04 g per plot or 42 Kg/Ha for low intensity rainfall runs. Thus, the erosion rate from plots in burned mixed pine hardwood site is low. The low erosion rates can be explained by high erosion resisting forces due to residual root mat layer after burning and low erosion driving forces due to high infiltration rates and low runoff from the plots.

Suspended sediment concentration and sediment transport rate for the steep slope high intensity rainfall runs of trial three is presented in Fig.2. The sediment concentration peak is about 8 g/l and sediment transport rate is over 3.5 t/ha/hr at the beginning of dry run, gradually dropping to less than 1 g/l and 1 t/ha/hr near the end of dry run and continuing at about the same lower rate into wet and very wet runs. This observation is explained by the higher infiltration rates and lower runoff volumes with time after rainfall starts.

Temporal variation in sediment production during the period July 89 to July 90 is presented in Table 2. The mean weight of sediment production per plot per run from high intensity rainfall ranged from 19.8 g in trial 2 to 39.6 g in trial four. There were no significant differences among the means of trial 1, trial 3, and trial 4. These results are consistent with the results of runoff production from high intensity rainfall which indicate no change with time due to lack of any significant changes in surface conditions of the plots.

Table 2. Duncan test of significance of sediment means.

All Slopes and Runs		All Trials and Runs		All Trials and Slopes	
Trial	Mean Sediment	Slope	Mean Sediment	Run	Mean Sediment
	g	%	g		g
		High intensity rainfall runs			
1	24.21 b a*	10	15.96 b	1	40.08 a
2	19.80 b	20	26.31 b	2	27.21 a b
3	37.34 a	30	48.47 a	3	23.45 b
4	39.63 a				
		High and low intensity rainfall runs combined			
1	23.97 b a	10	16.46 b	1	31.86 a
2	18.72 b	20	30.44 a	2	22.24 b
3	29.57 a	30	25.34 b a	3	18.13 b

* Means with the same letter are not significantly different at 0.05

Effect of slope steepness on sediment production can be evaluated using data shown in Table 2. The mean sediment production per plot per run based on high intensity rainfall runs for the three slope classes 10, 20, and 30 percent are 15.96 g, 26.31 g, and 48.47 g respectively. The mean sediment production from the 30 percent slope was significantly greater than in the 20 percent and 10 percent slope classes. No appreciable difference in sediment production ocurred between 10 and 20 percent slope.

Antecedent moisture condition also appeared to affect sediment production. The mean weight of sediment production per plot per run based on combination of high and low intensity rainfall runs were 31.86 g for dry run, 22.24 g for wet run, and 18.13 g for very wet run. The dry run value, thus obtained, was significantly greater than the wet and very wet run values.

CONCLUSIONS

This study indicated that runoff and sediment production from 1x5 m plots on burned mixed pine hardwood sites with slope steepness as high as 30 percent and intensity as high as 100 mm/hr is low. Temporal changes in surface conditions during a period of one year were not sufficent to result in significant differences in runoff and sediment production over time. Runoff and sediment production were most closely related to slope and rainfall intensity factor. Relatively high runoff and sediment production values occured during dry runs on steep slopes at high (100 mm/hr) intensity. This was due to temporary non-wettable condition of the dry surface organic material covering the mineral soil.

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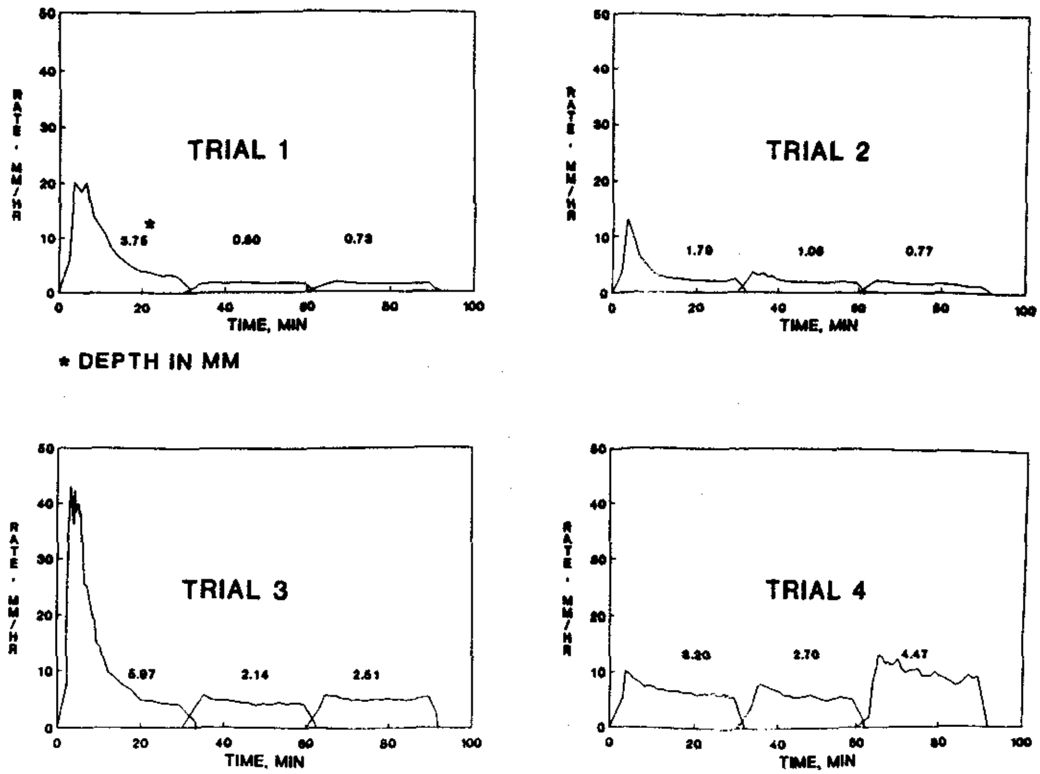

Fig. 1 - Hydrographs of Steep Slope High Intensity Rain Runs, Plot 2

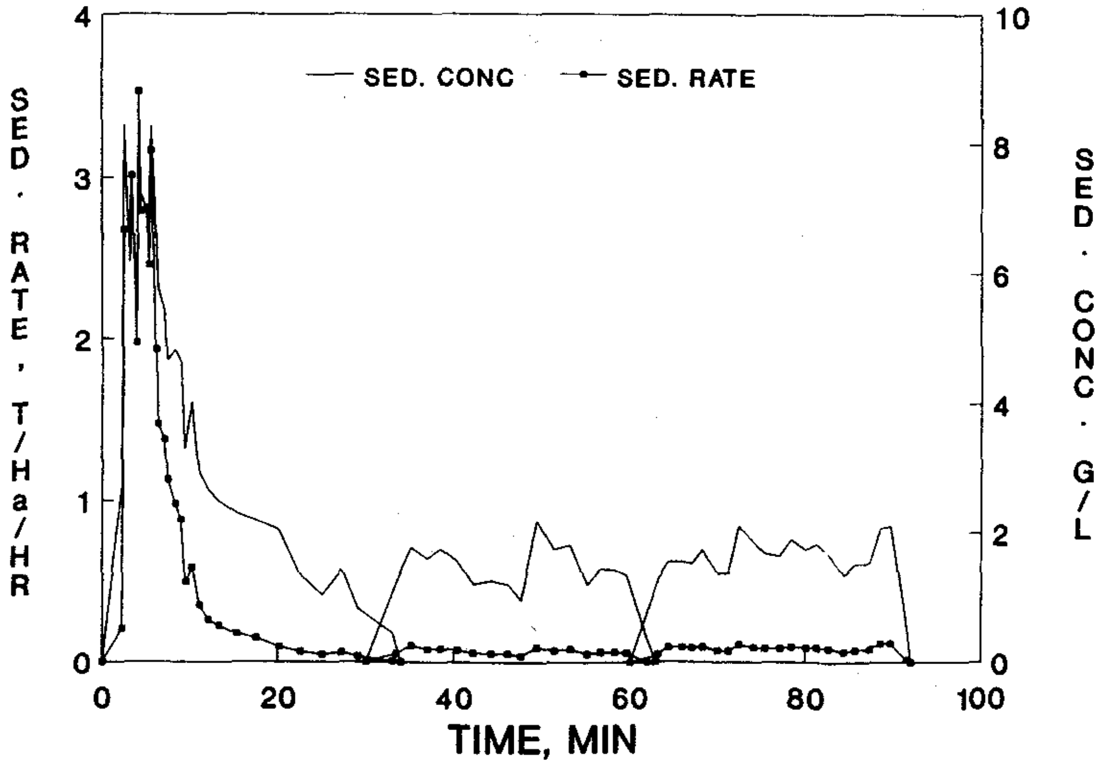

Fig. 2 - Sedigraph of Steep Slope High Intensity Rain Runs, Plot 2, Trial 3

SEDIMENT MANAGEMENT IN MAN-MADE BACKWATER AREAS

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ABSTRACT

The Missouri River from Sioux City, Iowa to the mouth near St. Louis, Missouri was channelized, beginning in the 1920's. The intent of the project was to provide streambank stabilization, flood control, and navigation. The natural channel was a wide braided river which was free to meander between the high bluffs. Natural alluvial processes and periodic flooding maintained a channel with small side chutes, wetland marshes, and oxbow lakes. These off-channel areas provided a diverse and abundant habitat for numerous fish and wildlife species. When the project was completed in 1978 the river was a single channel ranging in width from 600 feet at Sioux City to 1100 feet at the mouth. Nearly all of the side chutes, marsh areas, and oxbow lakes were cutoff from the main channel. At this time natural alluvial processes were essentially stopped, although periodic flooding continued to cause many of the low elevation areas adjacent to the main channel to be filled with sediment. Over a number of years these areas became part of the high bank and were cleared for agricultural proposes. The result of all of this was less habitat for fish spawning and rearing and fewer slack water areas for water fowl.

The Corps of Engineers, Omaha District has attempted to prevent further loss of habitat at a number of these areas through a series of notches in the dikes and revetments adjacent to the remaining low elevation areas. This has met with limited success. Other agencies have tried to create isolated perched lakes or flat water environments in old oxbows through diversions of cooling water from power plants. This approach has work relatively well but is very expensive to construct and difficult to maintain.

Mitigation of fish and wildlife losses along the Missouri River from Sioux City to the mouth has been authorized by congress. The Corps of Engineers, Missouri River Division has developed an implementation plan which includes conceptual designs. More recently the State of Nebraska has requested the Omaha District refine designs for specific restoration projects under a separate authority. These designs involve sediment management plans and hydraulic control structures which will increase the amount, quality, and diversity of the aquatic and riparian habitat adjacent to the main channel, and provide access to and from the river for migrating fish. Geomorphological analyses, sediment analyses, and hydraulic models are used to determine both the suitability of each site, and the most effective method for restoration.

INTRODUCTION

This study is part of an entire river corridor project aimed at increasing the quality and quantity of riparian and instream habitat, increasing recreational opportunities, and preserving cultural and historical sites along the right bank of the Missouri River from South Sioux City, Nebraska downstream to the mouth of the Platte River. The objective of this study was to investigate the possibility of restoring chutes and backwater areas along the Missouri River for fish spawning and rearing by either reconnecting the oxbow lakes to the main channel, or by dredging them to obtain adequate depths and surface areas. Emphasis was given to design considerations that minimized habitat loss by introducting as little sediment as possible into the chutes and oxbow lakes. The analyses presented in this paper are based on existing data only. No new data was obtained for this study. River surveys, water surface profiles, and discharge measurements are as recent as 1988; the topographic maps used to evaluate the backwater areas were based on aerial photography flown in the early and mid 1970's. Therefore, the results of this study must be viewed as preliminary.

PAST MANAGEMENT PLANS

The Corps of Engineers and the State of Iowa, Department of Natural Resources (DNR) have attempted to preserve wetland and slack water habitat along the Missouri River. The DNR has constructed an isolated perched lake along the left bank of the river just downstream of Dakota City, Nebraska between river miles 713 and 716. The lake, Synder Oxbow lake Complex, is an old river chute that was cutoff when the river was channelized for navigation. Subsequent streambed degradation and sediment deposition had left the chute almost completely cut off from the main channel. In an attempt to maintain water levels in the chute area the DNR constructed an earthen closure structure at the downstream end of the chute. The closure structure acts as a dam to prevent the backwater area from draining during the winter low water season. This closure completely cutoff the chute from the river. Therefore, any sediment that entered the chute at the upstream end would be trapped and over time the chute would be completely filled. To maintain water levels, a sediment free source of water was needed. Three water sources were considered: (1) diverting water from the Missouri River or small tributary streams into a sediment detention basin, allowing the sediment to fall out, and then releasing the water into the lake, (2) installing wells at the upstream end of the lake, and (3) diverting cooling water from nearby power plants. Diversion of Missouri River water would require a very large detention pond. There are no tributary streams close to the lake so this was not a feasible alternative. Wells at the upstream end could be installed to supply water to the chute area from the local aquifer. However, the operation and maintenance costs proved to be prohibitive. Because the site is located immediately downstream of the Iowa Light and Power Company's George Neal Power Station diversion of cooling water was selected as the source of water. A supply canal from the cooling water outlet to the lake could be constructed easily and relatively inexpensively, and the water supply was sediment free and reliable on a year around basis. The project now provides wetland habitat and fish spawning and rearing areas that are independent of the operation of the navigation project. The advantage to the isolated lake approach is the nearly constant control of the water surface area and depth. The disadvantage is that the chute area is permanently cutoff from the main channel which further decreases the habitat value of the main channel. This project has functioned quite well, however, construction of a similar project without a sediment free water supply, such as a power plant, immediately available would be very expensive.

The Omaha District began an intensive structure notching program in the mid 1970's in an attempt to create fish spawning and rearing habitat, as well as increase the flow capacity of the channel. Notches were placed in both dikes and revetments. The dikes were generally notched next to the high bank, and the revetments were notched near the downstream ends. The notches ranged for 20 to 50 feet wide and from 5 to 10 feet below the normal navigation water surface elevation or Construction Reference Plan (CRP). Studies of the notched dikes indicate an increase in the slack water areas downstream of the dikes. Surface areas increase 3.5 percent at the CRP elevation, 7.0 percent at two feet below the CRP, and 11.2 percent at four feet below CRP. A mean bed elevation in the plunge pool immediately downstream of the notch lowered approximately 2.0 feet. Although the area and the depth of the pool increased, this structure modification did not achieve the desired habitat improvements. This is likely due to the fish being trapped in the downstream scour hole once the navigation season is over. The pools are not large enough to support the fish through the river. This method slightly increased the wetland habitat along the main channel.

Because the isolated lake concept does not add to the habitat value of the main channel, and near channel habitat (notches) is very difficult to develop and maintain, it is necessary to restore as many side chutes and oxbow lakes as possible without effecting navigation.

ANALYSIS

Two separate types of analyses were performed: (1) a sediment concentration analysis at the Sioux City, Iowa and Omaha, Nebraska gages, and (2) a hydraulic analysis of three selected sites along the Missouri River in the Corridor Project area. A discussion of each analysis follows:

Sediment Analysis

The Missouri River Streambank Stabilization and Navigation Project, located between Sioux City, Iowa and St. Louis, Missouri, is a self scouring channel designed to eliminate routine maintenance dredging. This is achieved through constriction of the channel and flow regulation from Gavins Point Dam near Yankton, South Dakota. This has changed not only the sediment carrying capacity of the river but the types and amount of sediment available for transport. Therefore, an investigation of the Missouri River sediment concentrations through the project area was necessary to estimate project life and identify maintenance needs. Special point integrated sediment data at both Sioux City and Omaha were analyzed to determine the change in sediment concentration at various percentages of the water column. The data was examined three ways: (1) the average concentration versus percent of depth for each sampling date and for each grain size fraction, (2) the average yearly concentration versus percent of depth for the total concentration and each grain size fraction, and (3) the percent of change in sediment concentration in the top 25 percent of the water column. All of the concentrations were normalized to minimize the influence of total Missouri River discharge.

Sioux City Results

Special point integrated sediment data at this sampling station was available for the years of 1982 and 1984 through 1987. The analysis of each individual sample showed no discernible trends except for the 0.210 to 0.149 mm grain size fraction which showed a general increase in concentration during the months of September and October. Analysis of the yearly average concentration and the percent of change from 1982 showed a general decrease in concentrations. The total concentration in the upper 25 percent of the water column has generally declined from approximately 120 to 60 parts per million (ppm). The larger grain size fractions experienced similar decreases in concentration. The smaller grain size fractions showed no definite trends. The distribution of sediment per grain size fraction in the top 25 percent of the water column and the percent change since 1982 are illustrated in figures 1a and 1b. The total sediment load in the Missouri River has remained nearly constant from the closure of Gavins Point Dam in 1955 to 1972 after which no data has been collected. Bed sampling collected in conjunction with the suspended sediment samples and reviewed over time indicates that the bed in coarsening. Therefore, the decrease in the total concentration and larger grain size fractions in the upper 25 percent of the water column could be expected. As the river bed material continues to coarsen the concentration in the top 25 percent of the water column should continue to decrease.

Omaha Results

The period of record at this sampling station is 1984 through 1988. The analysis of this data revealed that the total concentration in the top 25 percent of the water column decreased by approximately 25 percent. The concentrations in the top 25 percent of the water column ranged from 135 to 78 ppm. The concentrations of all the grain size fractions fluctuated by at least 50 percent over the time period examined. The distribution of sediment per grain size fraction in the top 25 percent of the water column and the percent change since 1984 are illustrated in figures 2a and 2b. The fluctuations in the sediment concentrations evident for Omaha reflects the impact of sediment inflows from uncontrolled tributaries.

Discussion

The small size of the data base and the fluctuations of the individual samples, combined with influences from hydrologic events, prohibit accurate prediction of future sediment concentrations. However, if the average concentrations in the upper 25 percent of the water column remain constant, and only water from the top 25 percent of the water column is used, an estimated 2300 to 3200 cubic feet of sediment per day will be introduced into an oxbow lake or a chute for every 1000 cfs diverted from the Missouri River, depending on where the site location. A portion of this sediment may be deposited in the oxbow lakes or chutes, however, the exact amount of deposition will depend on velocities through the chutes and oxbow lakes can be maintained.

Hydraulic Analysis

Hydraulic analyses of three possible restoration sites were performed using existing data and the HEC-2 water surface profile computer model. The two basic types of sites are: (1) point bar chutes that cut across the insides of sharp bends in the river, and (2) oxbow lakes that are located on either side, and adjacent to the main channel. The three sites were selected based on the quality of the data and the proximity to the main channel. To determine the restoration potential of a particular site required the determination of three parameters: (1) the amount of water that could be diverted from the main channel without adversely impacting on navigation, (2) the amount of water that could physically be diverted, and (3) the capacity of the chute or backwater area. The analysis of the three selected sites are as follows:

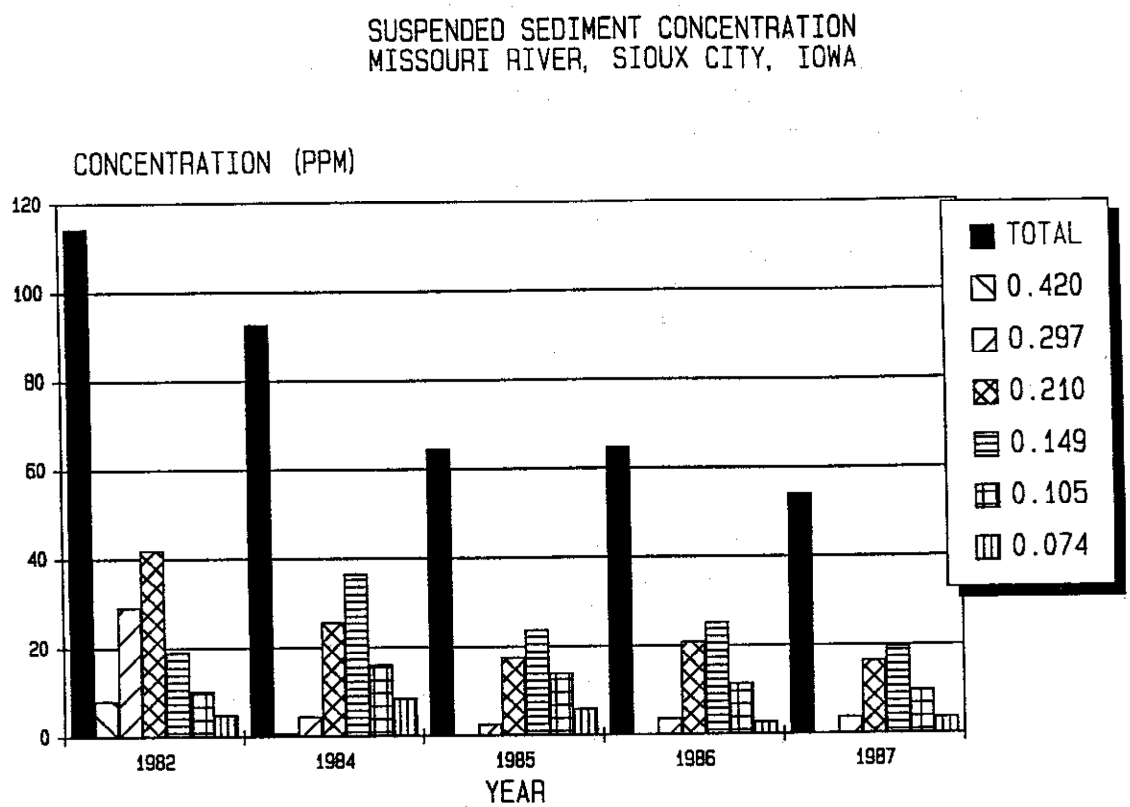

Fig. 1a. Sediment distribution per grain size fraction, Sioux City, IA.

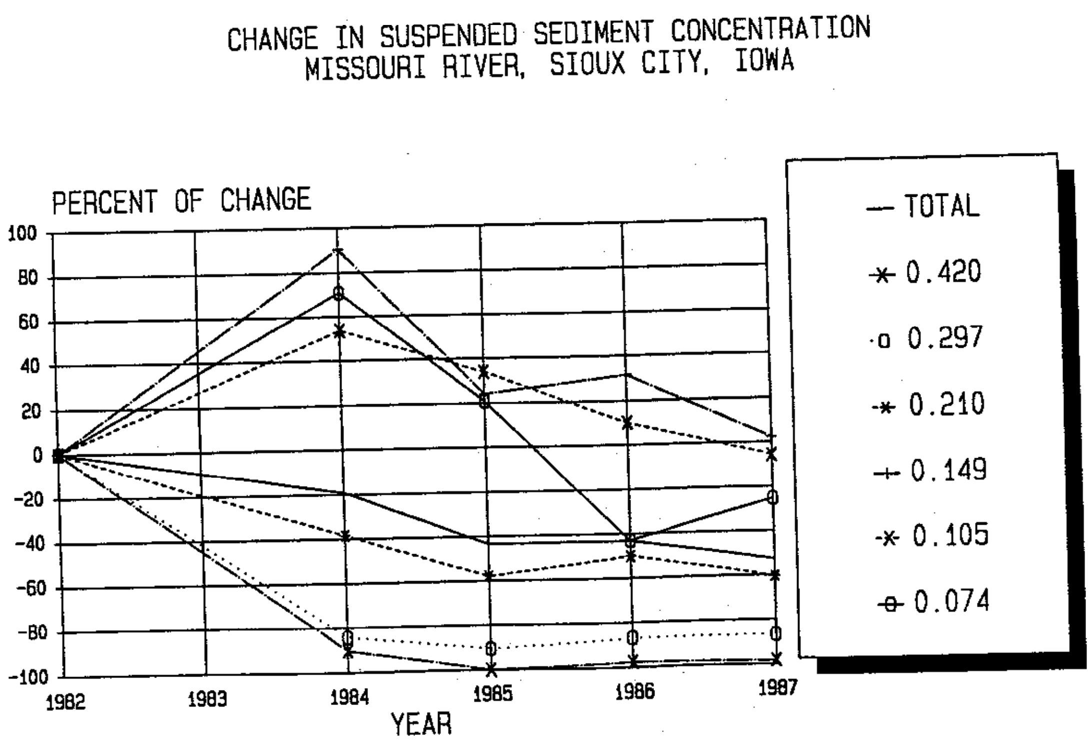

Fig. 1b. Percent change in sediment concentration, Sioux City, IA.

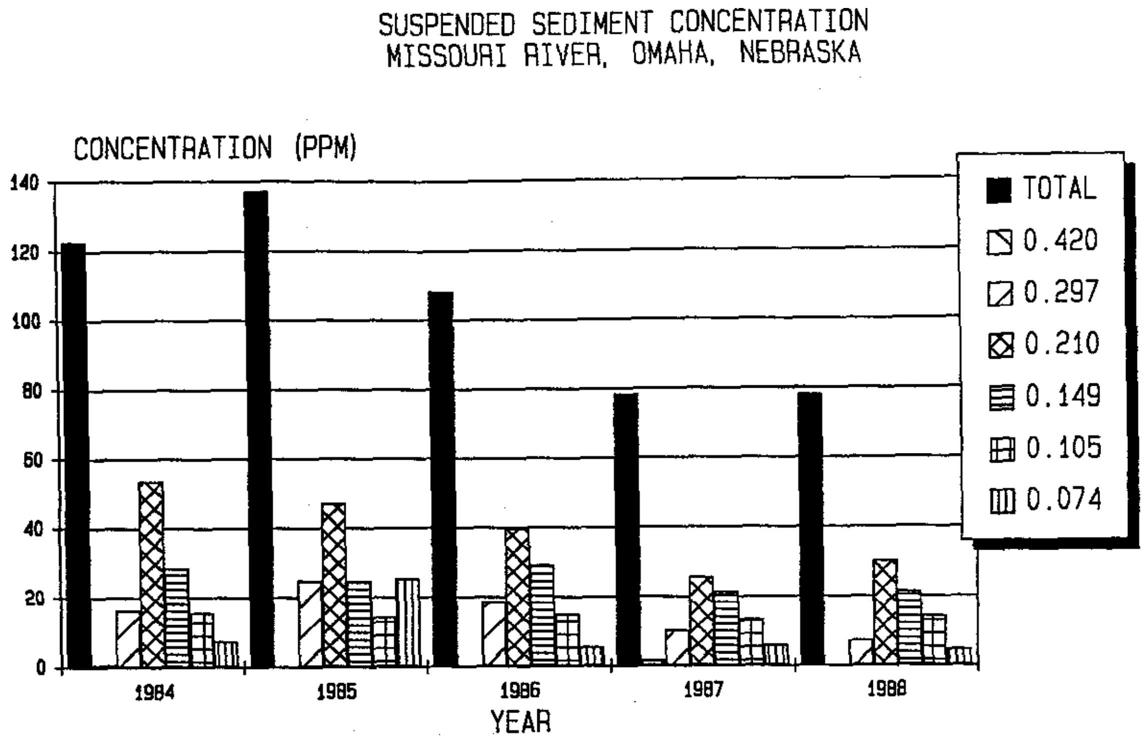

Fig. 2a. Sediment distribution per grain size fraction, Omaha, NE.

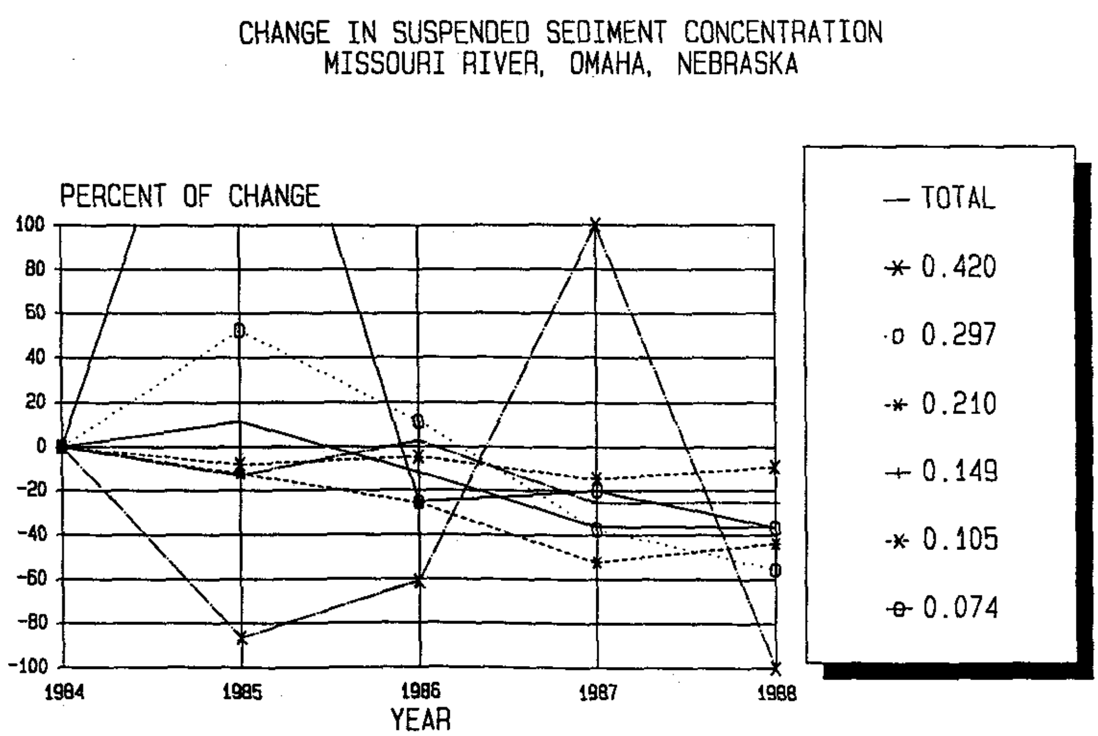

Fig. 2b. Percent change in sediment concentration, Omaha, NE.

Boyer Chute

Boyer Chute is a point bar chute in southwestern Washington County, Nebraska, between river miles 633.3 and 637.7. Standard backwater calculations for the Missouri River indicated that as much as 6000 cfs could be diverted from the main channel without impacting on the navigation channel (300' by 9' minimum). Next the amount of water that could actually be diverted into the chute was calculated. This was done by using the weir flow option of the HEC-2 model to insert a 500 foot notch (weir) in the river control structure at the upstream end of the chute. The bottom of the notch was set at 5 feet below the normal navigation water surface elevation. Calculated discharges through the notch ranged from 8300 cfs during weir flow control to 3500 cfs during normal depth control. To check these calculations a backwater model was set up for the chute. Channel geometry for the chute was developed from existing topo maps. Where necessary, channel geometry was assumed based on a constant bed slope from the upstream to the downstream end of the chute, a flat channel bed , and 1 on 3 channel side slopes. These backwater calculations indicated water surface elevations at the upstream end of the chute to be several feet above the Missouri River stage at the same location. Therefore a water balancing approach was adopted. This involved treating the area between the chute and the river as an island. A rating curve was developed for the upstream end of the chute, and this along with the rating curve for the discharge passing the chute were graphically added to obtain a total flow rating curve. This information was then used to develop a rating curve for discharge through the chute based on the total discharge in the river and the water surface elevations at both the upstream and downstream ends of the chute. This analysis indicated that the tailwater from the river would control the discharges through the chute, at least initially, allowing approximately 1300 cfs through the chute at normal navigation discharges (31000 cfs). As the chute develops (i.e. degradation, erosion, etc.) through fluctuating flow, weir flow or normal flow may control which would allow more water to be diverted into the chute. Computed average cross sectional velocities in the chute ranged from 0.55 fps at the notch to 2.5 fps near the mid point of the chute.

Harbor 671 and Hole-in-the-Rock

The other two sites analyzed are both oxbow lakes located along the outside of the main channel. Harbor 671 is located between river miles 670.5 and 671.5, and Hole-in-the-Rock is located between river miles 705.5 and 706.5. The method for analysis of these site was very similar to that used for Boyer Chute. The main difference is that the chute slope is less than the river slope where as the chute slope at Boyer Chute is greater then the river slope. Different notch and chute geometries were investigated. Notches ranged from 250 to 500 feet. Chute widths ranged form 150 to 500 feet and depths varied from 5 to 10 feet. Regardless of the size and shape of the notch or chute, calculations indicated that the tailwater would control the discharges through the chute. This would allow approximately 1700 cfs to enter Harbor 671, and 1500 cfs into Hole-in-the-Rock. Computed average velocities in the chutes ranged from 0.85 to 1.35 fps.

Discussion

Chute reconnection provided an area of relatively shallow and slow moving water that could be used for fish spawning and rearing. To be effective the average depth in the chutes should be approximately five feet during the navigation season. During the winter months these areas would be for the most part dry. However, because the chutes would be designed to remain open, fish would be able to migrate into the river. Velocities in the main channel are much lower during the winter. Habitat could also be created by dredging large ponds adjacent to the main channel and connecting these ponds to the river at the downstream end. These ponds would have to be dredged to a depth at least 15 feet below CRP to insure fish survival during the winter.

SUMMARY AND CONCLUSIONS

Past attempts to preserve or create habitat along the river have proven to be ineffective due to sedimentation in the areas that are cutoff from the main channel, or extremely expensive as in the case of isolated lakes. Isolated lakes similar to the Snyder Oxbow Lake constructed by the DNR provide an easily managed wetland and fisheries habitat, but do not significantly contribute to the habitat value of the main channel. Attempts to increase slack water areas along the side of the main channel by notching river control structures has met with limited success due to the extreme fluctuations in river stages. Therefore it is necessary to develop a sediment management plan to revitalize as many off channel fish spawning and rearing areas as possible to increase the habitat value of the main channel. Using water from the top portion of the water column would introduce less sediment into the chutes and oxbows. Opening the chutes at both upstream and downstream ends would maintain velocities which decreases maintenance costs.

Reconnecting the chutes and oxbow lakes to the main channel would provide an area of shallow, slow moving water that could be used as fish spawning and rearing habitat. These areas would provide access to and from the main channel during the navigation season. Although the majority of the chute area would be dry in the winter months fish would be able to escape into the main channel. The main channel velocities, in the winter months, are considerably lower then those during the open water season. Fish would be able to winter along the dike fields in the main channel and return to the chutes in the spring to spawn.

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SITE PRIORITIZATION FOR BANK PROTECTION: SACRAMENTO RIVER, CA

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ABSTRACT

The third phase of the Sacramento River Bank Protection project that is part of the Sacramento River Flood Control Project is currently in the planning phase. Downstream of Colusa (RM 144), bank protection is required as part of the flood control project to prevent erosion where flood control levees closely border the river. To date, approximately 500,000 linear feet of bank protection are in place along the river. The majority of the revetments consist of fullbank rock protection. However, environmental and other concerns now dictate that other forms of bank protection be considered. Prioritization of sites for protection also is required.

A procedure for the prioritization of bank protection sites was established on the following: 1) measured rates of lateral bankline migration (MR), 2) the distance from the top of the bank to the levee (D), 3) the requirement for a 30-foot wide buffer strip between the levee and the top of the river bank and 4) the time required to plan and construct a revetment (10 years from 1992). Where levee threat will occur by the year 2002 ((D-30/MR) < 16), high priority sites were identified. A 50 percent underestimation of migration rates was assumed to account for the event-driven nature of bank retreat and potential variations in hydrologic sequencing.

A design matrix was compiled to determine which bank protection techniques were technically appropriate for a given site. Eighteen bank protection techniques that range from fullbank rock revetment and bio-technical protection to combinations of techniques were included in the matrix. Six site characteristics were incorporated into the matrix: 1) extent of protection desired, 2) the latitude for encroachment into the channel, 3) the tolerance for change in bank alignment, 4) the resistance to erosion of the bank materials, 5) the sediment load of the river and 6) the effects of boat traffic and wind-generated waves. Numerical values from 1 (least appropriate) to 5 (most appropriate) were assigned to each bank protection method for each of the site characteristics. Summation of the values provides a ranking of technical suitability for a given method at a given site.

A decision matrix was compiled to evaluate the technically feasible bank protection methods that were identified by the design matrix. Factors in the matrix include: 1) life-cycle costs, 2) construction complexity, 3) availability of materials, 4) degree of top bank disturbance required to implement the method, 5) requirements for bank shaping, 6) conveyance effects on the channel and 7) environmental considerations such as preservation of riparian vegetation, fish habitat, shading and recreational hazards. A similar numerical ranking scheme then is used to select the most appropriate bank protection method for a given site.

INTRODUCTION

The Sacramento River Flood Control Project consists of 977 miles of levees plus overflow weirs, pumping plants and bypass channels along the Sacramento River from RM 0 near Collinsville to RM 194 near Chico Landing, including several sloughs and the lower reaches of major tributaries. The Sacramento River Bank Protection Project is designed to provide protection for the existing levees and flood control facilities of the flood control project. The initial two phases of the Sacramento River Bank Protection Project received authorization by Congress for a total of 835,000 linear feet of bank protection. The third phase of the Sacramento River Bank Protection Project is currently under study. An aspect of this study is the evaluation of the applicability of alternative methods of bank protection to the Sacramento River (WET, 1990). This paper presents results of this evaluation, and includes the methodology for the determination of high priority bank protection sites for the river between Verona (RM 78) and Chico Landing (RM 194) (Fig. 1).

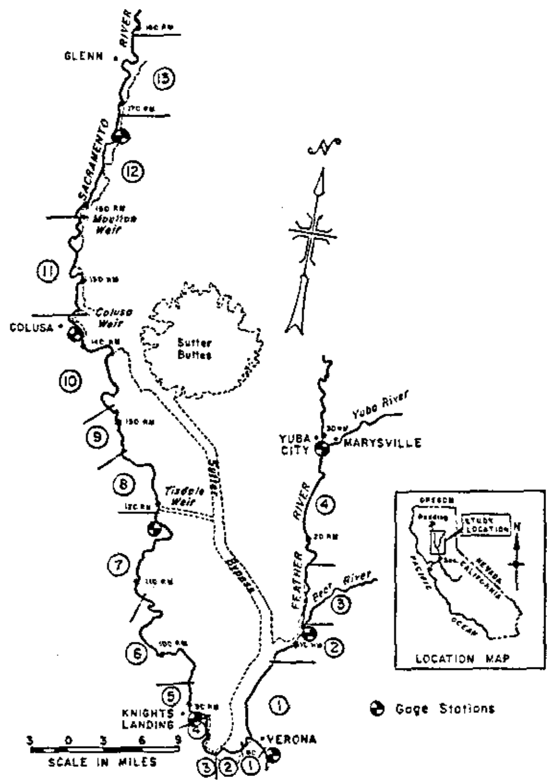

Figure 1. Location map of study reach showing subreach delineations.

GEOMORPHOLOGY OF THE SACRAMENTO RIVER

The Sacramento River is bounded by several natural floodbasins that have been incorporated into the flood control projects as overflow areas. The main river channel becomes smaller and finer-grained in the downstream direction as a result of the loss of high flows. An abrupt change in fluvial character is visible in the vicinity of Colusa (RM 144).

Upstream of Colusa, the Sacramento River flows in a wide, coarse-grained meanderbelt that is characterized by a dynamic planform, high erosion rates, and highly variable hydrologic conditions (WET, 1989). Active channel deposits are gravel-dominated. Fine-grained abandoned channel fills that accrete in oxbows formed as a result of meander bend cutoff are common. Levees are commonly set back between Colusa and Glenn (RM 178). Approximately 7 percent of the bankline consists of Pleistocene-age Modesto Formation which comprises cemented abandoned channel fill and point bar facies. These exposures provide resistance to lateral erosion.

Downstream of Colusa, the Sacramento River is a sand-dominated meandering stream that is closely leveed and extensively revetted (WET, 1990). Bankline migration rates are low due to the fine-grained, cohesive nature of the bounding sediments, which are composed of floodbasin deposits. The floodbasin deposits of the lower Sacramento River consist of silt and clay-rich, massive, impermeable sediments that contain preserved organic matter and manganese concentrations. These deposits are laterally extensive and commonly compose the bank toe of the lower Sacramento River. Due to the overall fine-grained nature of the lower Sacramento River, delineation of various depositional environments is difficult. The preservation of point bars, however, is clearly recorded by exposure of Lateral Accretion Surfaces (LAS) in the bank stratigraphy. These sediments consist of sigmoidal-shaped, alternating sand and silt packages that record bendway migration and point bar development.

The various types of sediments that compose the channel banks display significantly different geotechnical characteristics when they become sites of erosion (Thorne, et al., 1981; Thorne, 1982; Hagerty et al., 1983). Upstream of Colusa, the recent point bar deposits have little or no cohesion and are easily eroded.

Downstream of Colusa, point bar LAS deposits reflect highly variable cohesive strengths. In contrast, the Modesto outcrops, fine-grained abandoned channel fills, and floodbasin sediments tend to be cohesive and provide significant resistance to erosion when exhumed by river migration. Although volumetrically limited, fine-grained channel fill deposits have the ability to cause meander compression and distortion. The floodbasin sediments are laterally extensive and exert significant control on planform geometries of the Sacramento River downstream of Colusa. These inherent differences in geomorphic and sedimentologic characteristics between reaches of the river should be considered in any bank protection method selection process.

SUBREACH TYPES

The study subreaches have been categorized by general reach type to evaluate and recommend bank protection measures for the study reach. The similarity of subreaches allows this general categorization, which in turn simplifies and clarifies the bank protection alternatives scheme.

Type I subreaches include Subreaches 1, 3, 5, 7, 8 and 10 (Fig. 1). The levees directly border the river along these subreaches, and revetment has been extensive. A cohesive toe is generally present within Type I subreaches and, due to revetment, the channel planform is fixed. Bank erosion is slow; however, levee proximity imparts high priority to these sites. Bed sediment is dominated by sand, and progradational point bars are not present due to the extensive revetment that has fixed the channel planform.

Type II subreaches include Subreaches 4, 6 and 9, and are characterized by setback levees that allow channel migration, and by sand and finer-grained bed and bank sediment. Active channel migration within Type II reaches has generated bank stratigraphy that includes lithologically complex lateral accretion surfaces which record point bar migration. A cohesive toe composed of floodbasin deposits is commonly present.

Type III subreaches include the coarse-grained meanderbelt of Subreaches 11, 12 and 13 and the Butte Basin. The bed and bank sediment within Type III subreaches is dominated by coarse, erodible sands and gravels. Point bars are large and chute cutoffs are common. Fluvial subenvironments are extremely diverse. Annual peak flows are highly variable.

HIGH PRIORITY SITE SELECTION

The general bank protection strategy between Verona and Colusa has been to riprap remaining berms to the top of bank where levee erosion is imminent. Additional levee setback also may be feasible. On the leveed reach of the Sacramento River upstream of Colusa, the current bank protection strategy is to limit bank protection sites to where levee erosion is imminent, or where the effective operation of weirs may be adversely affected by channel migration.

Due to the fine-grained nature of the channel banks along the majority of the study reach, bank failure on the Sacramento River is commonly massive (Thorne, et al., 1981). Consequently, bankline retreat is sporadic. In addition, flood control levees are commonly located within 100 feet of the river. Sporadic mass failure of a bank therefore could jeopardize levee stability without following a constant, predictable rate of bankline retreat. As a result, a 30-foot wide "buffer strip" has been used in the third phase study in the determination of priority sites to allow for potential sporadic massive bank failure. Bankline migration rates were calculated using data retrieved from bankline comparisons of 1981 and 1986 aerial photography. This time period includes two flood years (1983 and 1986) and therefore provides a relatively conservative evaluation of levee threat. In addition, a 50 percent underestimation of migration rates has been assumed so that the event-driven nature of bank retreat and possible variations in hydrologic sequencing are accounted for. Priority sites were based upon the projected rate of retreat reaching a levee within a prescribed 10-year project execution period starting in 1992. Sites within the Butte Basin were evaluated in terms of levee threat and proper functioning of flood relief structures.

The following equation was utilized for site prioritization: $(D-30)/(MR * 1.5)$ = time to buffer zone (years) where D = distance to levee (ft) and MR = measured bankline migration rate (ft/yr). Using this equation, 17 sites have been identified as high priority, representing a total of 27,756 linear feet of bank. Sixteen high priority sites are present between Verona and Glenn. Each of these sites constitutes a direct levee threat.

Two high priority sites are present within the Butte Basin. One of these sites is high priority as it constitutes a hydraulic control of the Butte Basin. Channel alignment of this bendway must be maintained to ensure proper operation of a flood relief structure.

EVALUATION OF PROPOSED BANK PROTECTION METHODS

A comprehensive general plan for the Sacramento River based on subreach delineations is shown in Table 1. The plan was formulated by the incorporation of general subreach characteristics into the design matrix of Table 2. The determination of the most effective method of bank protection for a given site can be made using the design matrix of Table 2. Table 3 is a decision matrix that allows the incorporation of logistic and environmental considerations into the bank protection alternative selection scheme.

Table 2 is a design matrix that will aid in the determination of the most technically effective bank protection method for given site characteristics. The extent of protection will depend highly on the nature of bank materials present and associated failure modes. For example, where bank failure modes are dominated by undercutting of non-cohesive point bar sediments followed by cantilever failure of overlying overbank fines, toe protection may be sufficient. Latitude for encroachment refers to the potential application of a bank protection method that encroaches onto the channel. In many situations, there will be no room for such encroachment. It is critical to determine if alignment alteration is a goal of the bank protection strategy. In contrast to alignment change, the maintenance of existing alignment may be important. Several bank protection alternatives may apply to sites where topbank or total bank retreat is acceptable to some degree. The bank retreat potential depends on bank materials and hydraulic characteristics of the site.

Suspended sediment concentration is important to the potential application of bank protection alternatives that rely on the deposition of suspended sediment. Lastly, the presence of wind- and boat-generated waves will affect the bank protection method selection. Each of these site characteristics must be considered as part of the bank protection selection strategy. Using Table 2, the scores for each method are added. The several top methods of highest rank then should be considered as technically applicable to the site.

Table 2 aids in the determination of the most effective bank protection methods for given site characteristics. It may not be entirely appropriate, however, to choose a bank protection method purely on the basis of technical effectiveness. Numerous other considerations may need to be integrated into the bank protection strategy. These considerations include costs, maintenance requirements, construction complexity, materials used, shading, minimization of recreational hazards, topbank disturbance, bank shaping, and conveyance effects; and environmental considerations, such as preservation of riparian, fish and endangered species habitat. Bank protection method rankings under these considerations are shown in the decision matrix of Table 3. The environmental considerations of Table 3 are based upon short-term concerns related to bank protection method emplacement. Long-term strategies for habitat preservation may differ.

CONCLUSIONS

The approach to bank protection plan formulation on the Sacramento River should incorporate an understanding of the geomorphic character of the channel. Bank protection alternatives must consider both bank stratigraphy and channel alignment. Many alternatives to full bank revetment are available; however, the applicability of such techniques is site specific.

General conclusions of the study site prioritization and bank protection alternatives scheme may be summarized as follows:

- A total of 17 high priority sites within the study reach (RM 78 to RM 194) have been identified based upon the projected bankline retreat reaching a 30-foot buffer strip on the riverward side of the levee within a prescribed 10-year project execution period starting in 1992. A 50 percent underestimation of migration rates was included to account for the event-driven nature of bankline retreat and potential variations in hydrologic sequencing.

- One high priority site was identified on the basis of proper operation of a flood relief structure.

Table 1. Recommended bank protection for Sacramento River study reach.

Subreach	Location (RM)	Reach Type	Recommended Bank Protection	Bank Protection Alternatives
1	78-80	I	f; Close gaps in revetment; give particular attention to toe trench where cohesive materials exist	f,g+q,h, i (cohesive toe)
2	80-84.5	IV	p; Stabilize areas near weir with stone windrows	f
3	84.5-87	I	f; Close gaps in revetment; give particular attention to toe trench where cohesive materials exist	f,g+q,h, i (cohesive toe)
4	87-88.8	II	p,d; Provide stone windrows at all possible areas adjacent to levees. Monitor outside bendways where stone windrows are not possible. When necessary, stabilize bendways with stone hard points rooted at non-cohesive zones of lateral accretion surfaces	non-cohesive toe: f,g+q,h,j,k,l cohesive toe: i,m
5	88.8-95.5	I	f; Same as subreach 3	
6	95.5-107.5	II	p,d,n,o; Same as subreach 4, except stabilize bends with rock at toes when required and use upper bank vegetation	non-cohesive toe: f,g+q,h,j,k,l,n+o+q cohesive toe: i,m
7	107.5-118.5	I	f; Same as subreach 3	
8	118.5-125.5	I	f; Same as subreach 3	
9	125.5-131.5	II	p; Provide stone windrows at setback locations, similarly to subreach 4. Stabilize bends similarly (RM 126.2 priority)	Same as subreach 6
10	131.5-144	I	f; Same as subreach 3	Same as subreach 3
11	144-158	III	p,n,i,k; Provide stone windrows at setback locations. Stabilize critical bends with stone toes where non-cohesive. For upper bank, use stone blanket, live cribwall or brush mat, depending on room to lay slope back and hydraulic shear	non-cohesive toe: n+o,j,k,h,l cohesive toe: i,m,i+o
12	158-169	III	p,n,l,k; Provide stone windrows at setback locations. Stabilize critical bends with stone toes where non-cohesive. For upper bank, use brush mat or seeded slope reinforced by geosynthetic material	non-cohesive toe: j,k,l,n+o cohesive toe: i,m,i+o
13	169-178	III	p,n,l,k; Same as subreach 12	
14	178-194	III	p,n,l,k; same as subreach 12	

a Kellner Jacks
b Fence Dikes
c Rock Dikes
d Hard Points
e Hard Points & Armor
f Full Bank Revetment
g Partial Revetment
h Cribwall w/Rock Toe
i Cribwall
j Branch Pack w/Rock Toe
k Brush Mat w/Rock Toe
l Seed Slope w/Rock Toe
m Seed Slope w/Bankline Veg.
n Toe Riprap
o Bankline Vegetation
p Rock Windrow
q Setback Vegetation
r Overbank Vegetation

Table 2. Design matrix for determination of most effective bank protection method for given site characteristics.*

Site Characteristics	Extent of Protection Desired		Latitude for Encroachment			Importance of Alignment Change		Importance of Alignment Maintenance		Tolerance for Bank Retreat			Bank Retreat Potential		Suspended Sediment Conc.		Wind and boat generated waves	
Methods	Full	Toe	High	Low	None	High	Low	High	Low	Total Bank	Top Bank	None	Low	High	High	Low	High	Low
Kellner Jacks	1	2	5	3	1	5	1	4	1	1	1	1	1	3	5	1	2	3
Fence Dikes	2	2	5	3	1	5	1	4	1	1	1	1	1	3	5	1	1	3
Rock Dikes	5	5	5	5	1	5	1	5	1	1	1	1	1	3	5	3	4	3
Hard Points	3	3	3	5	1	4	1	3	1	1	3	2	1	3	5	3	4	3
Hard Points w/ Bank Armor	3	5	3	5	1	4	1	4	1	1	2	2	1	3	5	3	5	3
Full Bank Revetment	5	5	1	1	5	1	5	5	1	1	3	5	1	5	5	5	5	3
Partial Revet. to N.H.W.	2	5	1	1	5	1	5	3	3	2	5	3	2	5	5	5	4	3
Live Crib w/Toe Riprap	3	5	1	1	5	1	5	4	3	1	3	4	3	4	5	3	3	3
Live Crib w/o Toe Riprap	3	2	1	1	5	1	5	3	3	2	3	4	3	2	5	3	1	3
Branch Pack w/Toe Riprap	3	5	1	1	5	1	5	3	4	1	3	2	3	3	5	5	3	3
Brush Mat w/Toe Riprap	3	5	1	1	5	1	5	3	4	1	3	2	4	2	5	5	3	3
Seeded Slope w/Toe Riprap	2	5	1	1	5	1	5	3	4	1	4	2	4	2	5	5	1	3
Seeded Slope w/Bankline Veg.	1	2	1	1	5	1	5	2	4	4	4	3	5	1	5	5	1	3
Toe Riprap	1	5	1	1	5	1	5	3	4	1	5	3	4	5	5	5	3	3
Bankline Veg.	1	1	1	1	5	1	5	2	5	4	3	2	5	1	5	5	1	3
Riprap Windrow	4	3	1	1	5	1	5	1	5	5	5	1	4	5	5	5	3	3
Setback Bankline Veg. (trees)	1	1	1	1	5	1	5	1	5	3	5	1	5	3	5	5	1	3
Overbank Veg. (cane/grass)	1	1	1	1	5	1	5	1	5	3	5	1	5	1	5	5	1	3

*To rank methods, determine site characteristics and add scores. Highest scores delineate most effective methods for given site.

Table 3. Bank protection decision matrix for determination of which methods chosen from design matrix (Table 10.3) are most desirable.**

Bank Protection Method*	a	b	c	d	e	f	g	h	i	j	k	l	m	n	o	p	q	r
Life-Cycle Costs	2	2	4	4	4	5	5	3	3	3	3	4	4	5	5	5	5	5
Landside Maintenance	1	2	5	5	5	5	4	5	5	5	5	5	5	4	5	5	5	5
Waterside Maintenance	5	4	2	1	1	1	1	1	1	1	1	1	1	2	1	1	1	1
Construction Complexity	1	1	3	3	3	3	3	3	3	4	4	4	5	4	5	4	5	5
Natural/Local Materials	1	1	5	5	5	5	5	5	5	5	5	4	4	5	4	5	5	5
Topbank Disturbance	5	2	2	2	1	1	3	3	3	3	4	4	4	5	5	5	5	5
Bank Shaping	5	2	2	2	2	1	2	3	3	2	3	3	3	2	2	5	5	5
Conveyance Effects	2	2	1	3	3	4	4	4	4	5	5	5	5	5	4	5	5	5
Environmental- Riparian Vegetation	5	4	3	3	2	1	2	3	3	3	3	4	4	5	5	5	5	5
Fish Habitat	5	5	5	4	4	2	2	4	4	3	3	3	5	3	5	5	5	5
Bank Swallows	5	3	2	2	1	1	1	1	1	1	1	1	1	3	2	5	5	5
Rec. Hazards	1	1	3	3	3	4	4	3	3	5	5	5	5	5	5	5	5	5
VELB***	5	2	1	1	1	1	2	2	2	3	3	3	3	4	5	5	5	5
Shading	5	2	1	1	1	1	2	3	3	3	3	3	3	4	5	5	5	5

* Bank Protection Methods:

a Kellner Jacks
b Fence Dikes
c Rock Dikes
d Hard Points
e Hard Points & Armor
f Full Bank Revetment
g Partial Revetment
h Cribwall w/ Rock Toe
i Cribwall
j Branch Pack w/ Rock Toe
k Brush Mat w/ Rock Toe
l Seed Slope w/ Rock Toe
m Seed Slope w/ Bank Vegetation
n Toe Riprap
o Bankline Vegetation
p Rock Windrow
q Setback Vegetation
r Overbank Vegetation

1= Least Favorable
5= Most Favorable

**To rank methods, determine concerns affecting decision and add scores. Highest scores delineate most suitable method for given site. Use in conjunction with design matrix of Table 10.3.

***(Valley Elderberry Longhorn Beetle)

- A general bank protection plan has been formulated for each study subreach (Table 1). This plan utilizes the general geomorphic and hydrologic characteristics of each subreach to delineate which bank protection methods will be most applicable.

- A design matrix (Table 2) allows the determination of the most effective form of bank protection for a given site.

- A decision matrix (Table 3) may be used in conjunction with the design matrix to account for considerations other than simply method performance. These considerations include environmental concerns, construction complexity, topbank disturbance and life-cycle costs.

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EROSION AND SEDIMENT TRANSPORT ON CUT SLOPES

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ABSTRACT

A physics-based model is discussed for analyzing the flow and transport of sediment material on steep slopes. A kinematic wave approximation is utilized for the flow discharge. The processes of detachment, entrainment and deposition of sediment particles are represented in the transport mechanism of soil. Some preliminary results are presented showing the sensitivity of sediment discharge to flow parameters.

INTRODUCTION

Modern highways are often built to very exacting standards resulting in very steep side-slopes adjacent to roads. This is particularly true for the cut decomposed granite slopes along the new alignment of Route 299 in Shasta County in California. Decomposed granite is fairly cohesionless material and is prone to erosion when subjected to external influences like rainfall and snowmelt. The eroded soil fills up the drainage ditches and sedimentation basins along the highways, plugs the culverts and drains and causes washing of the road by surface flows. Cleaning up the drainage facilities after heavy rainfall events by maintenance crews is very expensive and is not a lasting solution.

The processes of erosion and sedimentation over land surfaces and rivers are complicated phenomena which are not yet completely understood. These processes include detachment, transportation and deposition of soil particles (dredge material). Detachment is the dislodging of soil and surface particles from the soil mass by the erosive agents. The subsequent entrainment and movement of the particles now suspended in the surface flow is termed as transportation. Deposition of the suspended sediment occurs when the carrying capacity of the surface flow is reduced due to some physical influences (like obstructions in the flow path, change of bed slope, slowing down of the flow, etc.). The sediment may be deposited only temporarily and may be moved further along the stream system due to subsequent storm events.

Analytical treatment of erosion on upland areas is possible for special cases only, due to the highly nonlinear flow equations and the simultaneous interaction of the flow and sediment transport processes. We therefore utilize numerical analysis techniques to simulate various physical scenarios on a digital computer. Our particular interest lies in the study of cutslopes subjected to erosive and transport agents of raindrop impact and runoff over the soil surface.

MATHEMATICAL FORMULATION

Various conflicting theories are present in the sediment transport literature which purport to describe the physics of eroding surfaces and the subsequent transfer of suspended soil particles through surface flows. We have chosen here a methodology which is intuitively appealing as well as physically justifiable. The emphasis has been on incorporating more of the physics (in the form of conservation of mass principles) and less empiricism.

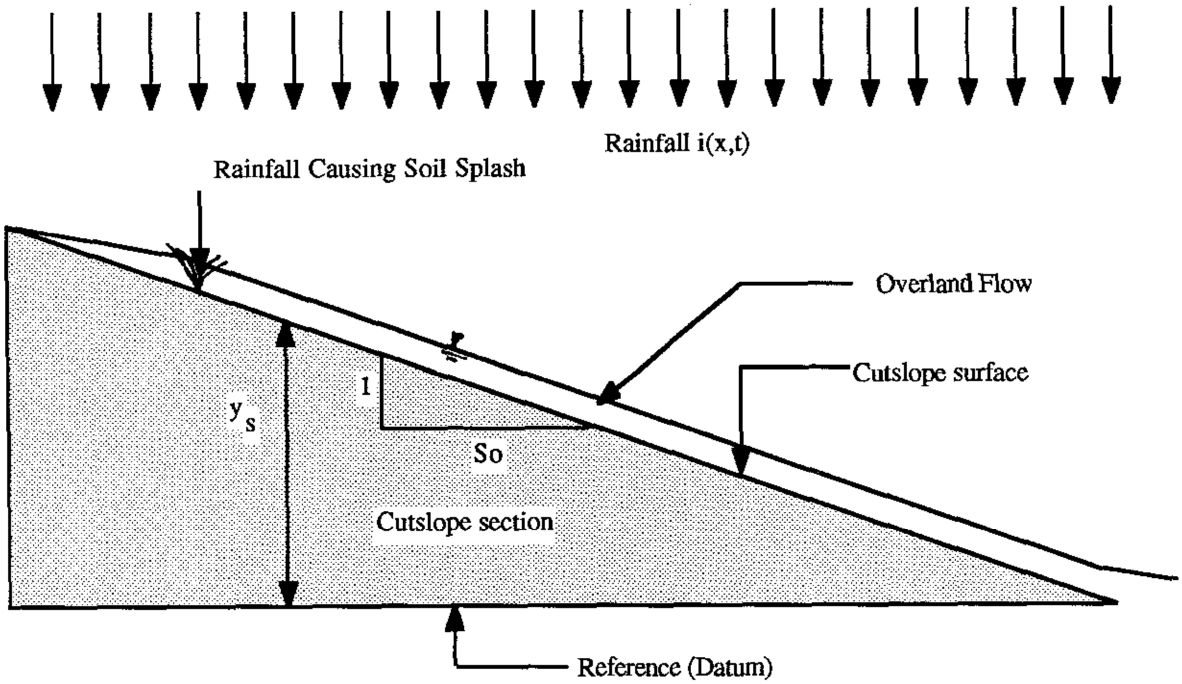

Fig. 1. Schematic view of overland over a cutslope

Figure 1 is a schematic sketch depicting the flow situation over the cutslope section. The sediment particles are removed (or deposited) at various time-space locations by the thin flow of water (resulting from rainfall) on the top of the sediment profile. Hence the active eroding agent in this situation is the overland flow. The equations governing such shallow water flows are derived from conservation of mass and linear momentum principles. The continuity equation for overland flows is (see, for example, Bennet, 1974)

$$\frac{\partial y}{\partial t} + \frac{\partial q}{\partial x} = i(x,t) \tag{1}$$

where $y(x,t)$ is the depth of water over the sediment surface (m), x is the spatial coordinate measured along the slope (m), t is time (sec), $q(x,t)$ is the discharge per unit width of the flow (m^2/sec) and $i(x,t)$ is the net lateral inflow function (m/sec). The momentum equation for overland flows may be modeled by the kinematic wave approximation given as

$$S_f = S_0 \tag{2}$$

where S_f is the friction slope or slope of the total energy line and S_0 is the bed slope over which the flow takes place.
The flow is completely specified by the use of a friction law relating the discharge and the depth at any time and at any spatial location. A frequently used friction law for turbulent flows is given by the Chezy relationship

$$q = C\, y^{3/2}\, S_f^{1/2} \qquad (3)$$

where C is Chezy's roughness coefficient. The process of erosion is presumed to occur so slowly that the change of the water profile with time is much faster than the corresponding change in the sediment profile. The conservation of mass equation for the sediment profile is given by

$$\frac{\partial(cy)}{\partial t} + \frac{\partial(cq)}{\partial x} + (1-\lambda)\frac{\partial y_s}{\partial t} = \frac{S_p}{\rho_s} \qquad (4)$$

where $c(x,t)$ is the volumetric concentration of suspended sediment (vol/vol), λ is the soil porosity (fraction), $y_s(x,t)$ is the sediment elevation from a fixed datum (m), S_p is the detachment rate from rainfall impact (kg/sq.m.-sec), ρ_s is the density of the soil (kg/cubic m). Bubenzer and Jones (1971) developed an expression for the soil erosion due to impact action of raindrops as

$$S_p = m\,(2.78*10^{-7}\, I)^{\alpha}\, k_e^{\beta}\, P_c^{-\delta} \qquad (5)$$

where S_p is the soil detachment rate (kg/sq. m-sec), I is the rain intensity (mm/hr), k_e is the total kinetic energy of rain drops (J/sq. m), P_c is the percentage of clay in the sediment, $m = 1.5 - 3.0$, $\alpha = 0.25 - 0.55$, $\beta = 0.83 - 1.49$, $\delta = 0.40 - 0.60$ and D is rainfall depth (mm). The expression for the kinetic energy of the raindrops is obtained as

$$k_e = 24.16D + 8.73\, D \log(I/25.4) \qquad (6)$$

The sediment continuity equation is by itself not sufficient to solve for the two unknowns c and y_s appearing in equation (4). We obtain another relation by considering a first-order reaction equation as proposed by Foster and Meyer (1975)

$$D_r = \sigma\,(T_c - q_s) \qquad (7)$$

where D_r is the erosion or detachment rate (kg/m-sec), σ is the first-order reaction coefficient, T_c is the flow transport capacity (kg/m-sec), q_s is the sediment load (kg/m-sec). Equation (7) states that the erosion (or deposition) rate D_r is directly proportional to the difference between the sediment transport capacity and the sediment load at any time-space location. Therefore the surface flow erodes at a maximum rate when there is no sediment load. The coefficient σ in equation (7) is determined by

$$\sigma = \frac{D_{rc}}{T_c} \qquad (8)$$

where Drc is the erosion capacity of the flow (kg/m-sec). The flow transport and erosion capacities are defined as follows.

$$T_c = C_t\,(\tau - \tau_{cr})^{1.5} \qquad (9)$$

$$D_{rc} = C_d \tau^{1.5} \tag{10}$$

where $C_t = 0.001 - 0.5$, $C_d = 0.004 - 0.8$, τ_{cr} is the critical shear stress (kg/sq. m), τ is the unit tractive force over the flow bed (kg/sq. m). The tractive force on the flow bed is given by

$$\tau = \gamma y S_0 \tag{11}$$

where γ is the specific weight of water (kg/cubic m).

Equations (1) to (7) therefore describe the interrelated processes of overland flow and sediment transport through the processes of erosion and deposition. The time-space evolution of the sediment surface is obtained through a simultaneous solution of the flow and sediment transport equations.

DISCUSSION OF RESULTS

A numerical centered implicit finite difference technique was used for the solution of both the flow and sediment transport equations. We first compare the performance of this physics based model with the laboratory results of Singer and Walker (1983). Their experimental set up consisted of packed soil (bulk density =1200kg/m^3) on a 3.0m by 0.55m flume.

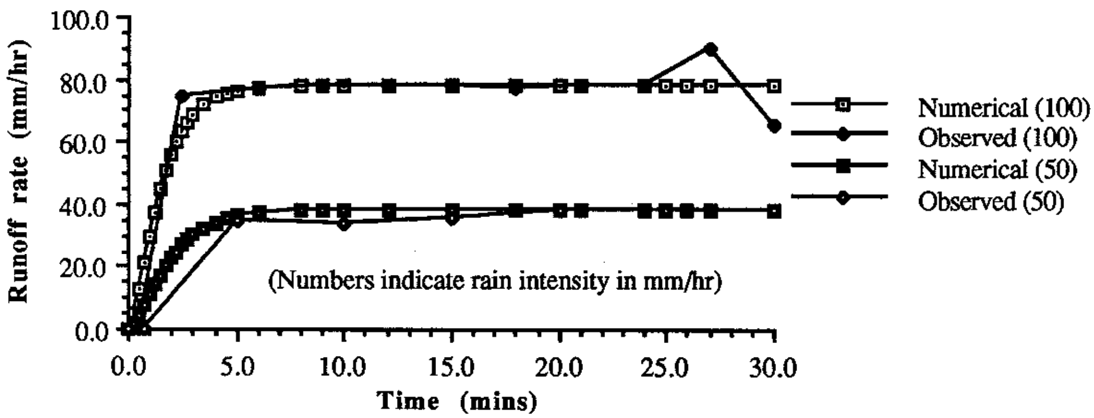

Fig. 2. Numerical and observed (Singer and Walker, 1988) outflow discharge hydrographs for rain intensities of 50 and 100 mm/hr.

Figure 2 shows experimental and numerical results for the flow discharge for two different rainfall intensities of 50 and 100mm/hr. (Some of the water was lost due to infiltration). The duration of rainfall and the experiments were restricted to 30 minutes. Both of these experiments were carried out on a 9% sloping bed. Figure 2 shows that the numerical results are able to reasonably model the flow dynamics of overland flows.

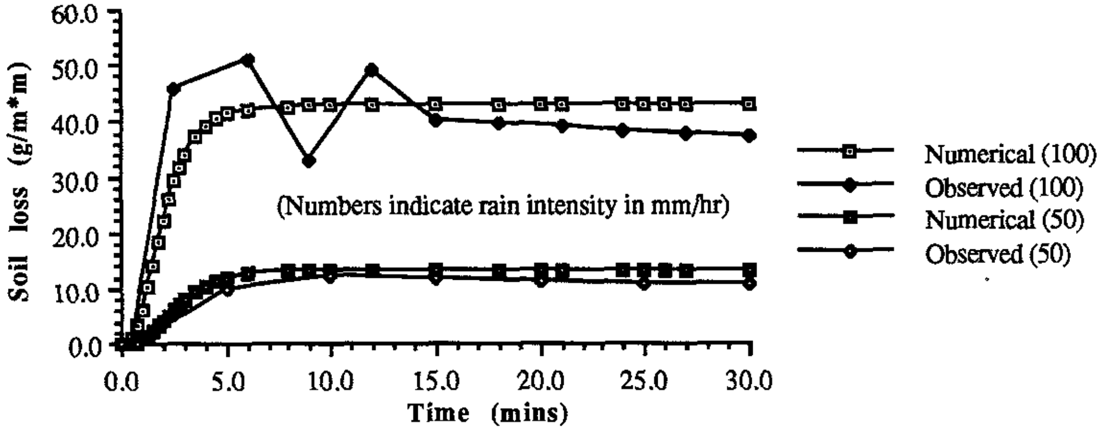

Fig. 3. Numerical and observed sediment discharge profiles (Singer and Walker, 1983) for rain intensities of 50 and 100 mm/hr.

Corresponding to these overland flow discharges, Figure 3 shows the observed and numerically simulated sediment discharges as a function of time. For results in Figure 3, the raindrop impact was assumed to have negligible effect on the sediment transport dynamics. Figure 3 indicates that the physics-based model presented here can be used for modeling the dynamics of sediment transport. Since the numerical model is specially designed for steep, long slopes, it is expected to perform well on actual hillslopes. We also show some results indicating the sensitivity of the sediment discharges or concentrations to various parameters.

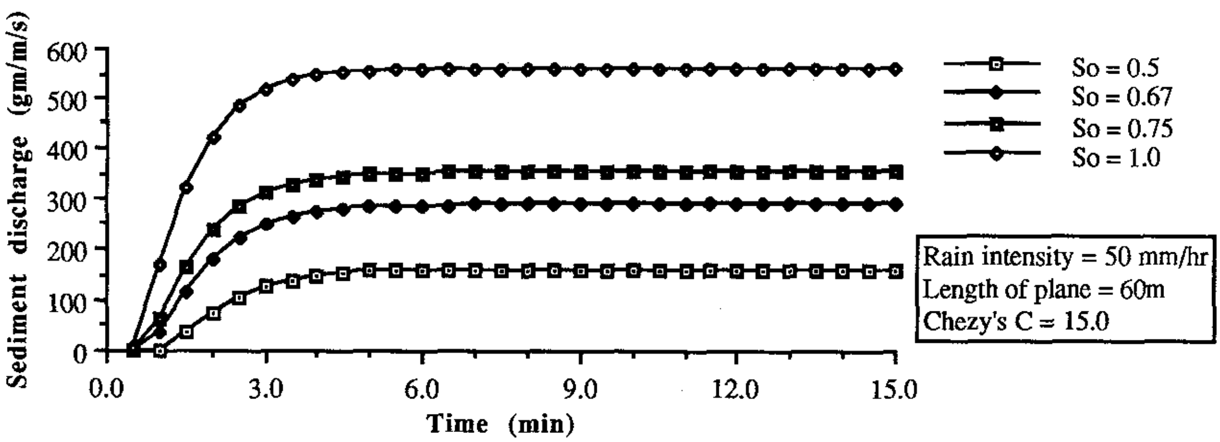

Fig. 4. Sediment discharge rate over the flow region vs time for different slopes S_0

Figure 4 shows the sediment discharge rate at the outflow end of a 60m flow section as a function of time for different slopes. The sediment discharge increases with increasing slope in this figure. The discharge rate achieves a constant value in about 5.0 minutes for all the cases shown in Figure 4, and so the cumulative sediment discharge increases almost linearly after 5.0 minutes.

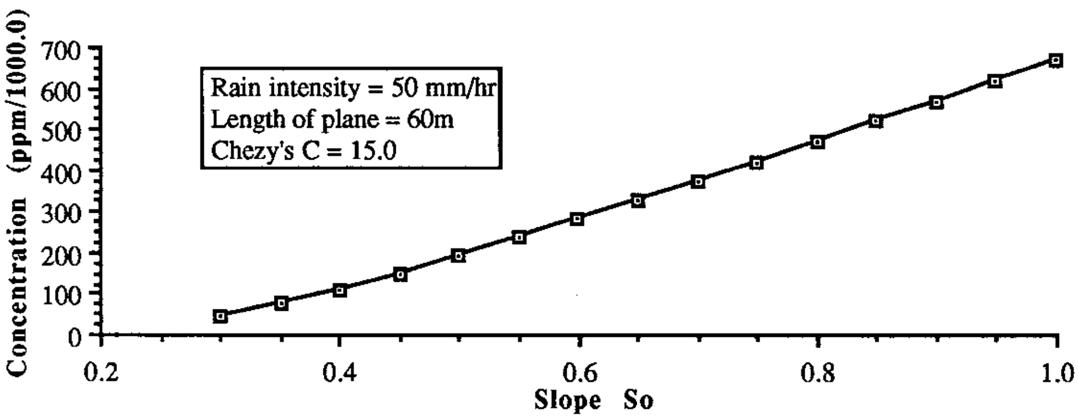

Fig.5. Steady-state outflow sediment concentration vs slope

Figure 5 shows the steady state sediment concentration at the outflow section of the 60m plane. The concentration of the sediment at the outflow section also increases with increasing slope. As the slope increases the velocity of water on the soil surface increases. Figure 4 shows that the sediment discharge increases with increasing slope. Since the steady state flow discharge is the same in both Figures 4 and 5, the concentration of sediment must increase with increasing slope as indicated in Figure 5.

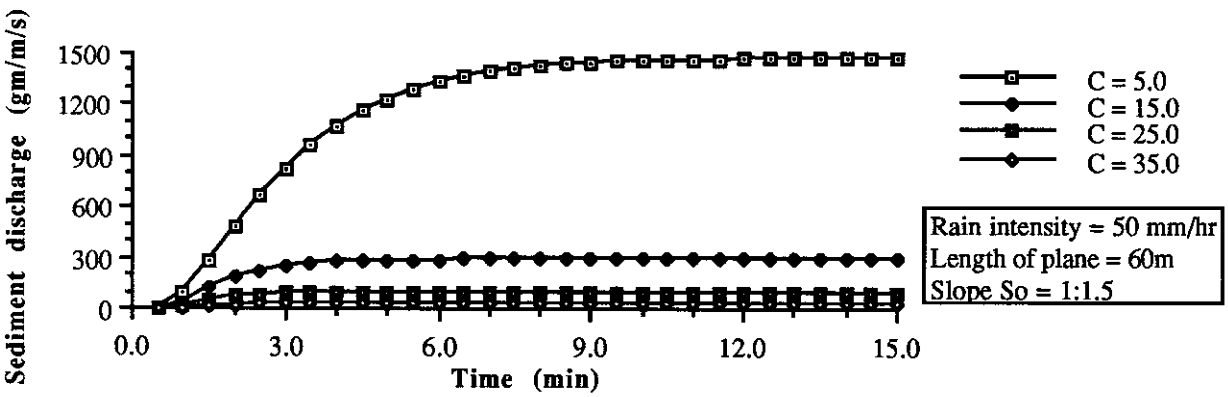

Fig. 6. Sediment discharge rate over the flow plane vs time for different surface roughness values (C = Chezy's coefficient)

Figure 6 shows the behavior of transported sediment when subjected to different soil roughness parameters as indicated by Chezy's C value. With increasing surface roughness (i.e. decreasing C), the amount of sediment transported also increases. Once the sediment discharge rate reaches a constant value, there is a linear increase in the cumulative transported sediment material over the whole plane with time. Figure 7 shows the steady state sediment concentration at the outflow section of the overland flow plane at 60m as a function of roughness. Figure 6 indicates that the concentration of sediment must increase with decreasing Chezy's C. This is confirmed in Figure 7 which shows an increase in the sediment concentration with increasing surface roughness.

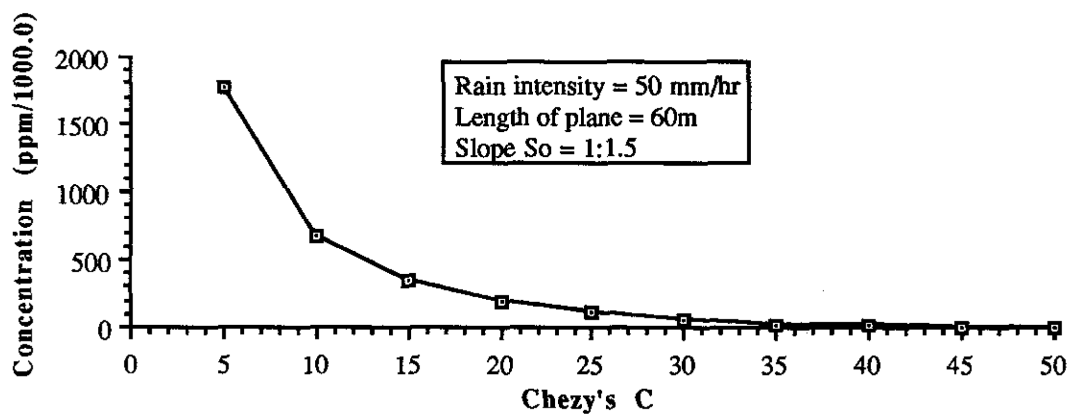

Fig. 7. Influence of surface roughness on steady-state outflow sediment concentration

Equation (9) of this paper indicates that the overland flow must develop enough tractive force to dislodge soil particles. This impending shear force is the critical shear stress which depends on soil composition and structure. Smerdon and Beasley (1959) developed an empirical relationship between the critical shear stress (N/m^2) and the clay content in the soil.

$$\tau_{cr} = (0.493)10.0^{0.0183.Pc} \qquad (12)$$

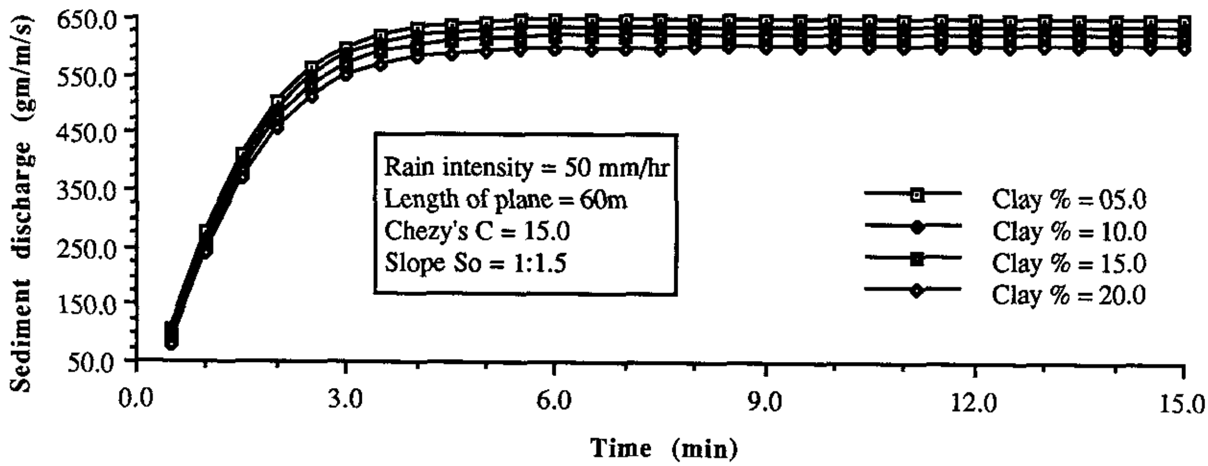

Fig. 8. Sediment discharge rate over the flow plane vs time for different clay percentages in the soil

Figure 8 examines the effect of clay percentage in the soil on the transport of sediment as a function of time. The clay within the soil binds it together thereby increasing its resistance to erosion by overland flow as reflected in equation (12). Therefore, increasing clay percentage in the soil decreases the sediment discharge as indicated in Figure 8.

SUMMARY AND CONCLUSIONS

We have presented a physics-based numerical model for the conjunctive modeling of water flow and sediment transport over planar land sections. A kinematic wave approximation has been utilized to model the flow dynamics. This model is particularly well suited for steep slopes as those found adjacent to highways in Shasta County in Northern California. The sediment transport model utilizes a continuity equation and a first order reaction. The rate of sediment transport is proportional to the difference between the transport capacity and the sediment load. The constant of proportionality depends on the transporting and eroding capacity of the flow and the critical shear stress of the soil in resisting erosion. Comparison with laboratory results validate the applicability of this model to physical situations. The results presented show the dependence of sediment concentration and discharge on factors like slope, surface roughness and clay content of the soil. An increase in surface roughness and slope cause an increase in the amount of transported sediment material but the sediment discharge falls down with increasing clay in the soil. Soils with high clay percentage are more resistant to erosion by the tractive forces exerted by the overland flow due to their high critical shear stress. Equation (5) indicates that clayey soils are less prone to detachment by rain drop impact also.

Field studies are now being conducted on the freshly cut decomposed granite hillslopes at Buckhorn Summit in Shasta County of Northern California. This model will be used to predict the sediment concentrations and discharges at this site. These studies will provide us with a better understanding of soil movement due to rainfall and help in the formulation of more effective abatement strategies.

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STREAMBANK EROSION: PROBLEMS, CAUSES, AND SOLUTIONS

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ABSTRACT

The Kenai River, which drains 2,200 square miles in southern Alaska, is the most heavily used freshwater fishery in Alaska. Streambank erosion along the Kenai River is threatening homes and is destroying fishery habitat. The streambank problems are caused by: bare slopes because vegetation was destroyed by construction or river access; toe undercutting of streambank materials; outside curves; overhanging vegetation causing too much weight; sloughing and slumping of unstable soils and strata; south facing slopes thawing first and being more susceptible to streambank erosion; boat wakes; and fisherman foot traffic collapsing streambanks.

Land-use decisions are being made without knowledge of the suitabilities or limitations of particular parcels or the impacts of specific activities. As a result, many homes have been built too close to eroding streambanks, and streamside vegetation is being destroyed for access and for boat docks, with associated erosion. In addition, some developments are experiencing undesirable side-effects such as septic tank effluent in permeable soils contaminating wells and leaching to the Kenai River. A Soil Conservation Service (SCS) inventory of the streambanks is evaluating geomorphic surfaces, as well as streambank height, position, slope, stratigraphy, texture, strength, vegetation, and cultural features. This is being used to evaluate specific site potential for streambank failure. This inventory data is supplemented with other data on streambed material, near shore velocity, and comparisons of streambank loss over time. From this data set, an array of potential solutions for various stream reach conditions is being recommended.

INTRODUCTION

Purpose and Scope

The SCS initiated a streambank erosion study along the Kenai River in Alaska in October 1989 (Reckendorf, 1989). A special report concerning that work characterized the streambank erosion that occurs throughout the entire Kenai River and presented site specific inventory information for a variety of locations. This paper summarizes the study, as supplemented by May 1990 detailed work along seven miles of the Kenai River,through Soldotna, Alaska.

Description of Study Site

The Kenai River is a glacially fed stream located in the Kenai Peninsula in southern Alaska. The watershed drains 2,200 square miles, flowing 17 miles from Kenai Lake in the Kenai Mountains to Skilak Lake, and 50 miles from Skilak Lake to its outlet in Cook Inlet.

Salmon spawn throughout the river's length and in many tributaries. The river is exceptional in supporting two distinct spawning runs of Chinook, Coho, and Sockeye Salmon.

Regulations have reduced the size of boat motors that can be used on the Kenai River, and the reduced boat wakes and wave action has reduced wave erosion of the streambanks in some areas. However, many significant erosion areas still exist (Reckendorf, 1989).

THE STUDY

Streambank Erosion Causes

Streambank erosion along the underfit Kenai River is caused by natural causes as well as being induced by human activities.

Natural streambank erosion is primarily that which occurs along the outside curves of the meandering Kenai River, especially where the river impinges along naturally unstable soils and the underlying unconsolidated material. This condition is worse where vegetation has naturally died out or been removed. Work in May 1990 indicates that streambanks composed of poorly graded gravels, are generally more erodible or undercut than the streambanks of silty sand or well or poorly graded sands. In addition, the cobbly gravels (GP) within three feet of the low water line, along the low and high floodplain, and Terrace One, are the most erosive units along the Kenai River.

On exposed slopes that are devoid of vegetation, streambank erosion has usually undermined the toe (slope) position. Removal of a toe material activates the upslope material to move by slumps, creep, or other mechanism. Movement downward of the sloping streambank material is highly dependent on moisture content and the maximum angle that the slope will support (angle of repose).

The thawing of soils and underlying materials on south slopes, makes natural south slopes generally more susceptible to Kenai River streambank erosion during high water runoff stages, especially when ice is present in runoff. Natural freeze-thaw also contributes to loosening and sloughing. In addition, fluctuating tidewater restricts vegetation establishment, especially on steep streambanks, and subjects the streambank to the natural diurnal susceptibility of the soils and underlying materials to river erosion and wave attack.

Human induced streambank erosion along the Kenai River is expressed by removal of vegetation for access, views, and the building of stairs and docks, and the associated trampling. This makes the riverbank materials more susceptible to streambank erosion processes, particularly where the soils and underlying materials have little resistance to tractive and seepage forces along the riverbank. If the unconsolidated material in the streambank is gravelly with a matrix of fines and sand, the fines and sand are constantly being washed out by repeated wave attack. The result is a collapse of the gravel fabric and then the collapse of the overlying material that has been undermined. This is a common occurrence between low water and about three feet of height. The root mat prevents the surface soil from being removed immediately so the streambank may have an irregular overhang from the top of the bank. Another area of human induced streambank erosion occurs along the boundary of streambank structures (riprap, wood, metal, etc.). This is differential erosion, with the structure serving as a hardpoint and with energy being deflected to the materials along the margin. The adjacent unstructured material is generally less stable.

Rate of Stream Erosion

Scott (1982) used aerial photography dated 1950 to 1977, and later field reconnaissance to conclude that erosion rates were constant between 1950 and 1977, but increased after 1977. Scott concluded that there were local areas above River Mile (RM) 39.4 and below RM 17.6 where low banks were eroding at rates up to 5 feet per year. However, his studies indicated that the general rule was erosion rates of less than 1 foot per year. Inghram (1985) did a follow-up aerial photo comparison of bank erosion at selected sites studied by Scott and concluded that most of the streambank erosion retreat was progressing at a geologic background rate.

Some changes in the streambank have been photo documented (Reckendorf, 1989). Photographs taken in May 1989 were compared with photos taken in October 1989. The highest flood flows in 12 years occurred in September 1989, so the photo changes of the streambank reflect high runoff (8 year or 12.5 percent chance event), without river ice. In general, the flood had a stage of up to 2.5 ft. deep on the low floodplain, and to the top of the edge of the bank of the high floodplain.

Along Centennial Campground, RM 20.5, a low terrace (T1) streambank with no vegetated cover was compared between May 23, 1989, and October 7, 1989. Only about two feet of loose material along the toe of the slope was removed with little evidence of streambank retreat of the cemented gravelly material.

Changes in one area previously studied by Scott (1982) and Ingram (1985) have also been documented (Reckendorf, 1889). This is the 3H curve at RM 16.7 - 15.3. Scott (1982) pointed out a continuing problem of streambank erosion along this cutbank because of boat wakes that he noted ran 3-4 feet up the loose gravel bank. Inghram (1985) noted that aerial photographs of this river curve appeared to be unvegetated, and show a natural unstable outside curve as far back as 1950. An evaluation at the upper end of the 3H curve (RM 16.2) showed the importance of toe protection by vegetation and debris for poor strength bank materials. The material in the streambank near the toe varies from poorly-graded to well-graded sand with low moist shear strength (Torvane Shear 0.5 - 1.5 tons/sq.ft.). Naturally accumulated trees with root wads, lay parallel to the base of the slope in front of the poor strength materials. There was essentially no change in the streambank erosion between May and October of 1989. However, at RM 15.8, which was in the middle of the outside curve, about 6-8 feet of loose material at the toe slope was removed along with about 2 feet of unconsolidated , well graded to poorly graded sand. In addition, some of the reach between RM 16.2 and RM 15.8 on the outside curve was very resistant, cemented, well graded sand with moist (Torvane Shear) of > 4.5 tons/ft.2. These very resistant streambanks had 1/2:1 to vertical slopes, and average annual erosion rates of <0.1 ft./yr.

Other Streambank Erosion Effects

Access along the streambanks of the Kenai River has caused trampling of vegetation and bank slumping. Extensive near shore sedimentation is produced by wave action on the slumped blocks of the trampled floodplain and this sediment intrudes into the matrix of gravelly bed material. This sediment intrusion is expected to essentially block the pore space for swim up salmon fry that may have spawned in these gravels as shown in other studies (Reckendorf and VanLiew 1988, 1989).

Past Streambank Erosion Treatments

A large variety of materials and methods have been used for streambank protection along the Kenai River. These have resulted in good to poor long-term protection. In some locations, large (>1.5 ft.) blocks of rock riprap or large cobbles have been hand placed along low floodplain streambanks and have stabilized the reach. However, along many areas, rounded gravel with a d50 of less than 6 inches has been placed only to be washed away by subsequent high water and wave attack. This has been especially true at the outside curve locations. A variety of materials other than rock have been tried such as tires, barrels, single large logs, wood boards, crib walls, and sheet piles. In addition, small trees with branches have been pinned or roaped to streambanks, and one slope revegetation with a grass mixture has been tried. This grass slope treatment showed little erosion or sloughing above high water level for up to two years after treatment. However, there was some toe slope undercutting. Because of the lack of maintenance fertilization and reseeding, few of the introduced plants were apparent at the beginning of May 1990, the third growing season.

Streambank Erosion Inventory Procedure

Table 1 shows the streambank erosion inventory form used in the study, with example sites characterized. Table 2 provides the legend for the inventory form. The inventory reaches discussed were assigned to particular geomorphic surfaces, which represent episodes in landscape development. This places all locations described in a geologic time, reference height, and stratigraphic material connotation for reference. Because of site characteristics, it was not possible to inventory every item in the legend in every reach. Six pebble counts were also taken of the stream bed using the procedure of Wolman (1954.)

Recommended Treatment

The Alaska Division of Parks and Outdoor Recreation (ADPOR) and the Alaska Department of Fish and Game (ADF&G) are the permitting agencies for streambank protection measures. Approved permits stipulate that staked locations of riprap slopes must be reviewed and approved by the Alaska Department of Fish and Game (ADF&G), Habitat Division, prior to beginning work. In addition, there are permit provisions that riprap follow existing bank contours as well as riprap minimum sizes, for riprap placed below high water line. Permits also specify that the streambank above the ordinary high waterline shall be revegetated with naturally occuring woody vegetation to provide cover for juvenile fish and to provide additional bank protection. Vegetation is to consist of those species identified in the Field Guide for Streambank Revegetation (ADNR, 1986) which is provided to permit applicants. The ADF&G is concerned with any structures that cause near shore velocity to increase over 2 ft./sec., which they feel is the maximum velocity that fry can withstand in their upstream migration to Kenai Lake to over winter.

The SCS inventory is providing information for identifying types of streambank conditions that could be be left alone or treated. The inventory information is also being used to develop conceptual job sheets and standard drawings. Conceptual treatments are grouped by geomorphic surface and stratigraphy. There will be the low bank treatment (primarily low floodplain and high floodplain from about 0-7 feet in height to be treated), an intermediate streambank treatment for low terraces (between 0 and 20 feet in height to be treated), and a high streambank treatment (between 0 and 50 feet of the bank is to be treated). There will also be non tidal as well as tidal treatments. The array of treatments such as riprap, gabbions, logs, cabled or buttressed trees, geotextiles, or revegetation will be recommended for the various site conditions. In all cases, toe protection needs to be provided, especially for the contact zone between the gravel that occurs between low water and about three feet of height and the overlying gravels or sands.

There is a river bed cobbly gravel which has an average size of six to nine inches in the middle reach of the river (RM35 - RM20, Scott, 1982). The SCS inventory indicates that the cobbly bed should be considered to be stable and non-mobile under the present flow regime of the Kenai River. This coarse cobble armored bed is also considered to be related to Pleistocene outwash discharges much higher than exist today (Scott, 1982). In many locations along the Kenai River a smaller gravel with a d50 of about 2 inches reflects the gravel bed of the present hydrology regime of the river, and the associated sloughing gravel along streambanks. This overlies the coarse cobble of the paleo-hydrology regime. All SCS conceptual treatments will recommend toe material excavation down to the level of the non-mobile cobbly material.

CONCLUSION

The principal conclusion that can be drawn from the SCS study is that there are particular strata in streambanks of certain geomorphic units that are most prone to streambank erosion from high flows or wave attack. The most unstable materials are coarse gravel with a sandy matrix that commonly occurs near the toe of streambank slopes of the low terrace (T1), high floodplain, and low floodplain. These are the lower geomorphic surfaces which are most prevelant along the Kenai River, and which receive the most pressure for development because of better potential access. The sands and fines in the gravel matrix wash out, and the gravel sloughs downward. This action undermines the overlaying material which tends to collapse with a vegetation overhang. Frequent wave action from boat wakes is probably the most significant cause of the removal of fines and sand from the gravel matrix once vegetation is removed. The wave action occurs much more often than the high water flow condition, which also causes the washing out of the fines and sand from the gravel matrix.

The access pressure for streamside fishing has locally trampled and increased riverbank slumping, particularly along the high and low floodplain and Terrace T1. In addition, riparian habitat that provides shade and rearing habitat is destroyed. The fines and sands in the slumps are disaggregrated by the wave action of boats and are dispersed over the near shore bed material. This causes extensive infilling of the coarse bed material and detrimental effects to any of the bed material that has been used for spawning. This infilling also produces near shore imbedded bed material that is much less favorable habitat for future salmon spawning.

Riprap and other structural materials have had a variable effect on streamside habitat, velocity, as well as increased streambank failure. A lot of the problem relates to the type of materials (barrels, boards, & tires) used in structural measures and/or a too small size (i.e., < 6-9 inch d50) used for riprap. The small riprap sizes result in a smoother surface along the streambank than would occur under natural conditions, and associated higher streambank velocities. The use of other inappropriate materials (boards, etc.) for a particular site also results in smooth surface high velocities and, in addition, these materials require the need for much repair and increased streamside and associated river bed disturbances. Velocities over 2 feet/sec. are considered by Alaska Department of Fish and Game to be detrimental to the upstream migration of fry to Skilak Lake to over winter. Near shore velocities over 2 ft./sec. primarily occur at the natural outside bends in the river and along smooth surface installed measures.

The SCS streambank inventory (Tables 1 and 2) provides resource information that can be used to select the most appropriate streambank measures for the various site conditions described. Treatment recommendations will be separated primarily by geomorphic surfaces (low floodplain and high floodplain; Terrace One; Higher Terraces); streambank and toe stratigraphy and texture; streambank position (inside, crossover or outside curve); streambank vegetated cover; and streamside velocity. It is anticipated that field trials will be conducted to test the proposed SCS conceptual designs for streambank protection.

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Example
TABLE 1

Kenai River Streambank Inventory

F. Reckendorf
J. Louthan
INVESTIGATOR C. Inman Date 5/30/90
NT (nontidal) or tide at sampling NT

River reach (river mile, side of bank and length)	Geomorphic surface	Surface condition	Streambank height (from "toe")	Streambank position	Graphic	Slope (by strata)	Strata material	Torvane (by strata)	Strata stability	Slope cover	Toe width	Toe texture	Large woody debris	Bed material (d50 w/in 10 ft)	Bed material (d50 beyond 10 ft)	Velocity	Velocity location	Cultural features	Houses (number and setback)	Comments
R-9 RM 23.6 (680')	T2	wv	D	lu		70% / 15%	SM / GP	— / —	st / ov	t / sh	0-e	GP	oc	—	—	3 ft/sec	D3	—	—	—
R-10 RM 23.5 (1520')	T1	wv / tb by houses	C	lm		60%	SM-SW / GP	— / —	st to ov to hm	t / sh / g	e	GP	—	10cm	15cm	2 ft/sec 2.9 ft/sec	D2 D3	bd,rg & RR	4,cl	Includes 200' strip with HFP in front of Tl, GP – Toe
L-11 RM 20.6 (760')	T1	tb	C	l		80%	SM / GP	— / —	ov / ls	very sparse g & h	n	GP	oc	10cm	—	2.5 ft/sec 3.0 ft/sec	D2 D3	foot paths	—	Extensive trampled causing loose sloughing
L-17 RM 19.5 (720')	T3	tb	E	O		80%	SM / GP	— / —	ov / ls	t / sh / ln	e	GP	many ln & sr	7-10 cm	—	2.1 ft/sec 2.1 ft/sec	D2 D3	wd	few sb	

TABLE 2

Kenai River Streambank Inventory Legend

Geomorphic surface	(LF) low floodplain; (HF) high floodplain; (T1) terrace 1; (T2) terrace 2; (T3) terrace 3; (MO) moraine
Surface condition	(wv) well vegetated; (vt) partially vegetated and trampled; (tb) trampled and bare or almost bare; (uv) naturally unvegetated
Streambank height from toe	(A) 0–4 ft; (B) 4–7 ft; (C) 7–20 ft; (D) 20–50 ft; (E) >50 ft
Streambank position	(I) inside curve (O) outside curve (X) crossover modifiers: u=upstream end of reach, d=downstream end of reach, m=mid-point (highest velocity position within curve reach)
Graphic	Draw approximate bank cross-section showing shape, slope breaks, relative thickness of strata, etc. The following five data categories should correspond with sketch.
Slope (by strata)	(V) vertical; (1) vertical 1:1 (100%); (2) 1:1 to 2:1 (50%) (3) 2:1 to 3:1 (33%); (4) 3:1 to 4:1 (25%); (5) >4:1 (25%)
Strata material	(U) unstratified; (GW/GP with sand), etc., Refer to National Soils Handbook (NSH Table 603–1); Other modifiers: Sd = sandstone, gr = gravelly, cm = cemented, cb = cobbly, om = organic matter
Torvane (by strata)	Measure vane stress with Torvane, record in Tons per sq. ft;
Strata stability	(st) stable; (ps) partially stable; (ls) loose sloughing; (bl) block slumping; (ov) overhanging; (hm) hummocky; (sl) slumping
Slope cover	(t) trees; (s) shrubs; (h) herbaceous; (g) grass; and (b) bare; and use modifier pb = partially bare (give % bare)
Toe width	(0) no toe "floodplain"; (e) edge, <5 ft; (n) narrow, 5 ft – 10 ft; (m) medium, 10 ft – 50 ft
Toe texture	use same material codes as Strata material (NSH 603–1)
Large Woody debris	(ln) leaners; (sr) sweepers; (oc) occasional large woody debris
Bed Material d50 w/in 10 ft	Estimate d50 in cm or d50 in cm from pebble count
Bed material d50 beyond 10 ft	Estimate d50 in cm or d50 in cm from pebble count
Velocity	(VI) <1 ft/sec; (V2) 1–2 ft/sec; (V3) 2–3 ft/sec; (V4) >3 ft/sec
Volocity location	(D1) 2.5 ft from toe; (D2) 5 ft from toe; (D3) 10 ft from toe
Cultural features	(st) stairs; (RR) large riprap; (rr) small riprap; (tr) tires; (wd) wood cribbing or boards; (cn) concrete wall; (rg) rock groins; (ca) canals; (bd) boat dock; (tv) tree revetments; (ot) other
Houses (number and setback)	(cb) on bank; (vc) very close within 10 ft of bank; (cl) close within 10 – 50 ft of bank; (sb) set back 50 – 100 ft of bank; (mt) more then 100 ft from bank

THE GEOMORPHIC APPROACH TO CHANNEL INVESTIGATIONS

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ABSTRACT

Successful channel modification projects depend on integrating the design with the existing fluvial system. Geomorphology provides principles and techniques for investigating fluvial systems. Geomorphic investigations reveal the state of equilibrium in the system. Studies in the present time frame, coupled with historic considerations, permit predictions of future responses of the channel network. Several examples of channel modifications designed by the Soil Conservation Service illustrate the geomorphic approach.

INTRODUCTION

Previous attempts at channel modifications by the Soil Conservation Service (SCS) sometimes either led to failure or to expensive overly conservative designs (Erion, 1991; Harvey and Watson, 1988). This dictated changes in the SCS approach to channel investigation and design. The resulting geomorphic approach evolved over the past decade. This approach addresses the multiple objectives of design performance, stability, cost effectiveness, and environmental compatibility as advocated by Williams (1990).

This paper presents the conceptual framework for making geomorphic investigations of channels and their watersheds. It incorporates some of the classical as well as more recent principles and methods of geomorphology (Erion, 1991; Lane, 1955; Leopold et al., 1964; Schumm et al., 1984; SCS, 1977; 1979; Waldo, 1983a; 1990; 1991). It suggests approaches to channel stability evaluations, a subject discussed in greater detail by Erion (1991) and Waldo (1991).

THE GEOMORPHIC APPROACH

Geomorphology encompasses a variety of subjects involving a number of disciplines. Geomorphic investigations therefore require streamlining to obtain results in a cost efficient and timely manner. Adherence to the basic scientific approach to problem solving achieves this result. An efficient geomorphic investigation of a channel modification project proceeds along the following lines.

1. Identify the problems the study must address.

2. Propose an hypothesis concerning the geomorphic attributes related to the problems.

3. Select methods of testing the hypothesis.

4. Collect data.

5. Determine test criteria.

6. Perform tests and/or conduct experiments.

7. Analyze results of tests and experiments, comparing to test criteria.

8. Draw conclusions and make interpretations and decisions.

Problem identification constitutes the first step in the investigation process. Problems associated with flooding, erosional or depositional stability, irrigation, and drainage may require channel modifications. Geomorphic investigations pertain to all types of problems, but those associated with channel stability are the object of this study.

THE GEOMORPHIC HYPOTHESIS

The geomorphic hypothesis consists of a tentative statement regarding the degree of stability in the fluvial system. The hypothesis provides a focus for the study by suggesting which analytical procedures should be used and by dictating data needs. Data are then collected and analyzed, and the hypothesis either accepted or rejected. The quantitative basis for making the decision provides guidelines for establishing geomorphic thresholds and proposing design criteria.

The choice of a geomorphic hypothesis depends on the results of a field reconnaissance and on the objectives of the study. Candidate hypotheses generally include 1)dynamic equilibrium; 2)dynamic disequilibrium; 3)mass balance; 4)dissipation or minimization of energy; and 5)dynamic maturity.

The hypotheses of dynamic equilibrium and disequilibrium state that the channel system or reach in question demonstrates the specified condition. The conditions of equilibrium and disequilibrium are described in the subsequent section of this paper.

Erion (1991) discusses the minimum rate of energy dissipation and its expression in the form of Lane's relationship (Lane, 1955). This relationship also serves as the basis for mass balance considerations. The hypothesis of mass balance states that, for a given reach of channel, the amount of water and sediment discharged from the downstream end will equal water and sediment introduced at the upstream end plus water and sediment introduced from valley walls and tributaries entering the reach minus sediment and water lost to the channel bed, banks, and overbank flooding.

The hypothesis of dynamic maturity states that the element of the landscape in question (e.g., a stream channel) has evolved to a mature configuration. Although not in general use, this approach has been applied to channels with comparatively small drainage basins (Waldo, 1983a; 1983b; 1990).

EQUILIBRIUM AND DISEQUILIBRIUM IN CHANNELS

The concept of dynamic equilibrium, or simply, equilibrium, is central to geomorphic analysis. A state of equilibrium exists in a channel when work done approximately balances the forces imposed (Leopold et al., 1964). Adjustments occur that maintain this approximate balance even as the channel network lowers the surrounding landscape, or uplift rejuvenates the terrain, or as geomorphic processes and their rates alter with changing climate. The adjustments may produce locally noticeable effects but usually do not disrupt the entire system.

The mean annual flow in perennial equilibrium channels in humid regions generally occupies about 1/3 of the channel depth (Leopold et al., 1964). The recurrence interval of the bankfull stage varies from about 0.5 to 5 yr., typically ranging from 1 to 2 yr., and averaging about 1.5 yr. Eroding banks occur on the outside of some meanders and generally not elsewhere. Mature vegetation covers the banks particularly in straight reaches. Beds remain comparatively stable with oversteepened zones underlain by bedrock or armoured by boulders. Islands and point bars demonstrate long term stability and may support vegetation. The planform may be straight, meandering, or braided. It may change from reach to reach, but generally remains the same within a reach for several years to decades or longer. A few isolated or localized groups of meanders may migrate or neck without completely disrupting the system.

General disequilibrium exists when adjustments cannot occur quickly enough to offset the changes imposed, and the channel system will initiate a prolonged or complex episodic response to restore equilibrium to the system (Schumm et al., 1984). The effects resulting from disequilibrium seldom distribute suddenly and uniformly throughout the fluvial system. Rather they consist of localized degrading or aggrading, widening or narrowing, and laterally migrating or avulsing reaches which migrate through the fluvial system sporadically or episodically over time. The system requires a finite period, or lag time, to readjust to a new equilibrium state.

Disequilibrium results in aggradation, degradation, or planform changes. Such disruptions may coexist throughout a given fluvial network and migrate through the network over time. Frequent overbank flooding and bankfull stages with recurrence intervals <1 yr. characterize aggradational disequilibrium. Specific gage studies (Robbins and Simon, 1983) indicate a rising bed elevation. Recent alluvial deposits occur in the bed and may be masked by vegetation in humid regions. Planform may remain the same, or in some instances meandering channels may change to straight or braided forms.

Degradational disequilibrium frequently results in deepened and/or laterally expanding channels of Types II and III (Schumm et al., 1984, p. 128). The recurrence interval of the bankfull stage generally exceeds 2.5 yr. Bank erosion occurs on both sides of the channel. Active slumping of banks occurs, as does mass wasting of valley walls immediately adjacent to degrading reaches. Specific gage studies indicate a declining bed elevation. Migrating overfalls, knickzones, and headcuts dissect streambed deposits, undermine banks and confluences, and create hanging tributaries. Accelerated incision and/or migration of meanders occurs at numerous locations. Straight or braided channels may or may not evolve to a meandering form.

TESTING THE GEOMORPHIC HYPOTHESIS

Numerous techniques exist for testing the hypothesis. Methods pertinent to channel stability investigations include downstream analysis of hydraulic and geometric relationships, minimum variance theory, analysis of channel form evolution, sediment transport relationships, sediment budget analysis, geomorphic thresholds, safety factor analysis (examines relation between forces promoting and resisting failure of the landscape by techniques such as allowable velocity, tractive stress, tractive power, and approximate analysis of landforms), comparative analysis (quantitative comparisons between similar channels), space-for-time substitution, stream order analysis, and geologic evaluation mapping. Physical and/or mathematical models of the channel and its watershed are advantageous in many situations. A given study will generally employ several methods depending on the hypothesis selected and study objectives.

Several geomorphic procedures exist for examining the hypothesis of equilibrium. Downstream analysis (Erion, 1991; Leopold et al., 1964; Waldo, 1991), safety factor analysis (SCS, 1977), and sediment transport analysis (SCS, 1979; Vanoni, 1975) are frequently utilized. Minimum variance theory has been applied in several instances (Williams, 1978). Analysis of geomorphic thresholds (Schumm et al., 1984) has gained popularity in recent years. A geomorphic threshold consists of the conditions of materials, landform, climate, and other landscape components above or below which the rates of a given geomorphic process or group of processes will dramatically accelerate or decelerate. Stream order analysis (Leopold et al., 1964) quantifies many useful relationships in equilibrium fluvial networks. Comparative analysis (Waldo, 1983a; 1990) can compare the states of equilibrium between two or more channels with similar watershed characteristics.

Analysis of the evolution of channel form during an episode of disequilibrium constitutes the primary method of evaluating the disequilibrium hypothesis. Schumm et al. (1984) discuss methods of quantifying channel form evolution. Downstream analysis, space-for-time substitution (Schumm et al., 1984), and sediment budget (Waldo, 1988), comparative, geomorphic threshold, and safety factor analyses supplement form evolution analysis.

Sediment transport analysis, the primary method of investigating the mass balance hypothesis, represents an approach commonly used by the hydraulic engineering profession to evaluate channel stability (SCS, 1979; Vanoni, 1975). Sediment transport studies must be integrated with a sediment budget analysis (SCS, 1979; Waldo, 1988) in order to properly evaluate channel stability. Downstream, safety factor, and stream order analyses also play an important role. Geologic evaluation mapping (GEM) provides an efficient means of conducting the sediment budget and other necessary studies. GEM consists of a map, or more usually a suite of maps, that conveys geologic information in a practical manner. For example, a GEM might identify the contributions of portions of a watershed to sediment deposits in a lake (Waldo, 1986).

Downstream analysis, minimum variance theory, and sediment transport analysis provide means of testing the minimization of energy hypothesis. Erion (1991) and Leopold et al. (1964) discuss minimization of energy in more detail. Stream order analysis examines the nature of the drainage network and

Table 1. Applicability of analytical methods to channel stability investigations.

analytical method	geomorphic hypothesis	sediment discharge	drainage area
DS	**1**,2,3,**4**,5	L to H	S to L
MV	1,4	L to M	M to L
FE	**2**,**5**	L to H	S to L
ST	1,**3**,4	M	M to L
SB	1,**3**	L to H	S to L
GT	1,**2**	L to H	S to L
SF	**1**,2,3,5	L to H	S to L
CA	1,2,5	L to H	S to L
STS	2,5	L to H	S to L
SO	1,3,4	L to H	M to L
GEM	3,**5**	L to H	S to L

method: DS = downstream, MV = minimum variance, FE = form evolution, ST = sediment transport, SB = sediment budget, GT = geomorphic threshold, SF = safety factor, CA = comparative, STS = space for time substitution, SO = stream order, GEM = geologic evaluation mapping. **hypothesis:** 1 = equilibrium, 2 = disequilibrium, 3 = mass balance, 4 = minimization of energy, 5 = maturity. Bold number indicates the designated analytical method considered essential to evaluating that hypothesis. **sediment discharge:** L = low, M = moderate, H = high. **drainage area:** S = small, M = medium, L = large.

therefore contributes to the understanding of how energy is distributed over the landscape.

Downstream analysis, form evolution, safety factor analysis, space-for-time substitution, and geologic evaluation mapping evaluate the dynamic maturity hypothesis. Approximate analysis of landforms can be employed as a method of safety factor analysis (Waldo, 1990) although it also has a broader range of applications in geomorphic investigations (Waldo, 1983a; 1986). Comparative analysis enables comparisons of states of maturity among two or more channels.

Table 1 lists the methods of geomorphic analysis discussed above. Note that most methods are not universally applicable. For example, sediment transport analysis, commonly employed for evaluating channel stability, is not applicable to all channels. Downstream analysis appears to be the most universally applicable method.

DATA COLLECTION, ANALYSIS, INTERPRETATIONS

Data collection serves several important functions. It provides information necessary to the test procedures. Test criteria, consisting of values of pertinent geomorphic thresholds, are formulated from field observations and appropriate models. The data document the scope of the study and convey information to the scientific community.

The results of the tests are compared to the criteria for stability. A decision is made as to whether or not the channel meets the hypothesized conditions. Future projections are made and the hypothesis evaluated for each planning alternative.

Erion (1991) and Waldo (1991) additionally discuss data acquisition, analysis, and interpretation.

CASE STUDIES

A review of eight projects investigated by SCS (Table 2) indicates that the geomorphic approach leads to less expensive designs than previously used methods of stability analysis (SCS, 1977). In each of the projects the estimated installation costs were reduced by amounts ranging from about 10 to 70%. Cost reductions of 30 to 50% were typical. The projects, located in Alabama, Illinois (2), Indiana (2), Mississippi (2), and Texas, represent a variety of physiographic conditions.

CONCLUSIONS

Geomorphology provides several techniques for investigating fluvial systems. Application of those methods leads to an understanding of the behavior of the channel and its watershed. Predictions can be made concerning present and future stability of the channel, both with and without proposed modifications.

Table 2. SCS channel projects investigated by the geomorphic approach.

channel	major problems	geomorphic hypothesis	analytical methods	estimated savings
Choccolocco, AL (Aycock et al., 1989)	FC	1	DS,ST,GT, SF,GEM	45%
Tinley, IL (Waldo, 1983b)	FC	5	FE,SF,CA	15%
Warsaw Bluffs, IL (Waldo, 1983a)	DS,LV	5	FE,GT,STS, CA,GEM	65%
Muddy Fork, IN (Harvey et al., 1984)	FC	2	DS,FE,GT, SF,STS	40%
Twin Rush, IN (Bernard, 1984)	FC	1	DS,SF	35%
Oaklimiter, MS (Schumm et al., 1984)	LV,DS	2	DS,FE,ST, SB,GT,STS	50%
Standing Pine, MS (Jones & Smith, 1989)	LV	2	DS,FE,GT, SF,STS	45%
Three Mile-Sulfur, TX (Bircket, 1979)	FC	3	ST,SB	--

problems: DS = downstream sediment, FC = flood control, LV = land voiding. **hypothesis:** 1 = equilibrium, 2 = disequilibrium, 3 = mass balance, 5 = maturity. **methods:** DS = downstream, FE = form evolution, ST = sediment transport, SB = sediment budget, GT = geomorphic threshold, SF = safety factor, CA = comparative, STS = space for time substitution, GEM = geologic evaluation mapping. **savings:** the percentage base is the estimated cost of designing by stability criteria in SCS (1977).

The overview of the dynamics of the fluvial system and the study of both present and historic conditions enable those predictions to be made. Geomorphic techniques frequently lead to less expensive measures than commonly used conservative criteria. Therefore geomorphology tends to optimize the goals of stablilty, functional measures, environmental compatibility, and cost.

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GEOMORPHIC APPROACH TO CHANNEL STABILITY ANALYSIS

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This paper provides a framework for performing a channel stability analysis using site-specific data, established fluvial geomorphic and hydraulic engineering principles, and a statistical basis. Reaches of the channel system determined by analysis and observation to be stable are used as a data source. Mathematical relationships between characteristics of the channel cross-section geometry, slope or planform and quantitative measure of the forces acting on the system are developed, using regression techniques, from measured parameters in the stable reach of channel. These relationships are used to predict the response of the stream channel to modification.

INTRODUCTION

Characteristics of alluvial streams in equilibrium such as planform, cross-section, and slope are a result of the fluvial regimen or the long term effects of some average features of the runoff hydrograph, sediment discharge hydrograph, sediment size range, and downstream water elevation (Corps of Engineers 1989). This is analogous to the adaption of plant and animal species to a locality as the result of the geology and climate. Unlike the climate, however, changes in the watershed regimen as a result of man's activities on the watershed or stream do occur, particularly on small watersheds. In addition, work is often undertaken to change the characteristics of a stream channel such as modification of the planform, section or slope. Changes in characteristics not modified, some of which are adverse and severe, are often noted following the implementation of modifications to the stream system.

In a natural setting, and given sufficient time, a reach of channel adjusts its characteristics in response to changes in the fluvial regimen reestablishing equilibrium conditions. If available, quantitative relations describing equilibrium conditions could be used to predict the adjusted characteristics responding to a change. Quantitative representation of the fluvial regimen consists of properties of the water discharge hydrograph, sediment concentration hydrograph, sediment size distribution, and downstream water surface elevation and can be considered to be "forces" acting on the stream system. Theoretical and empirical quantitative relations between these forces and the channel characteristics can, and have been, developed. However, considering the number of variables which measure the forces and channel characteristics, the number of interrelationships between variables becomes very large. The theory, research, or model development at the current time is not nearly complete enough to explain the interrelationships between all the forces acting and resulting characteristics for the natural conditions. This is even more the situation when modifications are made by man. Site-specific studies, where the data is collected from the system to develop fluvial regimen relationships, have the potential for greatly reducing the number of relationships needing to be considered for evaluating the effects of proposed changes.

Stream channels may be modified for purposes such as flood control, transportation crossings, and water supply diversions. In order to design such works, there is a need to assess the future response to modification of the stream system. Traditionally, the engineering evaluation of proposed channel work by SCS has been done primarily by "fixed boundary" hydraulic approaches. These are outlined in SCS TR 25 (SCS 1977) as the allowable velocity and allowable tractive stress methods. In instances where sediment concentration is not high the resulting installations have been reasonably successful if maintained properly. Some contend the projects based on these designs are overly conservative and costly (Watson et al. 1986) and, where high sediment concentrations are transported, result in aggrading streams.

In fixed bed hydraulics, energy is dissipated by friction and by turbulence. In natural alluvial streams part of the energy is used to transport the water as in the fixed bed case; however, part is used to transport the sediment moving with the water. Equations describing the conservation of energy for flow with a fixed boundary do not adequately describe flow in a movable boundary system because the energy required to transport the sediment is not accounted for. Ironically, no generally acceptable equation has been advanced to describe the movement of water-sediment mixture in a movable boundary system, although numerous equations and models exist which are partial solutions or answers to specific problems. The fact remains that a delicate balance exists between the water-sediment mixture, the hydraulics of the flow, and the material forming the flow boundary. The methods described herein alleviate some of the drawbacks of design by the fixed boundary approach when applied to alluvial channels.

In several instances channel enlargement and straightening were performed in the past with little design for stability resulting, in time, in deeply incised gullies. Means have been needed for assessing remedial measures to halt the progression of the destabilization and to rehabilitate the systems economically. Additionally, in environmentally sensitive stream systems means for evaluating the effects of alternative proposed modifications have been required. Site-specific mobile boundary hydraulic and geomorphic studies have been applied to these problems.

DESCRIPTION

The geomorphic approach to channel stability analysis is a method for determination of the likely response of a stream channel to modification. The likely response is determined using predictive equations based on assessment of data from a reach or reaches of stream determined to be in a condition of quasi-equilibrium. The method entails identification of the quasi-equilibrium condition, locating stream reaches exhibiting this state, obtaining physical data on this reach, developing hydraulic data, and determining mathematical relationships which describe the quasi-equilibrium condition and can be used for prediction.

Stable and unstable stream channels may be modified with a need to assess the future response to the modification. A stable stream channel is, by definition, in a condition of quasi-equilibrium. Modifications to stable streams may include enlargement to increase capacity, realignment, diversion of flow to or from the stream, or changes to the watershed affecting runoff and sediment delivery to the stream. Stream channels in disequilibrium may be modified to effect equilibrium or hasten the recovery to quasi-equilibrium conditions. Prediction of the response to any modification can be accomplished by this method.

Identification of the quasi-equilibrium condition and locating the stream reaches displaying this condition should be accomplished by personnel trained in geology and geomorphology. This is done primarily in the field and is based on established principles and definitive indicators. The geomorphic setting of the entire watershed should be assessed at the same time. This information is needed during the overall evaluation of the proposed modifications. The geomorphic principles and indicators are addressed elsewhere (Waldo 1991). Where streams in disequilibrium are assessed, determination of the quasi-equilibrium state for historical conditions or use of similar watersheds in the region may be necessary.

Physical data needed includes engineering (level and distance) surveys, soil sampling and testing, sediment sampling, and possibly other data.

Hydraulic data may be determined from measurements taken in the field at channel cross sections during flow events, or synthesized using water surface profile techniques and survey data. It may be appropriate to use both methods in some cases.

Mathematical relationships describing the stable or quasi-equilibrium condition are used as predictor equations to determine the likely response of the channel to proposed modifications. These are determined by regression analysis of the interrelationship of descriptive parameters from representative reaches determined to be in quasi-equilibrium. Variables representing characteristics of the stream system are generally dependent variables (effect) while those representing the forces acting on the system are independent variables (cause). Relationships between some sets of parameters have been found to be representative of rational morphologic behavior and are found in the technical literature. A relation between stream power and channel width (Schumm et al. 1984) and that between energy gradient and the ratio of mean bed material size and drainage area (Simons et al. 1980) are examples. When recognized morphologic relationships can be shown to be applicable to the specific system being evaluated, rational geomorphic behavior can be supported, strengthening the findings and conclusions of the study. An abridged listing of recognized geomorphic relationships is included in appendix B. Deviations from recognized morphological relationships, when the rationale for deviation can be logically explained, are also useful.

The limits of applicability of the developed relationships need to be determined. A geomorphic threshold represents a set of conditions at which a particular process rapidly accelerates or decays. Once a threshold is reached, relationships developed may no longer hold true; therefore it can be stated that the limits of a relationship are confined between threshold conditions. For example, a determined relationship between a set of energy and geometry parameters holds true as long as the slope, discharge, and sediment gradation are such that the planform remains in the meandering classification. Should the slope increase so that the planform changes to braided, the relationship would no longer hold true. Limits may need to be set on some parameter to confine the extent of application of the determined relationships. In the example just mentioned the width-to-depth ratio could be limited to a narrow range, consistent with that measured in the quasi-stable reach. This will limit the predicted results from exceeding a threshold.

Natural streams are "dynamic", not only in the sense that water in motion is dynamic, but also in the sense that the shape, size, and alignment of the channels are changing, within limits, with time. However, the characteristics of natural streams are the result of forces interacting with resistances, both of which are natural occurrences, so the resultant "natural" changes occur in a systematic way. The systematic aspect can be assessed by the geomorphic theorem of downstream analysis (Waldo, 1991) and by the minimum energy principle of dynamic equilibrium (Simons and Senturk, 1977).

The geomorphic theorem of downstream analysis states geomorphic and hydraulic parameters vary systematically and predictably in the downstream direction. For example, channel width and flow depth tend to increase in the downstream direction while energy gradient and mean bed material size tend to decrease moving downstream. Significant and random deviations from the systematic variation in the downstream direction of a given variable are an indication of instability. In nature, however, the forces are not always constant with time and the resistances may not be homogeneous in space or time. As a result the natural changes may contain perturbations which express themselves as deviations from the otherwise systematic changes. These anomalies are physically explainable and are separate from the deviations from the systematic which are indicative of instability. Plotting of the descriptive parameters against location (channel station) is very helpful in visualizing the downstream analysis. The plot may be examined for its trend and a determination made as to whether the trend is logical and explainable. These plots can also aid in determining reaches of equilibrium and disequilibrium by the presence or lack of uniformity in the variance of a parameter with distance. Predictor equations should be supported by this theorem. Statistical methods may be used to assess the significance of downstream trends and the resulting equation used, in some cases, as a predictor equation.

Another applicable principle is that the rate of energy dissipation is minimal for an alluvial channel in dynamic equilibrium for the existing climatic, geomorphic, hydrologic, hydraulic, and man-imposed constraints. If, for some reason, the channel is forced to deviate from its minimum rate of energy dissipation, it will adjust velocity and slope through changes in geometry, sinuosity, or roughness so that the rate of energy dissipation can again be minimized. The general relationship which can be related to minimum rate of energy dissipation, attributed to Lane (1955), is stream power is proportional to the product of sediment discharge and mean particle size of bed material. This expression of dynamic equilibrium, in its simplest form, is:

$$Q * S \sim Qs * D50$$

This proportionality must remain balanced. For example, if the water discharge (Q) and mean sediment size (D50) remain constant and an increase in slope (S) is proposed, the sediment discharge (Qs) must increase. Predictor equations must be consistent with the minimum energy principle.

APPLICATION

In the application of the geomorphic approach the predictor equations are developed and then applied for the hydraulic conditions of the proposed modification, changes made in the modified conditions, and the steps repeated until predicted performance is satisfactory. The details as to the source of data for development of the predictor equations and their application vary with the problem at hand. A few typical cases are outlined below.

One case is to define the quasi-stable condition as the evolutionary end product of a system in disequilibrium. In this case the quasi-equilibrium reach from which the relationships are obtained may be a stream reach which has evolved to equilibrium, historical conditions before advent of the events producing disequilibrium, or another stream system in a similar setting. Understanding of the evolutionary process of channel development from disequilibrium to equilibrium and time and space relations between a sequence of channel forms in evolutionary development are necessary. In its most advanced application this method can be used to model the evolutionary process and predict the time it takes for a given location to evolve from one stage to another (Watson et al., 1986). Even without attempting to model the time aspect, the evolutionary stage at the location where modifications are proposed must be known to predict the effect of the modifications. Time-for-space substitution concepts (Schumm, Harvey and Watson, 1984) applied to the evolutionary process from disequilibrium toward equilibrium are useful in this analysis. Basically the evolutionary stages passed through at a downstream location are similar, and to some scale the same, as those which will occur at the upstream location. Accordingly, the evolutionary stages passed through in time at a given location are similar to the stages passed through in moving down the stream channel at a given time; thus, the term "time-for-space substitution". Descriptions of these changes with time are useful in identifying the evolutionary stage of a channel reach and predicting the quasi-stable end conditions of the evolutionary process. This special case approach has been used to determine measures which can be installed to promote, or hasten, the evolution of a severely eroded stream channel (in disequilibrium) to a stabilized (equilibrium) condition (Schumm et. al. 1984, Watson et. al. 1986, SCS 1989). In these applications predictor equations were developed from properties of downstream reaches determined to be in equilibrium and applied to trial modifications of upstream unstable reaches until modifications predicted by equations to be in equilibrium were obtained.

A second case, useful for evaluating proposed changes to a stable stream, involves developing the predictor equations and constraining relationships on the same reach(s) as that to which the modification is to be applied. The equations are then applied to trial conditions describing the modified system which are adjusted until satisfactory performance is predicted by the equations.

A third case is similar to the second case except the source of the predictive equations is a nearby stream system, determined to possess similar geomorphic properties, to which the proposed modification has been successfully applied.

Assessing the effects of changes in sediment load, sediment transport, or sediment grain size may require a detailed study in itself with any of the cases described above and is certainly true when sediment changes are the primary problem. In the first case described above, the development of the evolution "model" used to predict the quasi-stable end conditions is based on the assumption of no interference (by man) in the process. The effects of sediment changes take care of themselves in the geometry changes modeled. When, however, engineered modifications to retard further deterioration or hasten recovery are being evaluated, assessments of the effects of changes in the sediment conditions are required. In the second case above, sediment changes would be significant in nearly all types of proposed modifications. Sediment changes are a consideration in the third case only if differences between the data source system and system to be evaluated are evident.

The geomorphic approach to channel stability analysis is based on accepted principles and, being a site-specific method, should, when appropriately applied, result in designed stream channel modifications which are in equilibrium for both natural and man-induced conditions.

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Equation	Source
$QS = f(W^a)$	Henderson 1961; Schumm et al. 1984
$y = f(Q)$	Simons & Senturk 1977
$W = f(Q, Q_s)$	Simons & Senturk 1977
$w/y = f(Q_s)$	Simons & Senturk 1977
$S = f(1/Q, Q_s, D_{50})$	Simons & Senturk 1977
$P = f(Q_s)$	Simons & Senturk 1977
$Q_s = f(tV, M, 1/D_{50})$	Simons & Senturk 1977
$S = f(D_{50}/D.A)^b$	Schumm 1977; Simons et al. 1980
$W = f(Q_m{}^c/M^d)$	Schumm 1977; Simons et al. 1980
$y = f(M^e Q_m{}^f)$	Schumm 1977; Simons et al. 1980
$S = f(M^{-g} Q_m{}^{-h})$	Schumm 1977; Simons et al. 1980
$W/y = f(M^{-i})$	Schumm 1977; Simons et al. 1980

Where:

S = slope

W = top width

Q = discharge, water

Q_m = mean annual discharge

Q_s = sediment discharge

D_{50} = mean bed grain size

M = % silt-clay

t = tractive stress

P = sinuosity

y = depth

a, b...i = exponents

BEACH EROSION CONTROL BY USING DETACHED BREAKWATERS

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ABSTRACT

Coastline protection works incorporating detached breakwaters which act as strong-points to counter wave attack and littoral drift have been proposed to protect a stretch of beaches along the northern shore of Negara Brunei Darusallam. A physical model study was conducted to evaluate the size and spacing of the headlands with respect to the shape and size of the beach formed behind the headlands. It was found that for the wave and beach condition in Brunei, the appropriate configuration is for headlands of 50 m spaced at 220 m.

INTRODUCTION

The coastal areas of Negara Brunei Darusallam had experienced coastal erosion, resulting in denudation of the natural beaches. The denuded coastline has resulted in the rear landward areas being exposed to wave attack and continuing erosion influences.

For the proposed shore protection works at the eroded stretches, the Jerudong beach, the Consultant (M/s SPECS Sdn Bhd) had selected the coastal protection scheme which has the important advantage of providing aesthetically appealing beaches. This not only protects the coastline but also has the effect of restoring beaches which had been denuded by erosion earlier. The coastline protection works incorporate headland breakwaters, acting as the necessary strong-points to counter monsoon wave attack and littoral drift.

Detached breakwaters have been used in various places including Singapore (Silvester & Ho, 1972). Various other researchers have published their findings on the mechanics and configurations of the beach formed behind the breakwaters. Notable among them are Silvester (1970), Davies (1964), Yasso (1965) and more recently Wong (1981) and Hsu et al (1989).

The writers at Nanyang Technological Institute were commissioned by M/s SPECS to carry out the hydraulic model study to ascertain the suitable spacings and other characteristics of the headlands to be installed, whilst indicating the likely profiles of beach bays which would thus be created. The Study covered the determination of the optimum relationship of the size, relative spacing and distance from the shoreline of the detached breakwaters taken into consideration the predominant wave characteristics and tidal conditions.

Various aspects such as length and varying spacings of headlands, as well as the distance of the headlands from the shoreline were examined. The last factor was significant since headlands need to be built farther from the shore. Considering that the sea bed usually shelves downward into deeper water when further from the shore, there are practical limitations and the economic factor to be considered on the extent that the headlands can be sited from the shoreline.

The current study centered on the coastal protection works along the Jerudong coastline with respect to spacing and other characteristics for the headlands and indication of the profiles for the beach bays.

PHYSICAL MODEL STUDY

The physical model of a beach with uniform fine sand and the detached breakwaters is shown in Fig. 1. The ratios of prototype-to-model scales are as follow:

horizontal scale	1:36
vertical scale	1:25

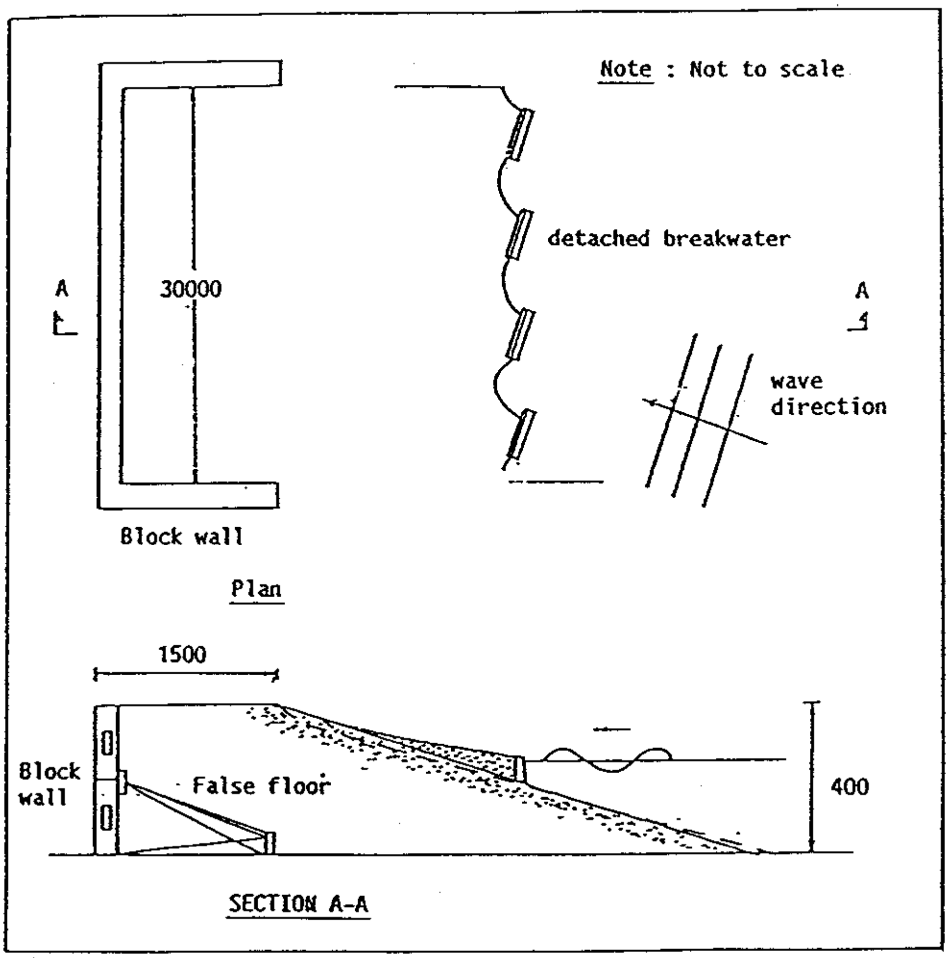

Figure 1. Layout plan and typical section of a model beach.

The model beach was initially placed at a slope of 1:10 approximately. Four headland configurations were examined in the model. The breakwaters were oriented in a direction normal to the incident wave. The angle between the wave crests and the shoreline was approximately 20 degrees. The leeward side of the breakwater was initially filled with model sand to form a tombolo (or crenulated slope) as would occur in actual prototype beaches to be formed. The base of the detached breakwaters were placed at 0 m ACD. The crests of the breakwater were set at 3.8 m ACD. Design prototype wave height of 1.2 m and sea water level at high tide (2.6 m ACD) were used as a guide in this study. Allowance was made to account for wave-breaking at the beach.

The results of the study were presented in terms of the equilibrium (or long term) shape of beach , distance from breakwater to shore (water line at 2.6 m ACD) and slope of the beach formed. The following scenarios were considered:

(1) Headland breakwater of 20, 50, 100, 150 m length at spacing of 220 m from centre to centre;

(2) Headland breakwater of 50 m length at spacing of 110, 165, 220, 250, 275, and 330 m from centre to centre;

MODEL RESULTS

Results from the model study were utilized to determine the size, relative spacing and distance from shore. Taken into consideration, the likely longer-term profile of the beach bays, the minimum width of beaches along the coastline could be ascertained. Providing more width of beaches was more beneficial both in protecting landward area and enabling an adequate strip of sandy beach material for recreational usage.

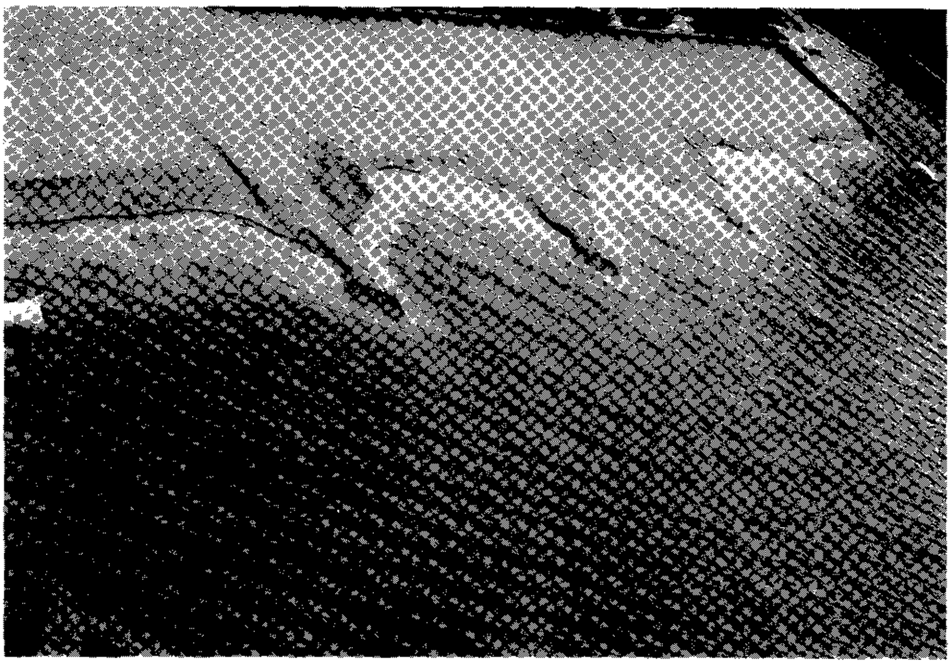

Figure 2. A typical view of the beach and detached breakwaters.

Beach form with detached breakwater

Figure 2 shows a typical view of the expected longer term equilibrium profile of the beach bays which were of crenulate-shape between the headland breakwaters. The size of the headlands shown in the figure correspond to that of the prototype situation of headlands spaced at 220 m centers and with headland length of 20m.

The equilibrium shape of the beach was formed after approximately 120 hours of continual wave action carried out on the hydraulic wave model. The wave orthogonals were generally at right angles to the longitudinal axes of such headland breakwater.

After the beach had reached equilibrium, the plan form and slope of the beach formed were measured. Figure 3 shows the shape of the equilibrium beach for detached breakwater of various lengths at 220 m (prototype) centre-to-centre spacing. Table 1 is a summary of the largest distance of the detached breakwater from shore and the slope of the beach formed for each size of the breakwater used.

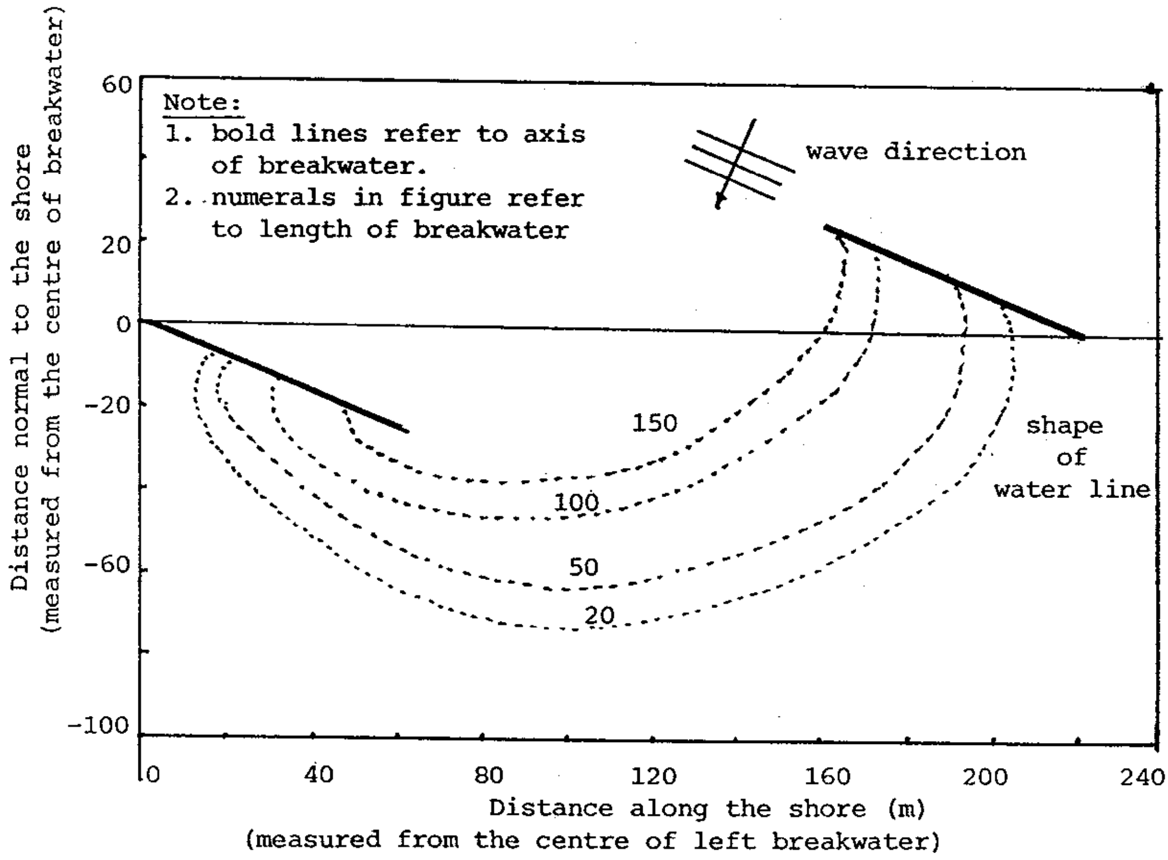

Figure 3. Shape of the equilibrium beach for detached breakwater of various lengths. The spacing of the headlands is 220 m.

Table 1 A summary of the distance of detached breakwater from the shore (at 2.6 m water line) for breakwater of various sizes, at 220 m spacing.

Length of breakwater	Distance from shore	Slope of beach
20 m	71 m	1:28
50	62	1:23
100	44	1:22
150	38	1:22

Figure 4 shows the plan form of the equilibrium beach for 50 m length breakwater at various relative spacing. Table 2 is a summary of the distance from shore (at 2.6 m water line) and the average beach slope formed for the same size of breakwater at various centre to centre spacing.

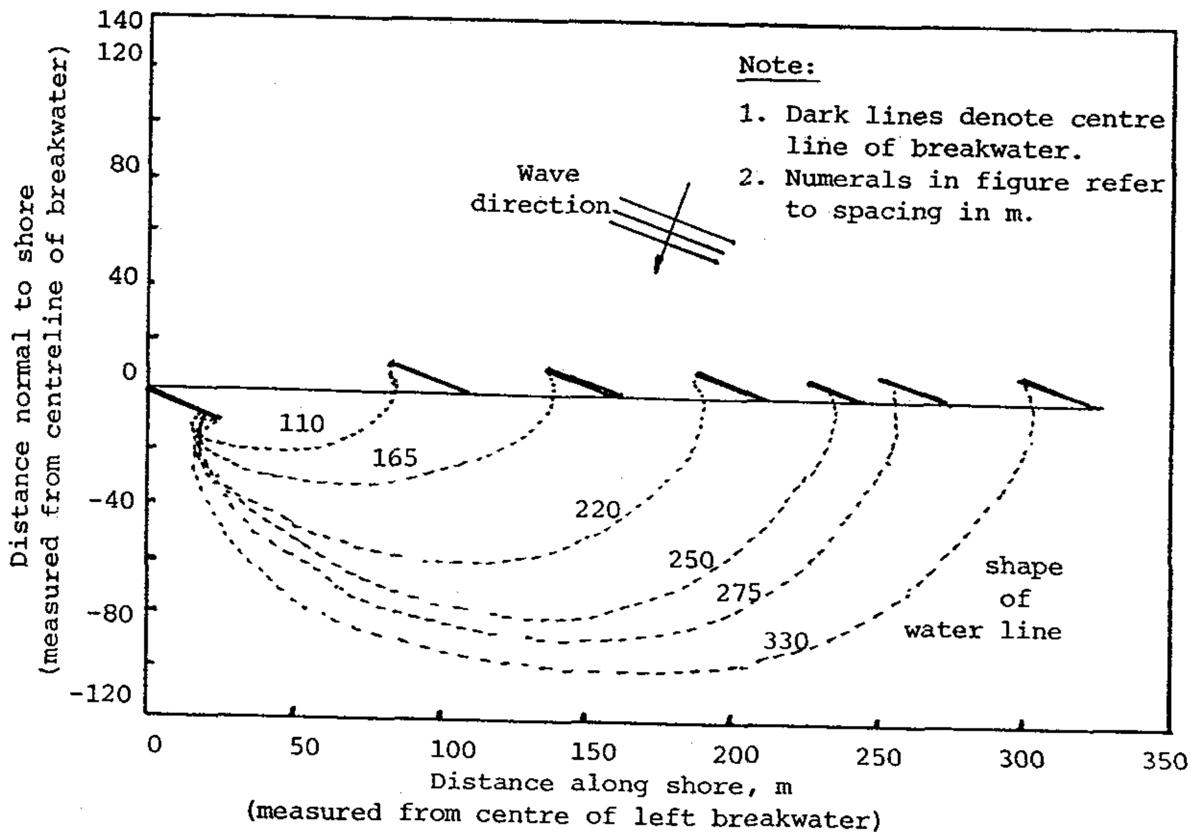

Figure 4. Shape of the equilibrium beach for 50 m length headland at various relative spacings.

Table 2 A summary of the distance of detached breakwater from the shore (at 2.6 m water line) for breakwater at various relative spacings.

Spacing of breakwater	Distance from shore	Slope of beach
110 m	20 m	1:30
165	36	1:27
220	62	1:23
250	80	1:24
275	89	1:28
330	100	1:27

The above results were all related to detached breakwater founded on the 0 m ACD contour line. It could be seen that the average slope of the equilibrium beach was 1:26. Figure 3 could be used to obtain the desirable length of breakwater at 220 m relative spacing and at a given distance from the shore line. Similarly, Fig. 4 could be used to estimate the desirable relative spacing of the 50-m long breakwater.

DESIGN ADOPTED AT JERUDONG BEACH

The beach protection scheme adopted at Jerudong Beach in Negara Brunei Darusallam utilizes headland breakwater of 50 m length at relative spacing of 220 m and the distance from shore to the headland breakwater is about 80 m. These figures were arrived at after considering various factors and constraints.

From the result of the model study, Fig. 4, it can be seen that the beach bay formed for such size and spacing of headland breakwaters would have a shore to headland of about 60 m. Therefore, the beach protection scheme adopted at Jerudong beach would result in the formation of a crenulated beach bay as well as a strip of sand beach of about 20 m wide.

The construction of the headlands at Jerudong beach is near to completion. Some bays have already been replenished artificially. The beach slope formed is about 1:30. In some locations where construction of headlands are in progress, preliminary sign of the formation of crenulated bays could already be seen.

ACKNOWLEDGEMENT

The support of M/s SPECS in the model study is duly acknowledged.

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DOWNSTREAM ANALYSIS

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ABSTRACT

Downstream analysis provides a means of testing geomorphic hypotheses concerning equilibrium or disequilibrium of stream channels. It develops quantitative understanding of the variation of important variables through the fluvial network. The method permits comparisons of channels and watersheds on the bases of geomorphic attributes and states of equilibrium. Downstream analysis in combination with other geomorphic techniques evaluates stability for present conditions and proposed modifications of the fluvial system.

INTRODUCTION

Geomorphology provides a means of investigating channels and their watersheds (Waldo, 1991). The suggested approach involves proposing and testing a geomorphic hypothesis regarding the state of equilibrium in the channel. Several techniques exist for testing geomorphic hypotheses. Among them, downstream analysis applies to most situations encountered during channel investigations.

The fundamental theorem of downstream analysis states that the morphometric and hydraulic characteristics of a stream channel vary in the downstream direction in systematic and predictable ways (Leopold et al., 1964). For example, the discharge observed for a specified flow event generally increases downstream in humid regions. Channel width, depth, and sinuosity generally increase downstream. Channel gradient and grain size of bed material frequently decrease downstream.

At-a-station analysis examines the variations of hydraulic and morphometric properties at a specified station on the channel as discharge varies. The variations are noted as the stage rises and falls in the channel (Leopold et al., 1964; Williams, 1978). At-a-station analysis deals with a variety of flow events at a single station, whereas downstream analysis considers variations during one event in an array of stations along the channel. At-a-station analysis complements downstream analysis.

Downstream analysis consists of two parts: station-by-station and total system analyses. Downstream analysis provides a basis for quantifying important geomorphic parameters and understanding their variation through the fluvial network. The method permits comparisons of channels and watersheds on the basis of geomorphic attributes and states of equilibrium.

STATION-BY-STATION ANALYSIS

Station-by-station analysis consists of making measurements and observations of morphometric, hydraulic, material, and process-related characteristics at selected cross-sections of the channel and its tributaries. Each cross-section generally represents a relatively homogeneous subdivision, or reach, of the channel profile. Some sections, however, represent specific features such as overfalls, meander bends, islands, confluences, bridge openings and other man-made features.

The values of each property are plotted versus distance downstream. Surrogates such as discharge or drainage area may be used in lieu of distance. Each plot demonstrates a unique relationship that can be examined for trend and judged for rationality. Apparent anomalies are identified, explained, and interpreted for implications concerning past, present, and future states of equilibrium.

Figure 1 illustrates some types of relationships encountered in the downstream array. Stream power (QS) varies in a normal fashion increasing in the downstream direction. Median grain size of bed material (d_{50}) varies abnormally, since the expectation is a downstream decrease. Top width to depth ratio (W/D) behaves chaotically, although a normally anticipated downstream increase apparently exists.

Sinuosity (P) gradually increases downstream to station 600+00 then gradually decreases to about 1300+00 (Fig. 1). An obvious anomaly exists in the sinuosity relationship downstream of 1300+00. Corresponding anomalies appear in the QS and d_{50} relationships at the same location. Similar anomalies may occur in the W/D relation but are masked by the overall chaotic nature of the trend.

Station-by-station analysis includes interpretation of the downstream array consisting of all relationships evaluated jointly. This evaluation reveals local conditions of equilibrium or disequilibrium. It provides data for defining geomorphic thresholds (Schumm et al., 1984) and proposing design criteria. At-a-station analysis determines frequencies of various flow stages and provides information for evaluating geotechnical and hydraulic stability of the banks and for considering environmental effects.

TOTAL SYSTEM ANALYSIS

Total system analysis treats stations and the reaches they represent as finite elements of a system. It sums, or integrates, the contributions of each station over the channel length thereby defining an attribute or property of the entire system. An array of various types of cumulative graphs defines several useful properties of the fluvial system (Waldo, 1989).

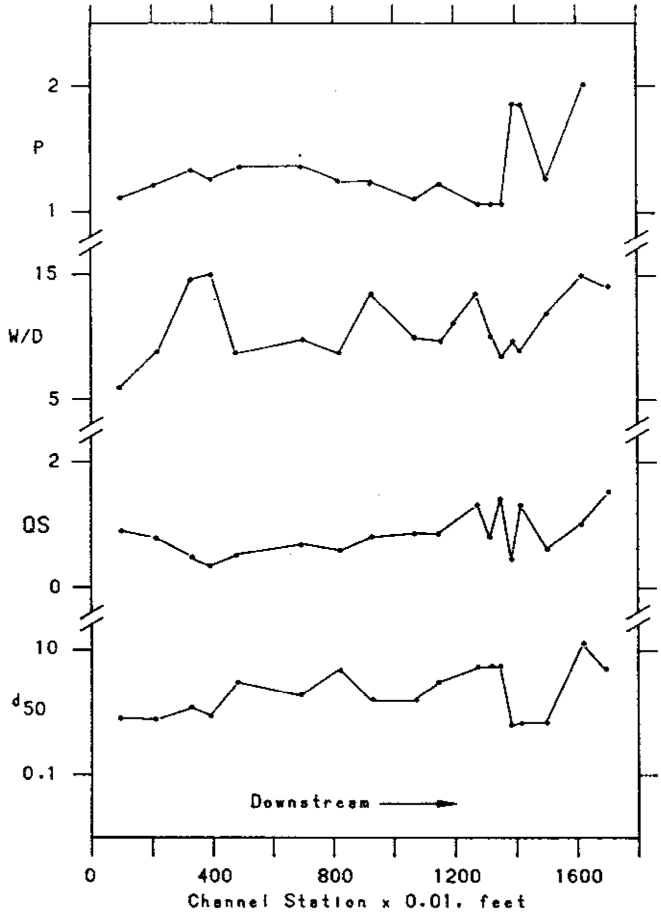

Figure 1. Array of downstream relationships, Choccolocco Creek, Alabama

P = sinuosity
W/D = ratio of channel top width to average depth
QS = stream power, cfs
d_{50} = median diameter of bed material, mm.

The array includes cumulative value relationships, cumulatitve percent curves, and length weighted mean values.

The cumulative value relationship shows how a given property is distributed along the channel. Cumulative value curves illustrate the function in whatever units the original measurements were obtained. The relationship between volume of bed material and distance downstream in Figure 2 serves as an example.

Cumulative percent curves offer several advantages. Use of a percentage scale removes the necessity of logarithmic transformations when dealing with certain sets of non-normally distributed variables. Cumulative percent curves enable comparisons between the distributions of two or more properties

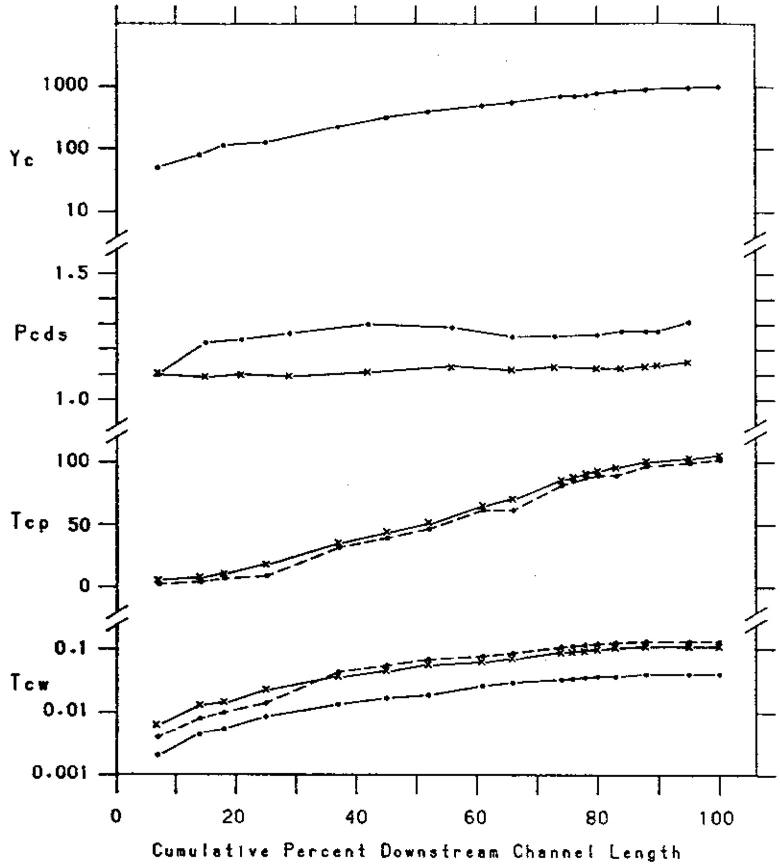

Figure 2. Four types of cumulative functions
Choccolocco Creek, Alabama

Yc = cumulative bed material, 1000 cu.yd.
Pcds = cumulative downstream sinuosity
Tcp = cumulative percent tractive stress
Tcw = cumulative weighted tractive stress, psf.
•—• = before project
×—× = after project
--- = allowable tractive stress

Tcp before project similar to TCP after,
therefore not shown.

measured in different units either in the same or different channels. See the relationship between cumulative percent tractive stress and downstream distance (Fig. 2) as an example.

Computation of length-weighted mean values of a property expresses it as a system attribute (Waldo, 1989). The length-weighted values can be summed sequentially as cumulative values and plotted against downstream distance. This relationship shows the contribution of each station or reach to the mean value for the system. The value of the cumulative function at the downstream end equals the length-weigted mean of the property. An example consists of the cumulative weighted tractive stress relationship in Figure 2. This technique gives meaning to certain cumulative properties, such as cumulative channel width, which would otherwise lack physical significance.

Some properties, such as sinuosity and average meander amplitude, can be computed either on a reach-by-reach basis or as progressive downstream values. When computed in the latter fashion, they represent cumulative downstream values and can be plotted against downstream distance (Fig. 2).

Total system analysis portrays the overall state of equilibrium in the channel as a fluvial system. It provides the basis for predicting future trends in the channel's geomorphic evolution and for estimating impacts of proposed modifications.

SELECTING VARIABLES FOR ANALYSIS

Minimum variance theory provides a means for identifying which hydraulic and morphometric variables to include in downstream analysis. Williams (1978) describes the method used in this study. Eight dependent variables were considered in relation to the independent variable, discharge (Q). The dependent variables consisted of width of water surface (W), average depth of flow (D), average velocity (V), slope of water surface (S), sediment concentration (C), friction factor (ff), hydraulic shear stress (T), and stream power (QS). Although the frequency at which bankfull stage occurs is not consistent throughout a system, it served well as the reference stage for determining values of the variables.

The author examined one case of variables considered essential to describing hydraulic stability (W, D, V, S) and 16 cases of variables grouped 5, 6, 7, and 8 at a time. Hydraulic exponents determined by minimum variance analysis were compared to observed data in Leopold et al. (1964) and Williams (1978). The observed data were assigned to groups defined by geologic conditions of bed and banks as described by Williams (1978). The three cases best fitting the observed data were:

cohesive banks: W, D, V, S, ff, QS
mobile bed: W, D, V, S, ff, T
general case: W, D, V, S, ff, T, QS

The downstream hydraulic relations of a channel can be characterized by considering the above list of variables as a minimum, the exact list depending on channel type (Williams, 1978). Additional variables require consideration for evaluating morphometric, planform, and geotechnical stability. They include width/depth ratio, sinuosity, grain size of bed and bank materials, bank height and slope angle, and sediment load. Other information including ground water, strength of bed and bank materials, and geologic characterizations may be necessary to address stability and environmental issues.

ANALYSIS OF GRAPHICAL RELATIONSHIPS

Identification of Trends and Anomalies

Other than the different nature of the functions involved, station-by-station and total system analyses proceed in similar fashion. Data are plotted, trends and anomalies are identified, and conclusions drawn concerning the state of equilibrium. Descriptions of trends and anomalies of station-by-station relationships appear above in reference to Figure 1.

Cumulative value graphs provide direct quantitative information concerning the variable in question. For example, estimates of dredging volumes could be determined from the Yc relationship in Figure 2. This relationship assisted in locating the sites for excavated sediment traps on Choccolocco Creek (Aycock et al., 1989; Waldo, 1989).

Cumulative percent graphs indicate relative rates of change along the channel when both ordinate and abscissa are percentage scales. Relative rates frequently suggest the presence or absence of equilibrium in the system. For example, uniformly distributed rates (i.e., 1:1 relationships on cumulative percent graphs) of downstream increases in bed material and tractive stress indicate equilibrium (Waldo, 1989). The Tcp plot on Figure 2 implies that much of Choccolocco Creek is in equilibrium and will remain so after the project.

Cumulative weighted value graphs serve several useful purposes. They indicate length-weighted mean values of variables and identify the contribution of each reach to the mean. They carry quantitative information unlike the cumulative percent graphs that have dimensionless values. A comparison of the Tcp and Tcw plots on Figure 2 illustrates the different nature of the two types of function.

Cumulative downstream graphs tend to smooth otherwise chaotic relationships and highlight the significant anomalies. The Pcds plot in Figure 2 masks the conspicuous anomaly at the downstream end in Figure 1. It suggests the decreasing sinuosity between stations 600+00 and 1300+00 constitutes the most significant anomaly in the channel as a system.

Interpretation of Trends and Anomalies

Interpretations of downstream relationships such as in Figures 1 and 2 should not proceed blindly. Field observations, geologic maps, subsurface investigations, and aerial photograph interpretations contribute to the understanding of downstream relationships. Aycock et al. (1989) and Waldo (1989) examined several downstream relationships in this manner.

Review of Figures 1 and 2 indicates that the channel length may be subdivided into three distinct regions: 100+00 to 500+00; 500+00 to 1300+00; and 1300+00 to 1700+00. The uppermost region corresponds to reaches underlain by sand-sized bed material and erosion resistant bedrock (Waldo, 1989). Geologic formations that produce substantial gravel by weathering underlie the middle region. The lowermost region represents a zone of alluvial accretion, with most gravel deposition occurring in prehistoric times (Waldo, 1989).

Anomalous trends and reaches in downstream relations of Choccolocco Creek do not indicate disequilibrium, at least not over a time frame measured in several years or decades. The anomalies are attributed to the introduction of coarse sediment in the middle reaches of the channel. The existing channel is generally energy deficient with respect to the coarse bed material. The project will increase energy, but not to the point it will disrupt the entire system. Sinuosity will be modified significantly by the project.

Reconciliation

Reconcilition constitutes the next step in downstream analysis. The interpretations of trends and explanations of anomalies in each relationship are compared. Unexplained differences between interpretations must be reconciled by reexamination of data, collection and analysis of new data, or application of additional theories and tests. Alternatively, unreconciled relationships and anomalies indicate disequilibrium (Waldo, 1989).

Review of interpretations in a matrix form assures that all relationships are considered. This reduces uncertainty which arises from possibly conflicting interpretations and from leaving some anomalies unexplained. For example, the relationships in Figure 1 appear in the matrix below. All apparently normal overall trends are designated by "0" whereas abnormal trends are indicated by "1".

	STA	P	W/D	QS	d_{50}
STA	0	0	0	0	1
P	0	0	0	0	1
W/D	0	0	0	0	0
QS	0	0	0	0	1
d_{50}	1	1	0	1	0

Abnormal trends in Choccolocco Creek involve the downstream coarsening of the bed material. Geologic circumstances in the watershed (Waldo, 1989) as discussed above adequately explain the abnormal trends. It is not necessary to resort to interpretations of short term disequilibrium.

Reconciliation should be performed for both station-by-station and total system cases. Overall trends and individually anomalous reaches should both be evaluated, and common explanations and interpretations found. Reconciliation applies to existing conditions as well as to each planned alternative. Reconciliation results in a thorough understanding of the fluvial system, both in its entirety and in its constituent reaches.

Synthesis

Synthesis represents the final stage in downstream analysis. This stage consists of formulating statements concerning equilibrium and disequilibrium, locally and system wide, past, present, and future, and with or without project installation. Results of station-by-station analysis identify localized conditions. Total system analysis provides insight into the system in its entirety.

Mathematical relationships may be developed as design aids (Erion, 1991). These depend partially on data being collected from stations scattered along most of the length of the channel profile. Otherwise the regression relationships may not prove significant and may even yield misleading results.

In cases like Choccolocco Creek, regression relationships cannot be developed with confidence. The graphical methods described above can be combined with techniques of geomorphic threshold determinations and safety factor analyses (Waldo, 1991) in such situations.

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RAINDROP IMPACT PRESSURE ON ELASTIC SURFACES

S. N. Prasad and M. J. M. Römkens[1]

ABSTRACT

The purpose of the present study is to examine the role played by the characteristics of raindrop and soil as an elastic material in the soil erosion process due to raindrop impact. The initial, pressure build-up phase, is analyzed in detail and a closed-form solution is presented for the duration of impact and crater depth.

INTRODUCTION

An understanding of the mechanics of raindrop impact on soil surfaces is of paramount importance in predicting soil erosion and surface runoff. Young and Wiersma (1973) showed that the raindrop impact effect is primarily responsible for initial soil detachment while the surface runoff transports the detached particles away from the impact sites. Depending on surface flow conditions the runoff may also erode the soil surface or may deposit a part of the suspended soil materials. The mechanics of soil detachment due to the impact of a single drop was systematically investigated by Al-Durrah and Bradford (1981, 1982a, 1982b). Huang et al. (1982, 1983) undertook numerical studies of the problem and focused on idealized cases when the flow is inviscid and incompressible. These cases, pertaining to a rigid and an elastic soil surface, were numerically investigated. Ghadiri and Payne (1981) studied the mechanism of impact by water drop experimentally and emphasized the significance of high impact pressure.

The theoretical study of the impact of liquid drops on solid surfaces, however, is relatively old because of the interests in turbine blade erosion and rain erosion of aircrafts. One of the first studies of this kind was by Cook (1928) who realized that high pressures could be accounted for in a water drop impact if the compressibility of the water was taken into account. He assumed that a cylinder of water struck by a solid from the side was equivalent to the sudden closure of a valve in a pipe line. He assumed that the impact pressure is the same as the water hammer pressure which may be calculated from the analysis of the one-dimensional Navier's equations of elastodynamics by considering impact of two rods with flat ends. This wave propagation analysis yields the following impact pressure, P

$$P = \rho_o C_o v \tag{1}$$

where ρ_o is the density of water, C_o is the speed of sound in water, and v is the impact velocity. This analysis was further modified by Gardner (1932) to allow for the compressibility of the solid surface which led to an improvement of the water hammer pressure and is given by

$$P = \rho_o C_o V / (1 + \rho_o C_o / \rho_s C_s) \tag{2}$$

where v is the velocity of water relative to the solid, ρ_s, the density of the solid, and C_s the speed of sound in the solid. Since $\rho_s C_s >> \rho_o C_o$ for most solid materials, the pressure calculated with this equation will be more or less the

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same as that calculated from (1). An exception, however, exists when the solid surface is highly compressible. This often is the case when the soil surface has been recently tilled and has not yet consolidated due to external load or negative soil water pressure. The soil matrix in this state consists of highly collapsible pore walls which may fail under compressive stresses of less than 1 MPa giving rise to a situation similar to a plastic wave propagation in solids under high impact loadings. Indeed, Ghadiri and Payne (1981) found experimentally that the impact normal stresses were in the neighborhood of 6 MPa when a 5.6 mm waterdrop collides with a velocity of 8.0 m/sec. The failure of the pore walls leads to compaction of the soil surface and this contributes to a surface seal which reduces the infiltration rate. In a separate study the authors have developed a model of soil compaction which is based on wave dynamics of soil surface. The objective of this study is to derive an expression for the penetration depth of an impacting raindrop as a function of drop and soil characteristics.

CHARACTERISTICS OF IMPACT

At the first instant of impact, ($t = o$) between a spherical drop and a planar solid, the contact between the drop and solid is a point circle. This is schematically shown in Figures 1(a). Fig. 1(b) shows the circular contact between

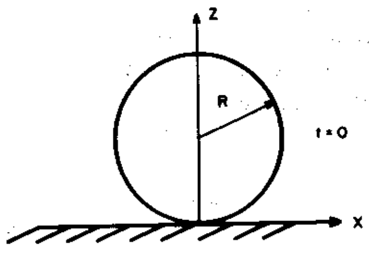

Fig. 1(a)

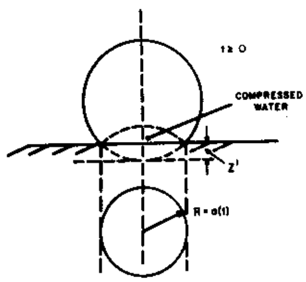

Fig. 1(b)

the drop and the solid at a time $t \geq o$, which radius before the occurrence of radial outflow is given by

$$a^2 + (R-Z')^2 = R^2 \tag{3}$$

The distance Z' through which the compression of water in the drop has taken place at the end of time t is vt. Therefore

$$a = [R^2 - (R - vt)^2]^{1/2} \tag{4}$$

From (4) we can calculate the rate at which the contact between the drop and the solid takes place. This is given by

$$\frac{da}{dt} = \frac{v(R-vt)}{[R^2 - (R - vt)^2]^{1/2}} \tag{5}$$

Equation (5), shows that the rate of contact depends on impact velocity, drop radius and elapsed time and changes most rapidly during a small fraction of a microsecond. This is the period of no flow in which direct compression of water in the region of contact takes place. In the subsequent phase fully developed lateral outflow occurs and pressure is released as the drop collapses onto the surface. Equation 5 is applicable to the impact of a perfectly compressible sphere on a rigid surface, or alternatively to a rigid sphere impacting a perfectly compressible target. Since the contact zone growth rate is initially larger than the compressional wave speed in the liquid, the boundary of the compressed zone with its geometric shape of an oblate hemisphere, moves parallel to the surface. This process continues until the jetting phase begins.

The onset of lateral outflow jetting has been a somewhat confusing issue in the field of liquid drop impingement. Some investigators have suggested that the onset of lateral outflow begins as soon as the drop strikes the surface. Whereas others assumed that the velocity of the contact zone between the drop and the plane surface must equal the shock wave velocity in the liquid. Bowden and Field (1964) assumed that a_c was the contact radius for which the compressional wave, initiated at the first instant of impact, reached a point on the boundary of the raindrop sphere. This also coincides with the time for full penetration into soil surface.

At this time radial outflow begins and the radius of contact a_c may be calculated from

$$\frac{a_c}{R} = 2 \frac{V}{C_o} [1 - (\frac{V}{C_o})^2]^{1/2} \tag{6}$$

In the same paper Bowden and Field (1964) presented a more representative relation of the wave structure at the drop interface. They reasoned that the velocity component parallel to the rigid surface at the time of lateral outflow initiation was equal to the critical velocity of the contact zone; then

$$\dot{a}_c^2 = C_o^2 - v^2 \tag{7}$$

using (5), the time at which the condition in (7) is satisfied is

$$\frac{vt_c}{R} = [1 - \{1 - (\frac{V}{C_o}) 2 \}^{1/2}] \tag{8}$$

Substituting this value for the time into (4), the critical contact radius is

$$a_c/R = v/C_o \tag{9}$$

The above results and various comparable analyses for the onset of lateral outflow jetting show that there is a need for a more physically based interpretation of the conditions at the boundary of the drop/surface interface. This is even more so for the interaction problem of soil raindrop impact in which the deformability of soil surface plays a significant role. In the analysis of Bowden and Field (1964) the assumption was made that the lateral outflow begins when the rapidly decelerating boundary of the contact zone between the drop and the solid surface equals the velocity of the shock front as it advances into the undisturbed liquid. It is only after the shock waves move past the expanding contact radius that an undisturbed surface is available to enable lateral outflow to begin. This initial phase may also be viewed as the contact pressure build-up phase in which the soil surface is increasingly being compressed in this interval of time. An analogy may be put forward by considering the raindrop as an impulsive indentor in which the indentation phase ceases with the onset of the jetting phase. Since one of our primary objectives is to obtain an estimate of inertial) of the system, an application of the principle of impulse and momentum appears plausible when the drop is assumed to be an indentor. The following analysis is motivated by these considerations.

IMPACT PRESSURE DETERMINATION

A theoretical solution of the problem of drop impact on a rigid surface was presented by Savic and Boult (1957) for an incompressible flow by solving Laplace equation for the velocity potential. Their analysis does not give a correct description of the initial phase of the collision due to the assumption of incompressible flow, but describes the later stages of impact correctly, at least for low impact velocities. The later stage of the impact has also been considered by Harlow and Shannon (1967). Assuming that the fluid was incompressible and viscous, they solved the Navier-Stokes equation for the impact of a cylindrical drop with a rigid surface using the marker-and-cell computing method.

Engel (1973) studied the impact of liquid drops against a plane, rigid surface using high-speed photography. She also measured the impact force with a barium titanate ceramic mounted on the surface of the target. Assuming compressible flow, she was able to estimate the average pressure on the surface during the impact and the outward flow velocity of the liquid from under the drop. Engel's calculated radial flow velocities in the intermediate and later stages of impact were found to be in good agreement with experimental results.

Johnson and Vickers (1973) measured the normal and shear stress distributions under a 50 mm diameter water jet when it struck an aluminum plate pressure cell at a velocity of 50 m/sec. The maximum pressure at the center of impact was approximately 2/3 $\rho_o C_o v$, rising to 3/2 $\rho_o C_o v$, at the edge of the jet, where V_o is the jet velocity. This pressure distribution is remarkably similar to the distribution obtained by pushing a rigid indenter with rounded ends into the surface of an elastic solid. Later study by Rochester and Brunton (1974) also confirms the same nature of pressure distributions in which the maximum pressure occurs toward the edge of contact.

The following analysis is presented in which the drop is considered as an indentor with rounded ends, which impacts an elastic half space. The assumption of an elastic half space limits somewhat the applicability of the following analysis. It is hoped that the results will at least be applicable in this case but an understanding of the nature of parametric dependence of inertial, geometric and material properties on the magnitude of pressure and impact duration will be helpful in the more general case.

The solution of the present elastic half space problem must be obtained by formulating a mixed boundary value problem of elasticity. Since the solution of the mixed boundary value problem of such a case is mathematically quite involved, we simplify the analysis by limiting ourselves to a few physically plausible pressure distributions. Thus, we consider the following three cases of pressure distributions:

(i) Uniform distribution under a circular area of radius a:

$$p = \frac{P}{\pi a^2} \ (constant); o \le r \le a \qquad p = o; r > a \tag{10}$$

(ii) Pressure under a flat ended indentor of radius a:

$$p = \frac{P}{2\pi a\sqrt{a^2 - r^2}} \ ; \ o \le r \le a \qquad p = o; \ r > a \tag{11}$$

(iii) Pressure under a rounded indentor of radius a:

$$p = \frac{3P}{2\pi a^3} \sqrt{a^2 - r^2} \ ; \ o \le r \le a \qquad p = o; \ r > a \tag{12}$$

It may be verified that in all the above three cases, the magnitude of the total force is the same P. Further, the vertical displacements under the various pressures are, respectively, given by

$$(i) \quad U_z(X, Y, o) = \frac{(\lambda + 2G)\ P}{\pi^2 a\ G(\lambda + G)} E(\frac{r}{a}); o \le r \le a \tag{13}$$

where $\lambda = 2G\nu/(1-2\nu)$ and E(r/a) is the complete elliptic function.

$$(ii) \quad U_z(X, Y, o) = \frac{(1-\nu)P}{4Ga} \ ; \ o \le r \le a \tag{14}$$

$$(ii) \quad U_z(X, Y, o) = \frac{(1-\nu)P}{4Ga} \ ; \ o \le r \le a \tag{15}$$

Note that except for (ii) the deformation under the pressure is one of depression with a maximum at the center. In case (ii) as expected the displacement is constant under the pressure. It can be readily inferred from (13-15) that the maximum displacement in all three cases are linearly proportional to the total force P but inversely proportional to the radius of contact a. G and υ are, respectively, the shear modulus and Poisson ratio of the soil. Thus, we may write

$$w = \alpha \frac{P}{a} \tag{16}$$

where w is the depth of the so called "crater" in the soil surface and α is the material constant whose values depend on the nature of pressure distribution and is given by

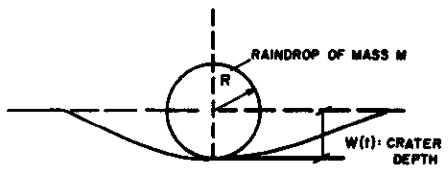

Fig. 2

$$(i) \quad \alpha = \frac{\lambda + 2G}{2\pi G(\lambda + G)} \tag{17}$$

$$(ii) \quad \alpha = \frac{1-\nu}{4G} \tag{18}$$

$$(iii) \quad \alpha = \frac{3}{8} \frac{1-\nu}{G} \tag{19}$$

With respect to Fig. 2, we now consider the conservation of momentum equation as applied to the drop. If m is the mass of the drop and U its velocity after impact, we have

$$m \frac{dU}{dt} = -P \tag{20}$$

The velocity U may be calculated from w and is given by

$$(iii) \quad U\,(X,Y,o) = \frac{3(1-\nu)P}{16Ga^3}\,(2a^2-r^2)\,;\ o \le r \le a \tag{21}$$

Eliminating P in (20) by utilizing (16) and using (21), equation (20) leads to

$$U = \frac{dw}{dt} \tag{22}$$

As discussed previously the radius of contact is changing quite rapidly during the pressure build up phase and may be given by (4).

$$a = [R^2-(R-vt)^2]^{1/2} = (2Rvt-v^2t^2)^{1/2} \tag{23}$$

Since the pressure build up phase duration is a small fraction of a microsec., t^2 will be much smaller than t and, therefore, an approximation for the radius a may be made by

$$a = \sqrt{2Rvt} \tag{24}$$

Thus, equation (22) may now be written as

$$\frac{d^2w}{dt^2} + \frac{wa}{\alpha m} = 0 \tag{25}$$

where

$$\theta = \frac{\sqrt{2Rv}}{\alpha m} \tag{26}$$

Equation (25) may be written as

$$\frac{d}{dt}(\dot{w}^2) + \theta\sqrt{t}\,\frac{d}{dt}\,w^2 = 0;\ \dot{w} = \frac{dw}{dt} \tag{27}$$

which may be integrated once to yield:

$$\dot{w}^2 - v^2 + \theta \int_o^t \sqrt{\tau}\,\frac{d}{d\tau}\,w^2\,d\tau\ 0 \tag{28}$$

The end of the pressure build up phase may be considered to coincide when the penetration ceases, i.e., when dw/dt = 0. Let this be known as the duration of impact and be given by the impact time T. Thus, the impact time T may be calculated from (41) by solving the following

$$\int_o^T \sqrt{\tau}\ \frac{d}{d\tau}\ w^2\ d\tau = \frac{v^2}{\theta} \tag{29}$$

If w_p is the maximum penetration w(t=T), (29) may be rearranged to yield

$$v^2 = \theta\ \sqrt{T}\ (w_p^2 - \frac{1}{2} \int_o^1 \frac{w^2}{\sqrt{\tau}}\ d\tau \tag{30}$$

The integral on the right hand side of (30) may be evaluated by various approximation techniques. Thus, a linear variation given by

$$w = w_p \tau,\ o \le \tau \le 1 \tag{31}$$

will yield

$$w_p = \sqrt{\frac{5}{4\theta}}\ \ \frac{v}{T^{1/4}} \tag{32}$$

Another relation between w_p and T may be obtained by solving (32). Unfortunately, no closed form analytical solution seems to exist for this type of differential equation. Therefore, a series solution of (25) is given below. For this purpose, we transform the original (25) into the following

$$\xi\ \frac{d^2 w}{d\xi^2} - \frac{dw}{d\xi} + 4\theta\xi^4\ w = 0 \tag{33}$$

where

$$\xi = \sqrt{t} \tag{34}$$

A series solution of (46) is given by

$$w = A(\xi^2 - \frac{4\theta}{35}\ \xi^7 + \frac{2\theta^2}{525}\ \xi^{12} + \ldots) \tag{35}$$

In the above A is a constant and it may be noted that (35) satisfies the initial condition that w(o) = 0. The second condition that dw/dt = 0; t = T leads to a determination of T which is given by solving

$$2 - \frac{4}{5}\ \theta\ T^{5/2} + \frac{24}{525}\ \theta^2\xi^5 + \ldots = 0 \tag{36}$$

An approximate value of the impact time T is given by

$$T = (\frac{10}{4\theta})^{2/5} \tag{37}$$

By an application of (50) the maximum penetration w_p is given by

$$w_p = \frac{1}{2} \sqrt{5} \left(\frac{4}{10}\right)^{1/10} \theta^{-2/5} \tag{38}$$

Equation (38) shows that surface compaction during the initial stage of raindrop impact on a soil is dependent on the raindrop radius, drop velocity, the soil shear modulus and the Poisson ratio.

SUMMARY

In this study the maximum pressure exerted by the raindrop and the crater depth at the site of impact are given special considerations for the purpose of understanding soil erosion. Closed form solutions are developed which show explicit dependence on the raindrop and soil characteristics. The solution is based on an application of the momentum balance equation in which various results of elasticity of half space and the dynamics of expanding surfaces are utilized.

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LAND REHABILITATION ON THE SUMTER NATIONAL FOREST

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ABSTRACT

When the overfarmed, eroded, or otherwise unwanted lands were acquired in the early 1930's to form the Sumter National Forest in South Carolina, few people would imagine the changes realized over the next 50-60 years. Approximately fifty thousand acres of actively eroding gullies, galls and roads were dissecting the Forest. Sediment continued to fill downstream areas as the channel gradients and water velocities dropped. Gully rehabilitation efforts on the Sumter National Forest (SNF) began with work by the Civilian Conservation Corps (CCC). Watershed characteristics were improved on many of these lands with just planting pine trees. Various other temporary to permanent practices were utilized in attempts to stabilize the erosion with limited budgets and plenty of manpower. But even after 50 years, many untreated areas continue to erode and cause downstream sedimentation. Since 1980, 1,600 acres of severely eroding land have been treated on the SNF with over 3,000 acres remaining on the inventory for possible treatment. Treatment includes a variety of techniques including land shaping, discing, subsoiling, fertilization, liming, seeding and mulching. Contour trenches prevent surface erosion following heavy rains. Gravel treatments have also proven effective in confined waterways and at gully heads. Most of the funding is allowed as part of timber sale improvements or provided with the Watershed Improvement Program of the U. S. Forest Service. Information is presented on watershed conditions, rehabilitation methods and economics using case studies or experience to provide examples.

INTRODUCTION

This paper briefly presents the history, methods and present status of the rehabilitation effort on the Sumter National Forest in South Carolina. The program is the largest in the U. S. Forest Service of over 150 Forests. Information transfer from past experience (both good and bad) may be helpful as others face similar decisions to prevent, mitigate or repair erosion and/or nonpoint pollution problems.

What started as a vision of land stewardship many years ago by various individuals and carried over into legislation by Congress (Weeks Act, Clean Water Act, National Forest Management Act, etc.) has resulted in many positive changes on the Sumter National Forest in South Carolina. Upon acquiring the worn out and abused lands of the cotton farming days, much of the Sumter National Forest looked like a barren wasteland. Repeated tillage of soil, burning of surface vegetation and failing to use or maintain adequate water controlling structures contributed to the erosion problems. Surface erosion left gullies and galls as a testimony to the reckless farming, mining and roading practices of the time. Sediment settled in downstream valleys causing impacts related to loss of stream habitat and channel storage. Land value and productivity continued to decline as long as the gullies were active. The slow natural process of restabilization was accelerated with the large scale planting of trees within the watersheds and some initial gully rehabilitation efforts by the CCC in the 1930's.

Gully rehabilitation started out slowly with only a few acres accomplished each year by the CCC or other special emphasis programs such as the PL-566 program administered by the Soil Conservation Service. Initial efforts had many successes and failures. Funding and programs were sporadic and little is known about any monitoring efforts to evaluate or maintain past accomplishments. By the latter 1970's, the Forest was annually treating 10-20 acres using U.S. Forest Service watershed improvement funds.

Funding for the rehabilitation program changed markedly after 1976 when the National Forest Management Act amended the Knutson-Vandenberg Act of 1930 to allow improvement of renewable resources such as soil, water and wildlife with timber sale receipts. An aggressive approach to improve Sumter National Forest conditions has resulted in a steady increase in correcting active gully, gall and road erosion. Annual accomplishments have jumped from 75-100 acres in the early 1980's to 175 acres in Fiscal Year (FY) 88, 155 acres in FY89 and 235 acres in FY90.

Funds to improve watershed conditions from National Forest timber sale receipts have net positive benefits to soil, water and wildlife resources. Success is not only measured in the reduced tons of sediment being delivered to streams or the marked improvement in land productivity, but it is also measured in the renewed interest in land ethics when the projects are shown to others. The opportunity to put something back into the land as opposed to the early farming practices of "just taking what you can get" is appreciated by Forest Service employees.

FOREST CONDITIONS

Gully and gall treatments on the National Forest were concentrated in the piedmont of South Carolina while much of the severe road erosion was treated in the mountains. The moderate continental climate with hot summers and mild winters has an annual rainfall of about 45 inches in the piedmont and about 67 inches in the mountains. Summer thundershowers or winter cyclonic storms can occasionally cause intense rainfall conditions.

Watershed conditions of the Sumter National Forest were generally poor when the lands were acquired. Normal surface hydrology was modified by roading and repeated tillage on sloping land. Inadequate or unmaintained surface drainage allowed erosion to occur unchecked for decades. Much of the erosion could have been prevented with adequate design and maintenance of structures (eg. broad based dips in roads and contour trenches on tilled areas). The results of erosion became a dominate feature of the landscape as farms and ditches were abandoned. Old roads became embedded into the landscape as surface drainage and erosion led directly into streams.

The severity of the erosion was also related to soil conditions. Surface soil properties were gradually lost in repeated tillage. Subsoil properties were often characterized as having a severe erosion hazard. Soils were derived from a variety of materials including mica-schist, granitics, slate and highly weathered gneiss. Some areas contained saprolite (disintegrated, decomposed gneiss) in the C horizon that is extremely erodible. Rapid gully expansion occurred if erosion reached the saprolite layer. Past erosion has left the soils across the piedmont in various stages of erosion. Often the surface horizon(s) are very shallow or not present. Also increased surface runoff and loss of nutrients has markedly decreased the site conditions for plant regeneration and growth.

Gully networks did not develop overnight. They begin with land practices that leave bare soil and do not deal with the problems of surface runoff and rill development. As these networks erode even deeper into the landscape, adjacent site conditions can be impacted. Shallow water tables are reached and depleted. Water and nutrients available for plant growth are reduced on effected areas. Gully development, once started, seems to be self-feeding and is a symptom of a watershed in trouble. Although gully formation and enlargement is largely episodic, long term erosion rates from a severe gully can easily exceed 1000 tons/acre/year.

Since acquisition of the Sumter National Forest, some gullies have remained active and continue to deliver sediment to downstream areas. Due to the widespread magnitude of the problem, most of the piedmont streams have silt bottoms and signs of fresh soil continue in the channels as a dominate characteristic. Downstream aquatic and riparian habitats were altered or destroyed. Increased flooding problems continue as channels have been filled in (aggraded). Today, wider riparian habitats on larger streams now provide a variety of conditions on the alluvial soils. As better land use practices prevail, streams are cleaner and able to begin the long process of flushing out some of the sediment that has deposited. Even though many of the gully sources are not as active as they once were, streams in the piedmont will remain turbid from the channel erosion for a long time.

METHODS

A variety of methods have been used over the years in numerous attempts to stabilize or rehabilitate the effects of gully, gall and road erosion. The term **stabilize** is used to refer to practices that halt expansion of erosion networks. **Rehabilitate** is used for types of practices that not only halt the erosion expansion, but also reduce the effects and enhance other resources as timber, recreation and wildlife. In concert with stabilization and rehabilitation measures, watershed conditions and treatments above the erosion may be critical to success. This section will present topical treatments that have been used and briefly discuss relative costs and success. Design and installation criteria are not discussed in detail.

Reforestation

Early efforts of the CCC to reforest significant areas of the Sumter National Forest were successful in reducing surface runoff and erosion to the extent that many active gully systems eventually healed themselves. This treatment was the most inexpensive and most utilized. The healing process was not immediate by just planting trees. Less eroded areas with better soils would respond best to this limited treatment.

Gully Plugs

Gully plugs consist of a soil dam constructed at some point along the gully with usually a drop inlet structure to lower water to the level of the downstream channel. Gully plugs have often been successful when applied to small gullied watersheds of only a few acres or less. Failure of this method is often associated with debris left in the gully plug fill or failure to provide for overflow (i.e. with an adequate spillway or drop inlet structure). When used on larger watersheds, success is related to drop inlet sizing, providing for emergency overflow, and ensuring proper installation to prevent structure clogging with debris or sediment. Fill materials should have adequate clay component to be compacted sufficiently and free of debris that could cause piping. Sediment will fill to the top of the drop inlet structure unless revegetation efforts are successful in reducing surface erosion.

Gully plugs are not presently used very often due to the cost of the drop inlet structures and the concern about failure. Where concurrent watershed efforts are implemented to reduce surface runoff through revegetation, the threat from active secondary nickpoints downstream is reduced. However, when properly designed and in conjunction with watershed revegetation, gully plugs can be very effective at providing long term stability.

Debris Dams

Debris dams were used on some of the early efforts to stabilize gullied areas. Structures were often made of small cedar trees piled between posts in the gully. Structures provided some short term stability success by slowing water movement and catching sediment. Debris dams break down after a few years and loose their effectiveness. Streamflow around structures is also a problem in larger streams. Straw bales with rebar support have also been used with similar results. Debris dams are not used much today on severe erosion problems associated with gully treatment. Where they can be used to provide some temporary improvement, costs are usually reasonable.

Rock Dams

Rock dams help stabilize eroding channels or waterways and provide permanent channel protection, water dispersal and allow for high water overflow. Costs, proper sizing of materials and frequency of structures are most critical considerations. Where materials are readily available and channel access is not a problem, costs can be reasonable.

Gravel Treatments

Gravel treatments have been a relatively recent addition to gully stabilization measures used on the National Forest. Gravel (surgestone - ungraded gravel about 4 inch minus) placed in gully heads or at nickpoints provides effective soil cover and erosion control while allowing present surface water movement pattern. This treatment is most likely to be considered when land reshaping is too costly, access is available or severe erosion on adjacent areas prevent water diversion.

Gravel is occasionally used to stabilize or maintain waterways, contour trenches or diversion ditches where excessive erosion occurs prior to vegetative cover. Gravel placement provides immediate benefits but costs can be a concern. Proper equipment and labor are necessary to move and place the gravel on the areas needing treatment. A small scoup on the back of a wheel tractor has proven to be excellent tool to move small amounts of gravel to specific sites.

Water Diversion

Water diversions are useful to stabilize gully heads if there is a place to safely divert the water without accelerating erosion on an adjacent area. Costs are generally very low when compared to other methods. The ability to divert water without other treatments may be limited in severely gullied terrain.

Several types of water diversions are used to improve road surface drainage. The types vary with the specific needs of the road but broad based dips, waterbars, berms, water leadouts (trenches) and even tank traps have been used. Some of the old roads treated had become entrenched into the landscape and a variety of practices were used to take advantage of opportunities to get the surface water off the road, to avoid excessive erosion and reduce the direct conveyance of materials into streams.

Land Reshaping

This is the most common method presently being utilized to rehabilitate gully erosion on the Sumter National Forest. Initial costs and maintenance are relatively high and can be several thousand dollars per acre. This method has provided the best long term rehabilitation for all resources. Practices associated with land reshaping may include building diversion ditches, waterways, contour trenches, ripping the soil (18-24 inches deep and sometimes in two directions), liming, fertilizing, mulching, seeding, planting trees and several years of maintenance. Primary costs are for equipment use.

Experience has shown that some practices could be altered to prevent or reduce operational costs:

1. Avoid land reshaping during the summer dry period to reduce costs. The soil strength is greater during dry periods causing increased equipment time to push in gully sides. Equipment use in the summer commands higher rates because of other projects (eg. construction, roads, farming).

2. Treat an actively eroding area before it erodes into the saprolite material if possible. Saprolite and mica schist are very erosive and provides a very poor site for plant growth. Avoid exposing these soils or using as fill material when possible. Gully expansion rate increases dramatically when the saprolite is reached, driving up treatment costs.

3. When moving large amounts of soil any distance, a pan earthmover is almost twice as effective as a dozer resulting in reduced costs.

4. Contour trenches that do not have constant slopes between 1.5 to 2 percent often have maintenance problems from either too much deposition or too much erosion.

5. Avoid placing debris into gully fills to prevent piping of water or settling of soil.

6. Planting trees in rips improves survival.

7. Utilizing experienced technicians to help plan and supervise the work reduces individual project planning and design costs.

8. Using grasses or legumes that provide both quality erosion control and wildlife habitat increases project benefits. Avoid using too much annual seed for rapid cover to ensure germination and survival of perennials needed for long term erosion control.

9. Instead of burning excess woody debris, utilize opportunities to trap sediment using brush or woody barriers downstream of treatments or pile debris for use by wildlife species.

10. Construct contour trenches of sufficient size (about 3 feet deep) to minimize maintenance and prevent failure due to settling of soil.

11. Tree survival and growth are increased if seedlings are planted during the same year as the grass mixture. Development of dense perennial cover makes reforestation difficult and costly.

12. Sometimes tree survival can be improved by inoculating seedlings with the appropriate mycorrhizal species.

13. Maintain the investment by checking and repairing water conveyances, refertilizing or reseeding if necessary. The most critical period is before the area becomes revegetated.

14. Contracting equipment through rental agreement (hourly rate) rather than by the project has generally reduced costs for projects and provided higher quality work. Less detailed planning, more flexibility to make changes, more control by experienced Forest personnel and less risk to contractor may be some reasons for rental agreements.

ECONOMICS

Some people take exception to the costs of accomplishing gully treatment. They are correct, costs are high. It is also costly to pull underground gasoline storage tanks, rehabilitate hazardous waste dumps and pick up trash along the highways. These are all forms of pollution. Most of the time, it costs much more to fix a problem that to prevent it. Had our forefathers used preventative land practices as Best Management Practices (BMPs), much of the costs of repairing or living with pollution problems would not be incurred. Yes, it even costs to live with pollution problems. If the sum of the costs could be listed, poor land ethics would be intolerable. Many of the costs are not obvious. Take gully or similar severe erosion problems as an example. If we could tally the costs, we would probably be amazed that gully erosion:

1. **Lowered land capability** to produce goods and services (timber, wildlife, vegetative crops, etc.). Often adjacent land areas upstream and downstream are also impacted during gully development.

2. **Produced turbidity** which reduces water quality and associated uses as fishing, recreation and swimming and increases water treatment costs for specialized uses.

3. **Increased sedimentation** altering aquatic habitats by filling in stream gravels, reducing reservoir capacity and changing flooding effects. Downstream effects may include wetlands, estuaries, and the ocean. Effects can last much longer than the gully or severe erosion that caused the problem.

4. **Reduced access** and ability to utilize land. Costs for road construction and activities to produce goods and services increase when gullies must be crossed or avoided.

5. **Promoted surface water movement** while ground water recharge may be reduced. Streamflow regimen may be changed to increased stormflow and reduced baseflow increasing flood and drought damages.

Because of the concerns about costs and benefits of land rehabilitation, in 1985 a programmatic economic analysis was developed to answer questions on economic feasibility. As with any analysis, assumptions were made on the market and nonmarket costs and benefits that could be associated with implementing the program on a project level. With the assumptions made, most projects have a rate of return in the 4-8% range.

An example would be the costs and benefits associated with a project accomplished in 1980 on the Tyger Ranger District near Union, South Carolina. The project in Compartment 47 included three active adjacent gullies and associated galled areas (4 acres of severely eroding land), road (1 acre) and surrounding area fertilized (2 acres). Costs of accomplishing the work included $11,628 to contract out land reshaping including pushing in the gully sides and reshaping the area, constructing water conveyances, subsoiling, fertilizing, seeding and mulching. An additional $3,200 was spent by National Forest personnel to plant loblolly pine seedlings, spot seed, mulch, fertilize and repair drainage structures for localized erosion control. Expected long term benefits over the next 50 years for the treatment include $7,300 for increased thinning and harvest of timber products, $26,000 for reduction in sedimentation impacts, $7,200 for wildlife improvements and $1,800 for visual enhancement for total benefits of $42,300. The calculated rate of return was 6.6 percent.

CONCLUSION

Early conservationists recognized the need to utilize resources wisely. Congress has been convinced and acted on the importance of land stewardship in significant legislation. With the passage of the Clean Water Act, preventative practices are designed and recommended by States to minimize impacts to water quality and associated uses. Implementation of the improved practices has begun through efforts to improve awareness and use of BMPs.

Other legislation has allowed the U. S. Forest Service to become very aggressive at treating erosion problems of the past. By taking advantage of the ability to utilize timber receipts to reduce gully, gall and road erosion, timber sales on the Sumter National Forest provide net benefits for soil and water resources.

Elsewhere there are opportunities to significantly improve water quality conditions inherited from the past through similar stabilization or rehabilitation efforts. Funding limitations must be overcome. Cost sharing opportunities are sometimes available from other agencies as the Soil Conservation Service or County Soil and Water Conservation Districts. Tax credits or incentives are another possibility. Landowners or agencies willing to pursue these opportunities may have to be creative in developing cooperative agreements with interested parties.

There are two basic courses of action that are selected, treatment or non-treatment. **Treatment** of severe erosion after it occurs is costly and has not gained much popularity with landowners that may have to foot the bill alone. Many of the benefits will not accrue to the landowner. **Non-treatment** is also costly, but the costs are not always obvious or eventually born by the landowner.

Hope for the future lies in the old saying "an ounce of prevention is worth a pound of cure". Utilization of Best Management Practices that are designed to avoid or minimize the effects of activities on water resources are reasonable and make sense to prevent nonpoint pollution problems. For future activities, landowners have a choice to accept voluntary BMPs or reject them. Rejection may encourage regulations that make BMPs mandatory.

Experience on the Sumter National Forest indicates that the "ounce of prevention" (BMPs) is the best course of action, but when necessary the "pound of cure" in the land rehabilitation efforts can be and is justified to achieve quality resource management. The results presented from the Sumter National Forest cannot be attributed to any one individual effort. Land rehabilitation has been achieved through time from early conservationists, appropriate legislation, identifying and using opportunities. Numerous individual efforts of Forest Service personnel with special programs have left a legacy of land ethics that we can all take notice of.

ACKNOWLEDGEMENTS

Ideas and work by hundreds of individuals have been important to the success of treating severe erosion problems on the Sumter National Forest for over 50 years. Although I could not name everyone involved with past and current programs, U. S. Forest Service individuals that should be mentioned for playing a significant part in the program over the last decade include Dennis Law, Forest Soil Scientist; David Rosdahl, Soil and Water Staff Officer; Donald W. Eng, Forest Supervisor; John Garrett, Contracting Officer; John Currier and Art Goddard, previous Soil and Water Specialists; District Rangers Larry Cope, John Cathey, Bud Johnston, Mike Vinson; District Staff Charles Drew, Ron Smith, Jimmy Shannon, Perry Shatley, Jim Abercrombie, Carl Arnold, Keith Barnes, Dennis Whitehead, Jorge Negron. Others that had higher level program responsibilities include Keith Grest, Region 8 (R8) Watershed Director, John Vann, R8 Improvement Program, Chester Robinson, retired R8 Improvement Program and Larry Schmidt, Washington Office Improvement Program. Thanks also go to Warren Harper, Washington Office for providing agency review of this document.

MEANDER BEND EXPANSION AND MEANDER AMPLITUDE DEVELOPMENT

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ABSTRACT

The movements of a meander bend may be resolved into two components: the bend expansion and bend migration. The bend expansion is the bend movement in the direction perpendicular to the down-valley direction. Such a movement will result in an increase of the meander amplitude. The bend migration, on the other hand, is defined as the down-valley direction movement of the bend apexes. In the present work, the study will be concentrated on the bend expansion and its direct result i.e. the meander amplitude development.

Meander bend expansion is caused mainly by erosion of the concave bank. Based on this principle, a formula is derived in wich the expansion rate is functionally related to the bend concavity. The theoretical model is verified and tested with experimental data and the results show good agreement between the predicted and measured meander amplitudes. Based on the relationl between the bent expansion anld the sediment transportation a meander bend expansion rate formula is derived, which expresses the bend expansion rate. The experiments were run under steady flow, in a movable-bed flume with uniform sand. The sediment transport coefficient is a linear increasing function of the discharge (Q).

INTRODUCTION

Leopold et al. (1964) have recognized three somewhat different kinds of channel patterns-meandering, braided, and sinuous. They were able to distinguish among these patterns by introducing a ratio of channel length to valley lenght. If this ratio, called sinuosity, is equal or greater then 1.5, the river is said to meander, otherwise the river is sinuous. Meandering and sinuous rivers flow in a rather well-defined channel. The braided river, however, does not at all have a single or well-defined channel, but a network of intrelacing streams. According to Blench (1961), these numerous streamls gradually reduce in number and develop into a sinuous or meandering river; braiding is considered as an incipient form a meandering. In an extensive study on river channel patterns, Leopold et al. (1957) have arrived at some wortwhile conclusions. It was found that braided-river reaches are generally steeper, wider, and shallower when compared with undivided reaches carying the same discharge Furthermore, a rather useful but preliminary criterion between channel slope S an bankfull discharge Q has been advanced, such as

$$S = 0.06\ Q^{-Q.44} \tag{1}$$

which distinguishes between meandering and braiding rivers. For any given discharge, meanders occur at smaller slopes; at larger slopes accompanied by strong sediment transport, braided streams are encountered.

As far as the geometry of meanders is concerned, important contributions have been made by Leopold et al.(1960).Three dimensions have been introduced to describe meanders. They are the length, width, and curvature of a meander, shown in Fig.1.

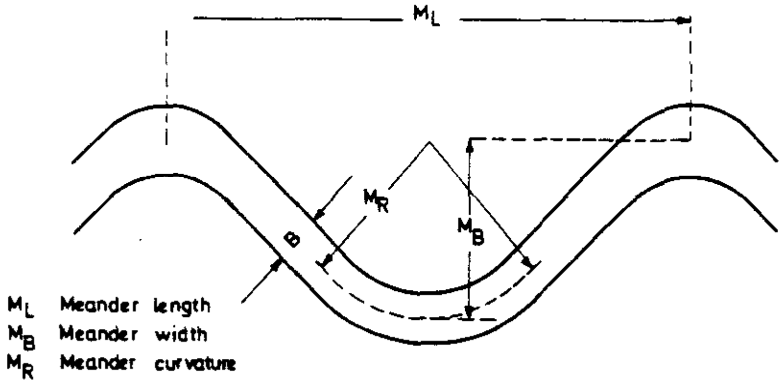


Fig.1. Definition sketch for meanders.

A relationship between the width B of a river and the meander width was suggested by Jefferson (1902), who also pointed to the fact that limited data suggest that a ratio between the meander length and the meander witht is of a constant value. The analysis by Inglis (1938) confirmed that both meander length and meander width vary with the discharge, and the meander length was given by

$$M_L = C_i \ (Q)^{1/2} \tag{2}$$

wherme C_i is $25 < C_i < 30$. Furthermore, it was found that the ratio M_B/M_L is of the same order, or

$$M_B/M_L \simeq 2.5 \tag{3}$$

for both incised and flood plain meanders. An incised meander is one whose free movement is restricted by the narrowness or absence of a flood plain. Incised meanders are sometimes called fixed meanders, while flood plain meanders are referred to as free ones. Thus a discharge increase is accompanied by an increase in the meander length, and bends of meanders move downstream. Working with models, Inglis (1947) reported a coefficiend of C_i as $30 < C_i < 37$ and the ratio of M_B/M_L of the same order as given by Eq.(3).
Leopold et al.(1960) found that the best-fitting empirical relations may be established between meander length or curvature and the channel width. They arrived at this conclusion after investigating all sizes of meanders, from laboratory streams to natural rivers of the size of the Mississippi. Recently, Zeller(1967) investigated mandering channels in alluvial matierial, solid rock, glacier ice, and limestone lapies. This study is particularly worthwile, because it not only includes further evidence of the existence of ice meanders and free meanders-exhibited by a density current flow in a storage basin-but also investigates the problem of miniature meanders. In a conclusion, Zeller (1967) suggests that meandering may occur in any Froude-and Reynolds-number range, and may be accompanied bay existence or absence of bedload transport.

Chitale (1970) has shown that the ratio of meander length to thalweg length and less the ratio of meander curvature to channel width are both dominant characteristics of river pattern. Data for 42 rivers were utilized to determine a statistical relationship.

The empirical relations give an idea as to what the geometry of any meander may be like. The field of mechanics, or hydraulics, of meanders, however, has not advanced very far.

It is konwn that flow through a bend is accompanied by:

1. A superelevation of the water at the outside of the bend.
2. A strong downward current at the outside of the bend, which will cause erosion if at all possible.
3. A scour action at the outside of the bend and deposition at the inside of the bend, forming the characteristic bend cross section.
4. A maximum forward velocity being close to the inside of the bend of a rectangular unerodible channel.
5. A tendency that, at any location within the bend, a water or sand particle at the bed (bottom) moves usually stronger toward the center of curvature than a water or sand particle at the top (surface). As a consequence of this, it is advisable to construct sediment-free takeoff canals on the outsied of a bend.
6. An extremely distorted and thus complicated velocity distribution throughout the bend.

MEANDER BEND EXPANSION RATE FORMULA

The rate of the bend expansiod defined as the distance by which the apexes of a concave bank retreats in the lateral direction per unit time, and is denoted as dy_b/dt.

In order to simplify the problem, the following assumptions have to be made:1) the flow is steady and the sediment is uniform and non-cohesive;2)the meander bends are circular; and 3)the bend expansion is due to the concave bank erosion. Based on the last assumption, the bend expansion rate can be related to the rate of the sediment transport rate by the following equation:

$$q_{b\ wh} = \frac{h_u \cdot L}{n} \cdot \frac{dy_b}{dt} \qquad (4)$$

where, $L = L(t)$ $L = L(t)$

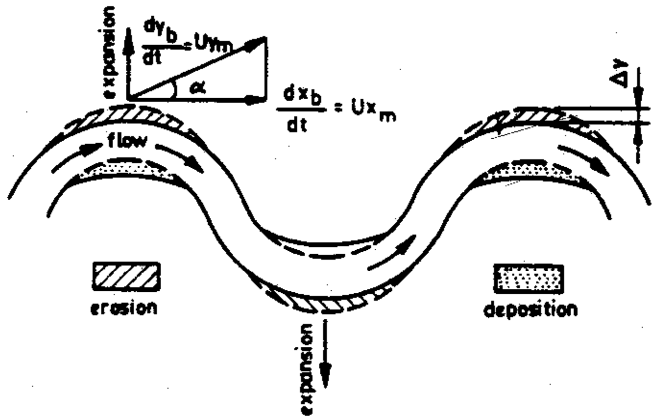
$q_{b\ wh}$ = the whole bedload of a meander bend.

h_u = flow depth at the meander bend axis.

L = meander bend length.

$\frac{d_{yb}}{dt}$ = the rate of the bend expansion.

$\frac{1}{n}$ = the constant; because the flow depth varies with time.



The movement of meander bend: (Fig.2.)

$$\tan \alpha(t) = \frac{\frac{dy_b}{dt}}{\frac{dx_b}{dt}} = \frac{dy_b}{dx_b} = \frac{U_{ym}}{U_{xm}} \tag{5}$$

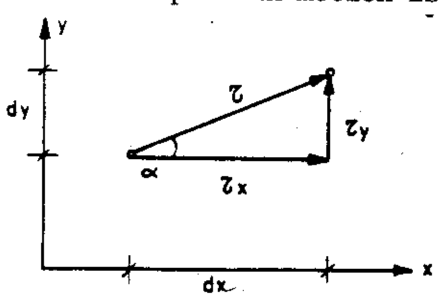
$$u_m = \sqrt{U_{ym}^2 + U_{xm}^2} = \frac{\sqrt{q_{bx}^2 + q_{by}^2}}{L.\ h_u\ .\ \text{factor}} \tag{6}$$

In order to simplify the problem, the following other assumptions have to be made:

$$q_{bx} \cdot d_{yb} + (-q_{by}) \cdot dx = 0 \tag{7}$$

$$q_{bx} \cdot d_{yb} = q_{by} \cdot dx \ ; \ y_b = \frac{q_{by}}{q_{bx}} \cdot x_b + C \tag{8}$$

In any approach to the problem of channel formation is the correct evaluation of the bedload. If that portion of the shear stress, τ_o, which is used to overcome the friction of the grains temporarily at rest is denoted by τ_c, the shear stress available to dislodge bedload and keep it in motion is given by



$$\tan\alpha = \frac{\tau_y}{\tau_x} \qquad \tau_b = \tau_a - \tau_c \tag{9}$$

Fig.3.Skin friction distribution in a flume boundary layer.

$$\tau = \tau_x + \tau_y \qquad \tau_b = \tau_o - \tau_c$$

$$\tau_x = \tau_{ox} - \tau_c \qquad \tau_y = \tau_{oy} - \tau_c$$

$$\frac{d_{yb}}{d_{xb}} = \frac{q_{by}}{q_{bx}} = \frac{\tau_{oy} - \tau_c}{\tau_{ox} \quad \tau_c} \tag{10}$$

Flow Dynamics

In a previous study (Holtorff, 1982,1983), it has been shown that the evolution of alluvial channels can be compared with a feedback system in which the effect, the geometry of a channel, and the cause, the stream flow, are interrelated. Suposing that a channel is fed by a discharge, Q, and a bedload rate, Q_b, the channel is in equilibrium if both Q and Q_b leave a control reach some distance downstream. If Q_b increases, the channel reacts by increasing the frictional resistance, by producing bed patterns. Keeping Q_b and the water depth, h, constant, Q can vary within the range of resistance of possible bed patterns. In meander flow, the channel cannot assume a stable configuration because of conflicting criteria for the bed and the banks. For physical fluids, however, a likewise oscillatory boundary layer exists which must be taken into account accordingly.
The oscillatory boundary layer flow can be laminar or turbulent Laminar flow has been investigated by Lin(Schlichting, 1965). If the stationary oscillation is described by

$$u(x) = u_m + U_o \cos kx \tag{11}$$

in which U_o = maximum velocity according to potential theory, the solution is given by

$$u(x,z) = u_m + U_{ou}(\cos kx - e^{-\phi z}\cos(kx - \phi z)) \tag{12}$$

in which $$\phi^2 = \frac{\sigma}{2\nu} = \frac{\pi . U_m}{L . \nu} \tag{13}$$

σ is the frequency of the oscillation and ν the viscosity. Köngeter (1980) have shown that for oscillatory flow, the boundary layer becomes turbulent if ϕ exceeds a critical value. The border between the laminar and turbulent region is about

$$\phi = \frac{400}{C} = (k.h.R_h)^{1/2} \text{ in which } R_h = h\, u_m/\nu \tag{14}$$

Hence, the following relationships can be developed for turbulent boundary layer flow;

$$\tau_{ox} = \tau_o - \frac{0.01}{\sqrt{2}} U_o.k.h.u_m.\ \rho(\sin kx - \cos kx) \tag{15}$$

$$\tau_{oy} = \frac{0.01}{\sqrt{2}} V_o.k.h.u_m.\ \rho(\sin kx + \cos kx) \tag{16}$$

EXPERIMENTAL SETUP

The experimental setup consisted of a 1.5 m. wide and 1.5 m. deep rectangular flume with rigid glass walls covered with artifical waves of iron plates.A (10 m.) straight length of channel has a bottom covered with artificial waves of iron plate, as the same of the walls with wave length=120 cm. and amplitude a=2 cm. The clear water supply was obtained from a 5 m. high overhead tank. In order to study the effect of oscillating shear stress distribution; the bottom slope was changed.

EXPERIMENTAL PROCEDURE

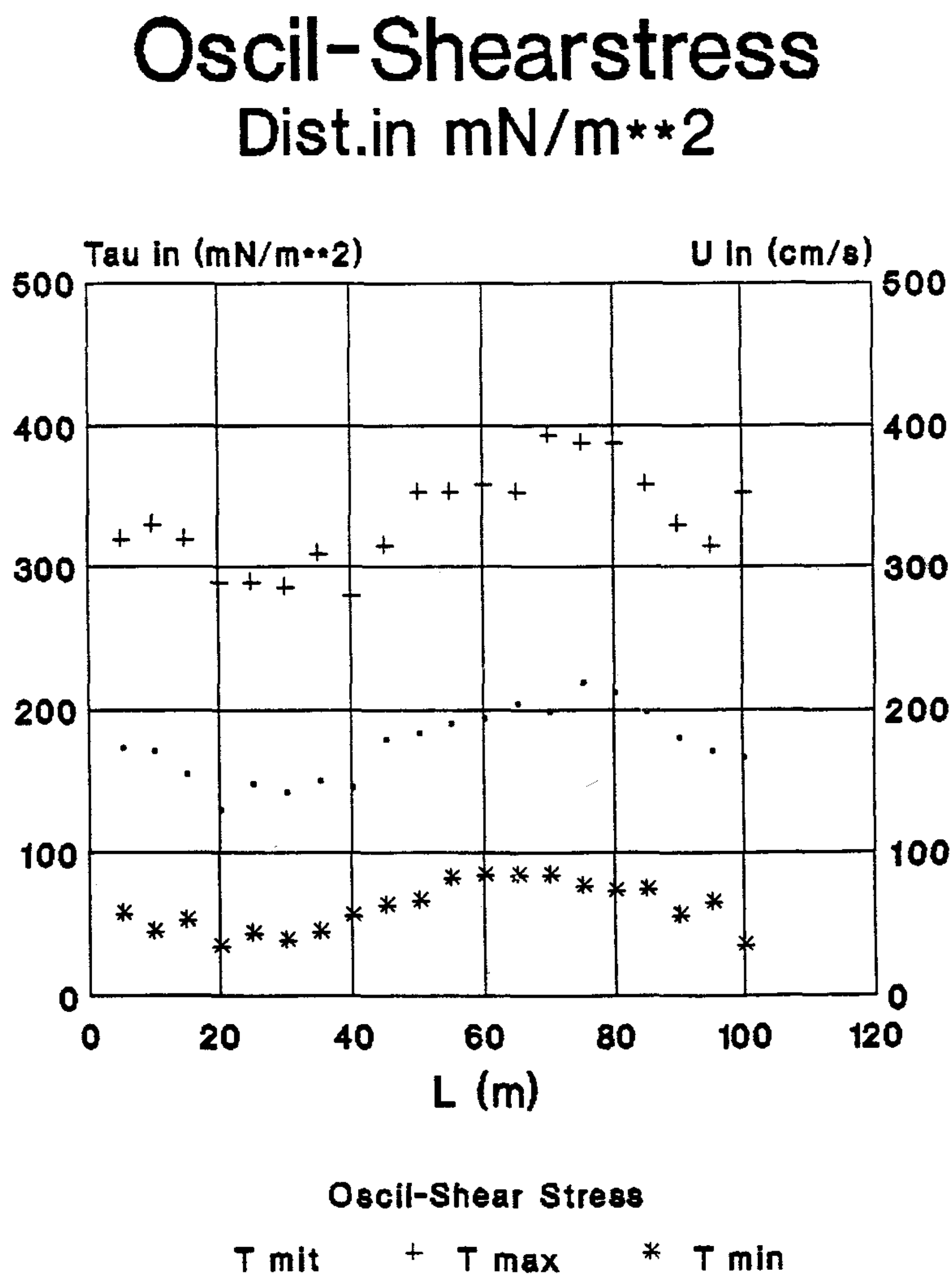
A given discharge was allowed to flow in the flume with a wavy bed. Uniformity of flow was ensured by adjusting the depth of flow constant near the entrance and exit reaches of the flume. The oscillating shear distribution on the wavy bed of the channel was measured with the help of a hot-film-WTG-50-sensor. A technique has been developed by Giselher Gust (1988), which measures skin friction in flume boundary layers, with and without suspended particulate matter, by constant temperature anemometry. With sensor scales of 3 mm. and frequency responses 20 Hz; flush-mounted, epoxy-coated hot films yield mean and fluctuating components of wall shearing stress at different water depths.
Fluctuating components of the skin frictionl $\tau_b = \tau_o - \tau_o$ are present in all turbulent flows, including those of hydrodynamically smooth walls (Eckelmann,1974), and require consideration in turbulent flow calibration techniques. In experimental runs mean friction velocities are obtained by multiplying recorded mean voltage signals by calibration coefficients obtained through either polynomial or exponential least squares fits of the calibration data. In turbulent flow the calibration curve of instantaneous (equal to mean) skin friction can be expressed for constant temperature anemometry as

Where the;
$$\tau^{1/2} = A_t \cdot E^2 + B_t \tag{17}$$

coefficients A_t, B_t are determined by least squares fits from the shape of the calibration curve (Hanratty and Campbell, (1983).

CONCLUSIONS

The flow resistance in a meander bend is considerably increased due to the form resistance of the patterns about which not much is known. It depends on a number of factors including grain friction, form resistance of two -and three- dimensional patterns, skin friction of the non-separated oscillatory component and the sediment transport rate. The sediment transport coeficient is a linear increasing function of the discharge (Q). The causes which are considered as the origin of a meander: (1) Local disturbances; (2) earth rotation; (3) excessive energy; (4) change in river stages;(5) forced oscillations. However, it will agreed that due to superelevation a spiral flow is introduced and, thus, a nonsymmetric mass transport is produced.



Q=26,551 l/s;Um=18,83 cm/s;Reh=44251

Figure.4. Oscillating Shear Stress Distribution at the wavy bottom of the channel.

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