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HARLAND CREEK INNOVATIVE BANK STABILIZATION DEMONSTRATION PROJECT

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Abstract: State-of-the art Bendway Weir and willow post method bank protection technology was applied to 14 eroding bends of a stream as part of the U. S. Army Corps of Engineers Demonstration Erosion Control (DEC) program.

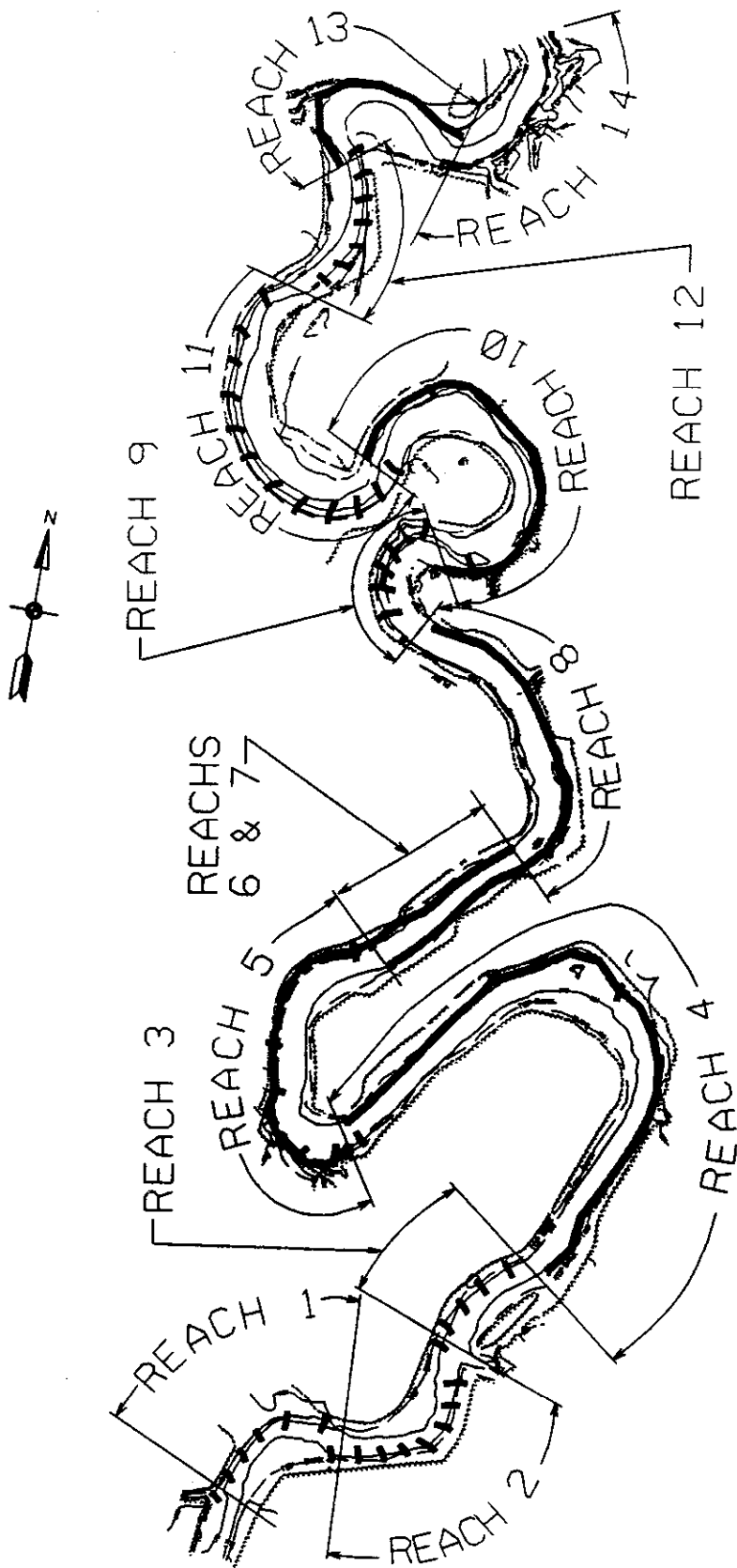
Introduction and Site Description: Presented herein are the planning, design, construction, costs, and first-year results obtained from a comprehensive bank stabilization/habitat improvement project for an 3,570-m (11,705-ft) reach of Harland Creek (a tributary of Black Creek) near Tchula, MS, located within the Yazoo River drainage basin (Map 1).

Stream dimensions vary widely. The average stream width is 29 m (95 ft). Bed and bank material ranges from clays and silts to gravel. This sinuous stream is of a "flashy" nature with the 2-year-recurrence discharge estimated at 106 m³/s (3,750 cfs). Sediment transport capacity is estimated at 8,700 metric tons per day at the 2-year discharge. Most surrounding land is forested. The stream is deeply incised, with the outer banks of all 14 bends in the reach being over-steepened and actively eroding. Bank height in the bends ranges from 1.5 to 13.4 m (5 to 44 ft), with most banks in the 3- to 6-m (10- to 20-ft) range. Pre-project cover on the banks ranged from bare soil to fairly dense vegetation. Bend radii ranged from 46 to 194 m (150 to 635 ft) while degree of curvature ranged from 0.52 to 3.11 rad (30 to 178 deg).

Aquatic Habitat Enhancement/Restoration: Stream corridor habitat enhancement/restoration was a top priority of this project. It was felt that the types of bank protection planned for this study would present a unique opportunity to demonstrate new channel stability methods while at the same time enhancing aquatic and terrestrial stream corridor habitat. The following habitat enhancing features (Shields, Cooper, and Knight, 1992) were identified and, whenever possible, incorporated into the project: occasional deep pools, stable scour holes, stable habitat, diversity of habitat, solid substrate for invertebrates, well sorted stone size gradation, canopy cover, and woody debris.

History, Theory, and Design of Bendway Weirs: Since 1988 Bendway Weirs have been tested in eight movable-bed models at the U.S. Army Engineer Waterways Experiment Station (WES). Because of this successful model testing program 80 weirs have been built by the U. S. Army Engineer District, St. Louis in nine bends of the middle Mississippi River. The weirs are submerged upstream-angled stone sills (2,270 kg (5,000 lb) maximum weight rock) located within the navigation channel of the bend. Model and prototype results suggested that several benefits gained from the weirs could be applied to the realm of small-stream bank protection.

The Bendway Weirs for the Harland Creek project were designed to reduce erosion on the outer banks of the bends by reducing near bank velocities, reducing the concentration of



FLOW

LEGEND

-  WILLOW POSTS
-  BENDWAY WEIRS
-  ϵ OF LONGITUDINAL PEAKED STONE DIKE OR STONE TIEBACKS
-  APPROXIMATE TOP BANK
-  APPROXIMATE TREE LINE

HARLAND CREEK TEST SITE
GENERAL PLANFORM
MAP NOT TO SCALE

currents on the outer bank of the bend, moving the thalweg in the bend from the toe of the eroding bank to the stream end of the weirs, and producing an overall better current alignment through the bends and crossings. The following design parameters were used in this project: all sets (fields) of Bendway Weirs were built in an upstream to downstream progression; all weirs were keyed into the bank using the standard Corps hard point key design; spacing was set at either 23 or 30.5 m (75 or 100 ft); weirs were sloped from an elevation 0.6 m (2 ft) above the bed at the stream end to 1.2 m (4 ft) above the bed at the bank end; crest width was set at 0.6 m (2 ft); side slopes were specified as the natural angle of repose (1 on 1.5); and all weirs and weir roots were constructed of R-650 stone (295 kg (650 lb) maximum weight stone). Maximum weir angle was set at 0.35 rad (20 deg). In most cases the angles of the weirs at the upper and lower ends of the bends were reduced so that flow would be well aligned when entering and exiting the bend. This was based both on experience gained from the movable-bed model studies and consideration of the geometry (small-radius bends and short crossing lengths) of the test reach. Weir length was determined by engineering judgment, based largely on the anticipated relocation of the thalweg and the expected reshaping of the point bar.

The Willow Post Method and Design Parameters: The modern willow post method, developed and used by Mr. Don Roseboom of the Illinois State Water Survey since 1986, controls streambank erosion through the systematic vertical installation of large native willow cuttings to stabilize eroding streambanks. The stabilization process is multifaceted. Willow foliage lowers floodwater velocities on and near the eroding bank, the root system of the willow binds the soil together, the post itself helps to stabilize the bank soil, transpiration aids in lowering the moisture content of the eroding bank, and the resulting stable bank allows the establishment of volunteer plant growth. Some of the advantages of the willow post method are: low costs in terms of materials, installation, and maintenance; protection is long-term; and aquatic and terrestrial habitat is provided and/or improved.

The guidelines used in this study are a slightly modified version of the design used by Mr. Roseboom: spacing specified as a 1-m (3-ft) grid, i.e., 1 m (3 ft) between rows and 1 m (3 ft) between posts in each row; minimum post diameter 76 mm (3 in.) at the butt end; minimum willow post length of 3.3 m (10 ft); posts planted at least 2.44 m (8 ft) deep using a 203-mm (8-in.)-diameter auger with no more than 1.2 m (4 ft) of the post showing above the ground. Native willows in good condition were used with the elapsed time between cutting and planting not to exceed 48 hours. Willows were kept wet after cutting with the tops of the posts marked to ensure posts were planted upright. **Willow posts must be planted when dormant.** Dormancy is defined as after the leaves have dropped and before the leaf buds have appeared (usually between 1 December and 1 March, depending on location and weather). Willows were planted for the entire length of the outer bank of each bend with no willows planted above top bank or in the water.

Project Construction and Costs: The construction contract was awarded to Procon, Inc., of Brandon, MS, a general contractor with some streambank protection construction experience but no experience with the protection methods employed in this study. The total contract amount was for \$322,845.00. The average cost over the entire reach was \$90.43 per meter (contract cost of \$322,845.00 divided by the total length of protected bank, 3,570 m). The construction contract bid was broken down as follows: \$75,000.00 for willow

post planting (divided by 9,383 posts for a per-post cost of \$7.99); \$161,832.00 for construction of 54 Bendway Weirs and keys; \$70,913.00 for three sections of longitudinal peaked stone toe protection and tiebacks; and miscellaneous costs of \$15,100.00 (mobilization and demobilization, debris removal, erosion control).

The weirs and longitudinal peaked stone toe were constructed in August 1993, while the willows were planted when dormant (February-March 1994). One bend consisted of willows behind longitudinal peaked stone toe, four bends were willow posts only, five bends were Bendway Weirs only, one bend had weirs and longitudinal stone toe, two bends had willows with Bendway Weirs, and one bend used all three protection methods.

First-Year Results: A comprehensive project evaluation was carried out in the fall of 1994. Since March 1994 the project has been subjected to the 2-year recurrence interval discharge about once every 2 months. Nonetheless, initial results have been satisfactory, with most areas of the project appearing stable and maturing quickly.

The Bendway Weir reaches appear especially stable. As expected with any discontinuous type bank protection some scalloping has occurred between weirs, but the bank slopes have readjusted and benched and native plants have quickly taken root. Included in these plants are a large number of native volunteer willow trees. The biggest problem noted was some bank erosion and caving between the last two weirs and downstream of the last weir in all bends. This problem has cured itself in all but two of the bends where there is still some bank erosion downstream of the last weir. Repairs were carried out in these two bends in June of 1995. Longitudinal peaked stone toe protection keyed into the root of the existing weir and extending 30 meters (100 ft) downstream of the last weir were constructed in these two bends in June 1995. All bends experienced the predicted amount of erosion on the point bar with the channel thalweg moving to the stream ends of the weirs. Within the weir fields many wide and deep stable pools have formed. A very important habitat feature is the stability of woody debris trapped within the weir fields. Even with several flood events this debris has stayed in place. A large storm in January 1995 where flow overtopped the county highway bridges had only a minimal effect on the project. Most bends with Bendway Weirs experienced sand deposition on the outer banks of the bends.

Most willow post areas appear stable, but overall survival rates have proved to be less than expected. In most areas the row of willows closest to the water have died or been washed away (the sacrificial row). According to Mr. Roseboom this is fairly typical. However, in some areas more than one row has been lost. The survival rate in June 1994 was 80 percent, (Watson et al, in preparation) but had dropped to 42 percent by October 1994. The reasons for the low survival rate are being investigated. Several factors probably come into play, including soil conditions, the fact that due to contract language the auger holes were not properly backfilled, and a large flood event 3 weeks after planting broke off most of the new branches. However, many of the willow posts that have appeared dead for months have sent up live shoots from their base or just below the soil line of the post.

Monitoring: The project will be intensely monitored, surveyed, and documented over the next several years by both WES and Colorado State University personnel. Findings will be published in the DEC Project Monitoring Program fiscal year reports for 1993-1997. The

Agricultural Research Service National Sedimentation Laboratory, Oxford, MS, is also conducting a 3 year evaluation of the environmental effects of the project. First year sampling results have shown that twice as many fish were found in the areas using Bendway Weirs.

Conclusions: Using a general contractor and a Corps design featuring new bank protection methodologies, a 14-bend reach of a sinuous, incised, eroding stream was largely stabilized and rapidly maturing 1 year after project completion despite at least four out-of-bank flood events. Stream corridor habitat was improved and project costs were approximately half of other projects in the area.

Acknowledgements: The presented research was conducted by WES under the DEC Project Monitoring Program sponsored by the U.S. Army Engineer District, Vicksburg (CELMK). Permission was granted by the Chief of Engineers to publish this paper. The author would like to thank the DEC Project Program Manager at CELMK, Mr. Franklin Hudson; the project co-designer, Mr. David Abraham of WES; and CELMK personnel Dr. Phil Combs and Messrs. Jasper Lummus and James Ross for prototype data and design assistance.

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ECOLOGICAL STUDIES ON THE DEMONSTRATION EROSION CONTROL PROJECT IN THE YAZOO BASIN

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Abstract: Improved management of agricultural watersheds is a major goal of state and federal agricultural and environmental agencies in the United States. Assessment of the ability of a watershed management strategy to control non-point source pollutants or channel instability and erosion is central to evaluation of the strategy's effectiveness, but appraisal of a watershed's ecological well being is often mandated by a concerned public, regulatory agencies, or law. Evaluation of a management strategy's impact on watershed ecology is difficult because of the complex nature of aquatic ecosystems. Both survey and individual experiment approaches provide vital information necessary for ecological evaluation. Ecological surveys provide an overall picture of plant and animal diversity/distribution in addition to a measure of patchiness and watershed heterogeneity. A survey approach to evaluation has the advantage of documenting landscape scale watershed conditions and general quality and is suitably broad-based to examine linkages between different ecological components. Although surveys highlight major problem locations or subwatersheds, they are too general to adequately evaluate success of individual management activities. Specific experiment approaches are more narrowly focused, providing ecological impact information for individual management techniques. Because specific experiment approaches acquire data at a higher resolution, they can detect ecological changes that are masked in a survey approach due to watershed complexity and variability. Specific experiments also provide the necessary information for "fine tuning" management techniques to achieve greater ecological benefits. A combined approach has been taken in evaluating the Demonstration Erosion Control (DEC) Project in the Yazoo Basin. Watershed surveys of water quality parameters are used to highlight problem subwatersheds and overall performance of erosion control measures. General survey data from fish, invertebrates and habitat are incorporated into biotic and ecological indices so that different watersheds can be compared. Watershed changes over time can also be detected with this approach. Individual management techniques such as field scale grade control pipes, stream grade control structures, and structural/biotechnical bank protection measures are evaluated and modified for improvement through specific experiments. Combining survey and specific evaluation investigations also documents cumulative contributions of individual projects to watershed-scale trends.

INTRODUCTION

The history of natural science has been shaped by classical faunal and floral studies. Many studies over the centuries have purposed to name or list species occurrence; others have compared floral and faunal changes resulting from a defined set of differences. Documentation of floral and faunal occurrence coincided with the settling of the Americas. Classical studies to understand basic ecological processes in aquatic ecology in the United States centered around university activity in the 19th Century and early 20th Century. As human influence touched significant portions of the landscape, faunal and floral changes were documented, thus setting the groundwork for survey-type studies that evaluated environmental conditions or changes (Forbes 1928). Survey-type studies in the 1990s vary from classic natural history designs to environmental impact statements (EIS). Specific experiments also date back to the beginnings of science. They commonly have some combination of control/variable design, and their purpose is more specific than general surveys.

Watershed management must use multipurpose approaches to address complex problems that have resulted in both physical and ecological damage. Holistic approaches for addressing erosion/sedimentation problems are rapidly evolving. These approaches focus on the watershed as a basic management unit and consider interactions between and among land use practices, small-scale water and erosion control measures, reservoirs, and channel control structures. Watershed/channel stability projects resulting from holistic approaches include suites of practices and structures applied across the landscape. Evaluation of a project's effectiveness in controlling contaminants or erosion/sedimentation should be a priority for cost-effectiveness; a holistic evaluation must include environmental impacts and ecological processes. While the survey approach to ecological evaluation has the advantage of

documenting landscape scale watershed conditions and general stream health, specific results are necessary to evaluate individual activities or projects.

In 1984 the US Congress directed the U.S. Army Corps of Engineers and the USDA Natural Resources Conservation Service to establish demonstration watersheds to address critical erosion problems on land and in stream channels of hill lands and Piedmont regions across the U.S. Specifically, development and testing of systematic watershed soil conservation, channel stability, and flood control programs in these Demonstration Erosion Control (DEC) watersheds were requested. This demonstration included many individual structural and non-structural conservation and stabilization efforts combined into a total system of watershed management. The USDA National Sedimentation Laboratory was directed to evaluate project effectiveness in controlling erosion, sedimentation and pollution as well as environmental impacts on watershed ecology (Cooper and Knight 1991a). This article describes the development and some immediate results of an ecological evaluation of the DEC Project.

MATERIALS AND METHODS

Study Sites: Seventeen watersheds located in the East Gulf Coastal Plain Physiographic Province along the bluffline of the Mississippi River Valley were selected for the DEC project (Fig. 1). These loess hill watersheds were chosen based on their history of disturbances, locations and intensive agriculture. Grissinger and Murphey (1982, 1986) described the area's history of large scale erosional sequences which include upland gully formation, agricultural land soil losses, and resulting floodplain sedimentation.

Project design included long term water quality, habitat and biological monitoring of sub-watersheds, and testing of specific management practices or structures. Number of sample sites and sampling frequency are found in Table 1. Sub-watershed sampling provided focused information on performance of management techniques and allowed detection of problems at the level needed for initial decision making. While sampling sub-watersheds furnished a broad view of cumulative effects of watershed treatments, testing of specific practices or construction designs gave performance information which may be used to predict applicability in other watersheds or sub-watersheds.

Physical: Habitat data collection and analysis were based on standard methods (e.g., Petersen, 1992; U. S. Environmental Protection Agency 1989, Berkman *et al*, 1986) but were modified to describe conditions in the channelized, eroding stream beds of northern Mississippi. Depth and bed type were sampled at about 100 points at each site at roughly equal intervals along 20 transects. Water surface width was measured for each of the 20 transects as was depth, bed type, top bank width, channel depth, and bank vegetation. Beaver dams and man-made structures (e.g., weirs, revetments, jacks, etc.) within or immediately downstream from each reach were noted, and the area of each large woody debris formation in the plane of the water surface was visually estimated. Channel bed slope measurements and bed sediment size distributions were obtained for as many sites as possible from data files.

Water Quality/Chemical: Temperature, conductivity, dissolved oxygen, and pH were measured in-situ routinely by electronic water quality meters. Total solids, suspended solids, dissolved solids, nutrients and coliforms were analyzed by standard methods (APHA 1975, USEPA 1974). Pesticide and contaminant metal samples from storm flows were analyzed by gas chromatography and atomic absorption spectrophotometry (USEPA 1971). Storm runoff was sampled seasonally. Samples were collected in the upper third of the flow depth.

Biological: Fish were collected from all major habitat types: pools, riffles, debris piles, dikes, and grade control structure pools. Sampling methods varied by habitat type and local conditions. Seines, fish traps, fish toxicants (rotenone), and boat and backpack mounted electroshocking equipment were used. Shocking times were used to calculate catch per unit of effort. Because of variability in actual shocking time and instream habitat sampled per reach, an index of stock abundance (catch per unit of effort) provided a better means of evaluation than the absolute weights or numbers of fish collected (Knight and Cooper 1991a). Macroinvertebrates were collected from all major habitat types according to procedures described for Rapid Bioassessment Protocol (USEPA 1989).

Table 1. Watershed, sampling frequency, and sampling sites for ecological evaluation of DEC project.

Long Term Monitoring

Watershed	Water Quality			Fish, Invertebrates, Habitat		
	Sites	Frequency [†]	Period	Sites	Frequency	Period
18	75	Weekly; biweekly	3-9 Yr	42	Summer	2 Yr
				50	Intensive	1 Yr

[†] Biweekly on a Wet season/ Dry season Basis.

Specific Testing

Watershed	Structures or Measures evaluated	Sites	Frequency [‡]	Period	Parameter
Long Creek	Rehabilitation	2	Sp/ Fl	3 yr	Fish, Habitat, Invert.
Hotophia Creek	Rehabilitation	1	Sp / Fl	3 yr	Fish, Habitat, Invert.
Hickahala Creek	Rehabilitation	2	Sp / Fl	3 yr	Fish, Habitat, Invert.
Toby Tubby Ck	Rehabilitation	1	Sp / Fl	2 yr	Fish, Habitat, Invert.
Goodwin Creek	Rehabilitation	1	Sp / Fl	3 yr	Fish, Habitat, Invert.
Batupan Bogue	Bank Protect.	12	Sp / Fl	3 yr	Fish
Black Creek	Bendway weirs	9	Sp / Fl	1 yr	Fish, Habitat
Goodwin Creek	Grade Controls	6	Summer	2 yr	Fish
Otocalofa Ck	Snag Debris	4	Sp / Fl	2 yr	Fish
Long Creek	Grade Controls	6	Summer	2 yr	Fish
Otocalofa Ck	Watershed components	41	Summer	1 yr	Pesticides and Metals
Otocalofa Ck	Storm events	1	Seasonal	2 yr	Pesticides and metals
Coldwater River	Water/sediment	12	Seasonal	2 yr	Pesticides and metals
Coldwater River	Bioassays	12	Fall	1 yr	Sediment Quality/Toxicity

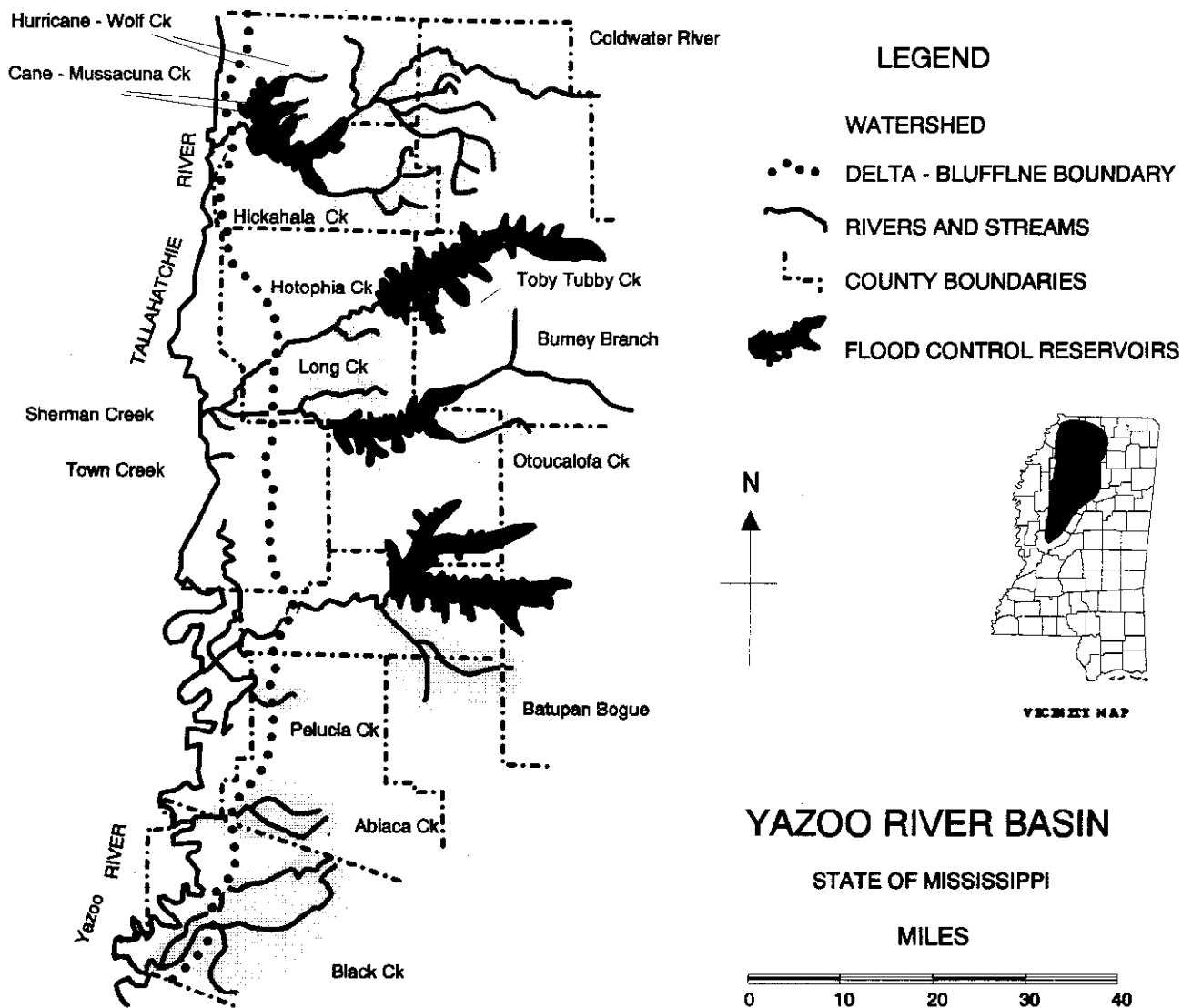
[‡] Sp = Spring; Fl = Fall

RESULTS AND DISCUSSION

Ecological response to watershed management practices was measured in three major areas: habitat, water quality and biota. Due to the complexity of aquatic ecosystems, no single area provides a complete measure of ecological changes. For example, changes in habitat may be immediately detectable, while biological response to moderate perturbations may take longer to emerge. Although more quickly detected, habitat changes may or may not indicate ecological problems. Moderately disturbed habitats are often the most productive and have higher species diversities (Ross et al. 1987). In general, water quality parameters such as low dissolved oxygen concentrations or high suspended solids are good indicators of watershed problems; however, many species of aquatic life are adapted to survive short term declines in water quality (Cooper and Knight 1991b). Numbers and diversity of aquatic organisms provided a direct comparison for evaluating DEC efforts.

Habitat: Habitat changes were more readily and rapidly detectable than biological responses to channel stabilization and habitat restoration measures. Habitat related parameters such as velocity, depth, substrate and percentage of pool habitat were valuable in evaluating restoration components of the DEC project. Habitat availability, heterogeneity, and stability were important factors in determining the numbers and species of animals in aquatic habitats. Water depth, cover, and bed composition were critical to stream fish diversity. Research relating habitat and fisheries resources provided the information needed to modify existing erosion control structure design to enhance fish habitat (Shields et al. 1993).

Fig 1. Map showing the locations of the Demonstration Erosion Control Project watersheds.



Water Quality: Because all aquatic life is adapted to specific limits and rates of change in the physical and chemical characteristics of water, water quality constitutes an important component of a watershed evaluation project. Although water quality sampling may not be sensitive enough to detect gradual changes from watershed management short of some threshold value, it may help to eliminate possible causes of deterioration in ecological health of rivers and streams.

Base flow water quality was within acceptable limits in DEC streams; however, water quality deteriorated during storm events (Cooper and Knight 1991b). Management practices that reduce storm flows such as flood water retarding structures improve storm driven water quality. Sampling sub-watersheds in the DEC project allowed

watershed managers to target problems such as excessive nutrients associated with specific farming operations or waste water treatment plants.

Biota: In a simple sense, biological data provides the easiest measure of watershed management success. The very presence of plants and animals indicates that conditions are sufficient to maintain life. This apparent simplicity is misleading because it represents a single point in time and does not address trends, sensitivity or diversity of organisms. Ecological evaluations that depend solely on population size or spatial distribution of organisms do not document changes in diversity or biological integrity. Changes over time in populations of indicator species provide a clearer measure of watershed success than population studies.

Of the popular approaches to biological assessment, the most common relies on the concept of indicator species to identify low to high quality (Hilsenhoff 1977, 1982). A variation of this procedure measures the relative abundance of desired species such as sport fish (Coble 1982). Another approach is the computation of diversity indexes which combine the number of species present and evenness, or the degree to which all species are equally represented (Wilhm and Dorris 1968; Pielou 1975). The Index of Biotic Integrity (IBI), originally based on attributes of fish assemblages, was designed to integrate information from individual, population, community, zoogeographic, and ecosystem levels into a single ecologically based index of the quality of a water resource (Karr 1981). Biological indices are potentially powerful evaluation tools if methods of calibration and interpretation are adequately developed.

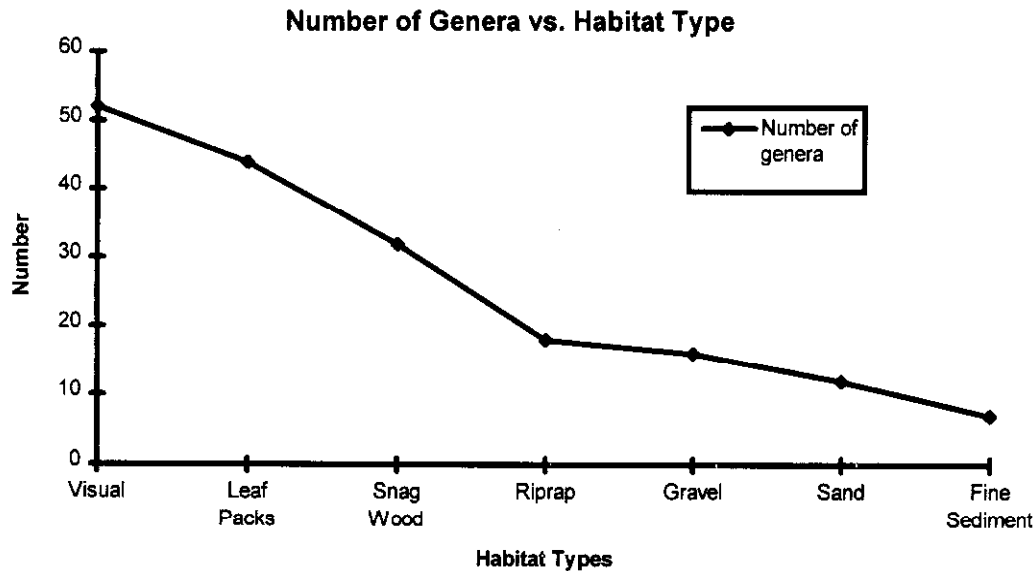
Macroinvertebrates have several advantages as indicators of ecological soundness. They are taxa rich, and occupy different niches. Many species have short generation time, making cohort analysis simple. Macroinvertebrates represent a range of tolerance to both pollution and habitat degradation. Functional groups of invertebrates, indicator species, species diversity, and biotic indices can help evaluate aquatic habitats. As demonstrated by the DEC project, numbers of genera vary significantly among habitat types (Fig. 2). Limiting invertebrate analysis to genera level identification reduces labor costs and level of taxonomic skill.

Fish and fisheries data are also excellent biotic tools in watershed evaluations. Fish tend to be taxa rich and occupy many niches. They occupy the role of top predator in aquatic ecosystems and are subject to bioaccumulation of toxic chemicals such as residual insecticides (Knight and Cooper 1991b). Because of trophic position, they are susceptible to any impairment that occurs within the food web. This makes them excellent indicators of ecological damage.

The potential for high species diversity makes fish good indicators of environmental change, but that diversity adds to difficulty of analysis. Calculations of indices of biotic integrity (IBI) and multivariate analysis are two popular approaches to fisheries data analysis. Multivariate analysis requires rigorous statistical analysis while IBI is more intuitive. Indices of biotic integrity are somewhat difficult to calibrate and may not relate well to physical changes in habitat. Classical fisheries characteristics have been the most consistent and reliable parameters used to evaluate DEC project watersheds. Difference in catch per unit of effort (CPUE), numbers per unit of effort, biomass, and average length were used successfully to evaluate bank protection techniques. Knight and Cooper (1991a) found that CPUE was significantly higher adjacent to transverse dike (stone groins) than CPUE from lateral stone bank paving.

DEC Examples: Shields et al. (1994) verified the water depth limiting effects of channel incision on warm water fisheries by detailed watershed surveys. They coupled results from individual experiments on improved management practices (Knight and Cooper 1991a) with survey data and demonstrated that fish species diversity could be increased by enhancing the scour hole formation using transverse dikes (Shields et al. 1993). By the addition of 18% more stone to extend the lengths of transverse dikes, fish species diversity and average fish length increased significantly. Cooper and Knight (1987) found that grade control structures provided stable deep water habitat which resulted in successful spawning and recruitment of both game and non-game fish species. Conversely, natural pools only produced sporadic spawning because of their instability. This led to modifications of grade control pools and an improved understanding of survey fisheries results.

Fig 2. *Number of macroinvertebrate genera found at each habitat type in the Demonstration Erosion Control Project watersheds.*



A case study from Otoucalofa Creek, Mississippi further shows how general survey and individual experiment results improve results. A rigorous woody debris removal to provide relief from urban flooding destroyed both pool and cover habitat, removed canopy and reduced fish biomass by 70%. Because baseline ecological survey results were available, habitat loss was isolated as the problem. Comparison of before and after fish data documented fish response. After three years the fish community stabilized at 60% below its pre-snag state. Specific experiments on lateral stream stability measures (Knight and Cooper 1991a) had shown that spur dikes created a mesocosm of habitat by forming scour holes at the tip of each dike. Thus, spur dikes were constructed as lateral stability/habitat improvement measures in the snagged reach. Within one year, fish populations recovered to within 20% of pre-snag populations. Also, results of the initial debris removal were inserted immediately into the project evaluation system so that an additional contract retained more canopy, cover, and woody debris.

A clear understanding of goals is needed to evaluate status or measure the success of any watershed management project. Although baseline information should cover a range of parameters that will highlight likely problems, evaluation should be directed toward variables that may be impacted by one or more management strategies. Sampling protocol should be sufficient to detect differences in highly variable parameters, such as invertebrate densities, fish species diversity, and suspended solid concentrations. A standard to measure watershed changes should be established. Reference sites may be used for comparison purposes if suitable non-impacted watersheds are located (Green 1979). The management goal of returning to some former pristine condition may not be practical, and evaluating project success against such a standard would be suspect.

SUMMARY

Evaluation of watershed management success can be difficult because of the complexity of agricultural watersheds. Ecological processes are inherently linked to both the biotic and abiotic components of a watershed; therefore, all management strategies have an impact on watershed ecology. Examination of a single indicator of environmental health does not provide enough information to adequately evaluate a watershed management strategy, but multiple indicators that account for major watershed components provide the best measure. Ecological responses to watershed management practices are typically detected in three major areas: habitat, water quality and biota. Watershed evaluations should be designed to provide the information necessary to "fine tune" a watershed management project. Evaluations that go beyond monitoring single or general watershed changes and test specific

components of the management project provide feedback that can greatly enhance the original management strategy.

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WILLOW POSTS CONSTRUCTION AND EVALUATION ON HARLAND CREEK, MISSISSIPPI

by

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Introduction

The erosion and degradation of alluvial stream banks have become one of the most significant sources of sediment transported through the river system. Sediments impact stream stability, habitat, ecosystem quality, recreation, and water quality in every watershed. One means of reducing the sediment inflow into the river system is to stabilize the stream banks. Commonly used revetments such as riprap and manufactured products have proven to be effective, but in most cases quite expensive. In recent years, bank stabilization methods have focused on bioengineering and vegetative techniques to enhance bank stability and reduce the loss of sediment to the river system.

One bank stabilization technique that is being evaluated throughout the United States is the installation of live willow posts. The propagation of plants through hardwood cuttings is one of the oldest methods for the revegetation of bare soils (Schiechtl, 1980). Observations of selected woody plants (i.e., willows) that can be propagated without roots are traced to 1781. It was suggested that the transplanting of poles that were cut from the main stem of a tree, 5 to 10 cm in diameter and 2.5 to 4 m in length, would not only survive but also flourish.

The suggestion of using live willow poles for stabilizing eroding stream banks emerged in 1925 (Engineering News Record). Later, Fry (1938) reported on the application of live willow on several restoration projects that occurred during the depression era. For example, live willow plantings were integrated into the Cook Creek watershed project where posts were placed for bank stabilization. Fry noted that during the first two years after planting, the willows were vulnerable to immediate flood and ice flows, and temporary protection may be warranted. After approximately two years, the willow appeared to be sustaining and effective in stabilizing the bank.

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Since Fry (1938), numerous applications of willows for streambank stabilization have been reported such as Dupre (1948) in Ohio, Carson and Preston (1976) in the northeastern United States, Workman (1974) in Montana, Roseboom et al. (1990, 1991, 1992) in Illinois, Bhowmik (1993) in Illinois and Shields et al. (1995) in Mississippi. From these experiences, consensus guidelines and criteria that may be considered are as follows:

- The stream bed should be stable and not subjected to progressive vertical change (i.e., 0.3 m or more).
- The proposed plantings must have access to direct sunlight.
- Source willow sites should be near the project site.
- Plant posts should be planted within 1 to 7 days from cutting.
- Posts should be cut from the main stem of the tree.
- Posts should be planted with bark on the stem.
- Drilling of post holes prior to post placement is recommended over post driving.
- Posts should be oriented as cut and once placed into the hole, fill soil around post.
- Sloping of the banks is recommended when possible.
- Plant the first row of posts at or adjacent to the low water level.
- Posts should not be smaller than 2.54 cm in diameter.
- The bottom of the post should be placed at the groundwater level and at least 40 percent of the post should be placed below ground level.
- It is advantageous to spray for insects at earliest opportunity after planting.
- Growth hormones are not recommended.
- It is not recommended that posts be placed in areas that vine plants (i.e., kudzu) will affect the plantings.

Consensus guidelines for appropriate soil conditions, optimal post spacing, optimal post sizes, and optimal post lengths have not been fully resolved. Further, the survival rates of the post plantings range from 0 to over 80 percent after two years. However, a target success rating has not been developed. Although there is agreement that live willow post plantings are an

alternative for stream bank stabilization, limited information and criteria are available to guide the user as to a) which site specific conditions lend to the successful establishment of willows; b) what is the acceptable lower limit of post survival that will provide bank stabilization; and c) which post sizes will maximize plant survival.

The objective of this presentation is to describe a case study in which willow posts were used to stabilize a reach of Harland Creek located near Tchula, Mississippi. Data were collected pertaining to willow post survival. The results of this study provide criteria that may assist in the use of willow posts at other locations.

Study Site

The study site is located on a segment of Harland Creek that is part of the Black Creek watershed contained within the Yazoo River basin, Holmes County, Mississippi as indicated in Figure 1. The study portion of Harland Creek is approximately 3.5 km in length and segmented into 14 reaches. The study site was selected because the bed is considered vertically stable, contains meandering as well as straight reaches, is not deeply incised (2-6 m), has bendways with active bank erosion, and is accessible. Channel top bank widths range from 23 to 46 m. The stream is routinely subjected to high intensity, short duration storms resulting in rapid changes in the stage. The adjacent properties are under cultivation or forested.

Planting Guidelines

Willow posts were placed in 8 of the 14 reaches of Harland Creek as indicated in Table 1. The creek reaches extended from 141 to 631 m in length. Two to five rows of willow posts were located on the outer bend of each meander comprising the placement of approximately 9,383 posts. The number of rows was based upon the bank height, degree of erosion activity, and steepness of the bank. Willow post planting guidelines were compiled by the U.S. Army Corps of Engineers Waterways Experiment Station (Derrick and Abraham, 1993) and used as the working specifications during post placement by the contractor. The willow post planting guidelines included:

- Native willows in good condition should be planted.
- Posts must be planted when dormant (i.e., December through February). Dormancy is defined as after the leaves have dropped and before the leaf buds have appeared.
- Posts should be cut, soaked in water and planted within 48 hours.
- At the time of cutting, the tops of the posts should be marked to assure planting in the upright position.

- Post diameter should be a minimum of 7.6 cm at the butt end.
- Minimum willow post length should be 3 m.
- Post spacing should be on a 1 m grid (i.e., 1 m between rows and 1 m between posts in each row).
- The willow post holes should be formed by a 20 cm diameter auger with a flat end.
- Posts should be planted from 2 to 2.4 m deep. Care must be taken to insure that the butt of the post is in contact with the water table.
- Not more than 1.2 m of the post should be above the ground.
- The butt ends of the willow should make contact with the bottom of the hole.
- Willows should not be planted above top bank.
- Willow posts should be planted for the entire length of the outer bank of each bend.
- The first row of willow posts should be planted at the water's edge.

The willow posts were cut, soaked and planted in January and February 1994 as specified throughout the 3.5 Km segment of Harland Creek. Posts were planted in 2, 3, 4, or 5 row configurations.

Monitoring

In May 1994, monitoring cross-sections were established starting at the downstream boundary of the Harland Creek study segment and extending upstream every 152 m. Eleven of the cross-sections intersected with willow post stabilized banks. Each cross-section was aligned with a column of the posts. A study sample of posts for each cross-section was comprised of the posts on the section, three columns upstream of the section and three columns downstream of the section. Therefore, a bank planted with three rows of willow posts would be represented by a sample of 21 posts. A summary of the cross-section locations is presented in Table 1.

Three monitoring efforts were performed to determine the survival of the willow posts. In May 1994, the 11 sections containing willow posts were monitored to determine only the survival rate of the sample posts. The second monitoring effort was performed in October 1994 while the third monitoring visit was in August 1995.

In August 1994, the diameter of each willow post was measured with a caliper to provide a data base for subsequent assessments. The post diameters were determined within ± 3 mm for both living and dead posts. The elevation of each post was determined using a hand level. The elevation was defined as the distance between the post-soil surface interface and the adjacent water surface elevation.

Results

The results of the Harland Creek willow post data collection effort will be summarized. It is important to acknowledge that these data are site specific and applicable for the planting specifications presented.

The willow posts were planted in January/February 1994. The May 1994, October 1994, and August 1995 survival rates were 81%, 43%, and 41%, respectively. A summary of the post survivals is presented by cross-section in Table 1. It is apparent that the willow post survival rate is lower than observed in other areas of the United States. The high mortality rate may be attributed to poor soil conditions, failing to backfill the posts when planted and prevailing flooding conditions that occurred during the first year. However, the post mortality appears to have stabilized. Approximately 7% of the posts (primarily front row) were not accounted in the August 1995 inventory.

Post Diameter

The willow posts sampled in the study segment ranged from 2.5 to 16.3 cm in diameter. The posts were categorized as indicated in Figure 2 for analyses. Posts 5.1 cm and smaller averaged a 31% survival rate. Posts 7.6 cm and larger exhibited a 51% survival rate. It appears that in a potentially harsh environment (i.e., poor soil conditions, canopy cover, etc.), the diameter of the willow post influences survival. In environments where the soil conditions allow drainage and aeration, the post size appears to have a lesser influence on survival.

The post distribution among the rows was reviewed to insure that the larger posts were not biased toward a more advantageous location (i.e., the larger posts were not all located on the first or second row). Willow post plantings, where either 2 or 3 rows existed, indicate an equal size distribution on each row. Locations in which 4 rows of posts were placed indicated a slight bias toward the larger diameter posts planted in the upper 2 rows while the smaller diameter posts were nearer the water surface. Five row plantings indicated that the first row and fifth row were biased toward the smaller post diameters. The distribution of post sizes appears to be generally random throughout the Harland Creek monitoring cross-sections.

Post Elevations

The elevation of the post-soil interface relative to the water surface was determined for each post in the sample. It was assumed that each post was approximately the same length and

that the post extended at least 1.3 m (4 ft.) into the bank or bed. Post-soil interface elevations ranged from 0.6 m to 2.3 m above the water surface elevation. The percent living was plotted against the elevation interval as presented in Figure 3. It is observed that posts placed in the stream or at the water surface elevation exhibited a low survival rate, less than 20%. Posts installed in the first row appear to be sacrificial. Posts that were positioned from approximately the water surface to 1.5 m above the water surface exhibited survival rates from 39% to 58%. Posts placed approximately 1.2 m above the water surface appears to optimize survival for this site. Post survival significantly decreased when the post-soil interface exceeded 1.5 m above the water surface.

Conclusions

The Harland Creek willow post planting project integrated the experience of several previous demonstration and erosion protection programs and applied the findings to a potentially harsh environment in northern Mississippi. Post survival one growing season after installation on Harland Creek was approximately 43%. Approximately 41% of the willow posts survived 1.5 years after planting. It appears as if the post mortality has stabilized after 1.5 years. The profession prescribes a survival rate of approximately 80% to be considered a successful erosion protection and/or bank stabilization effort. However, the lower survival rate may be acceptable for bank stabilization projects in harsh climates or locations. Each erosion protection and bank stabilization project is based upon site specific characteristics, therefore post survival rates may widely vary.

The Harland Creek project has resulted in the compilation and development of a comprehensive set of installation guidelines of willow posts for erosion protection and/or bank stabilization. A field monitoring program performed after post installation indicated that post diameters greater than 5.1 cm have a better chance of survival than small posts in harsh environments. Also, posts have the greatest chance for survival if placed above the low-water surface elevation.

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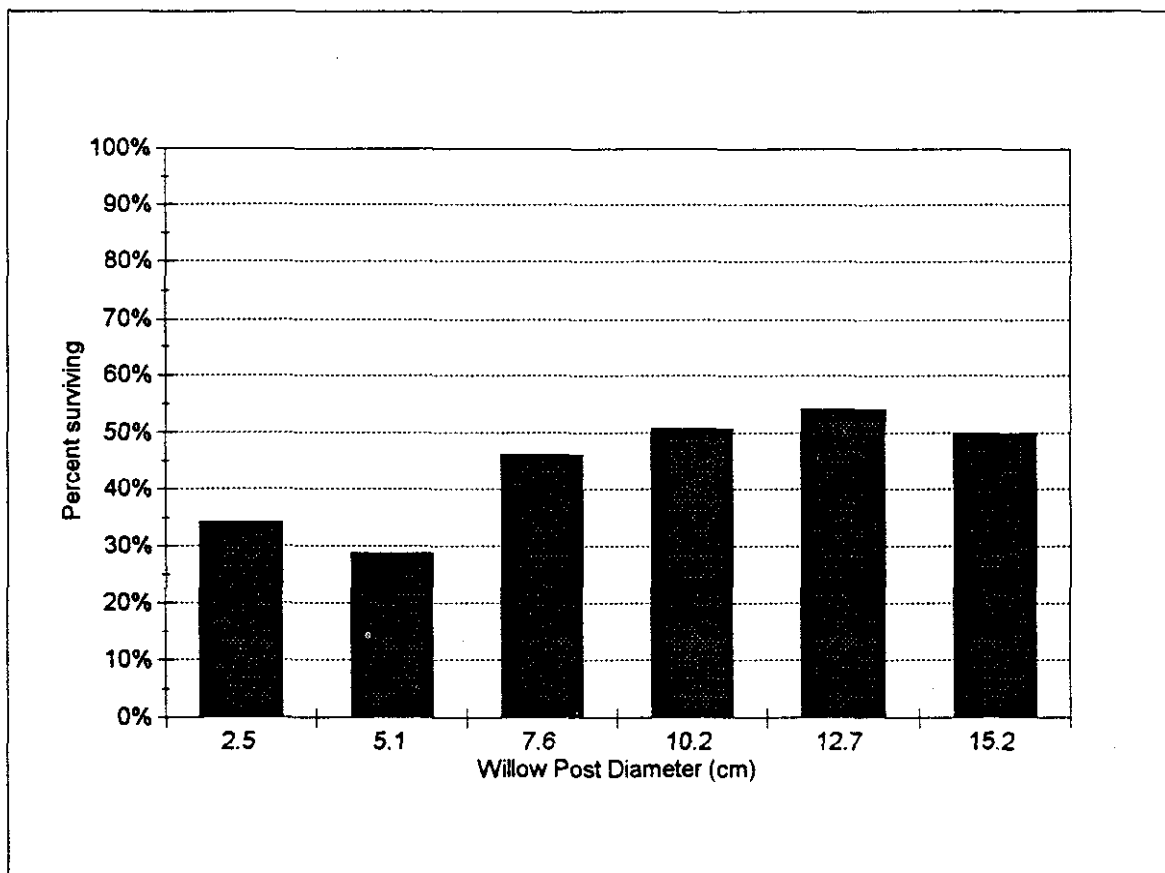


Figure 2 Willow Post Diameter Versus Percent Surviving (Oct. 1994)

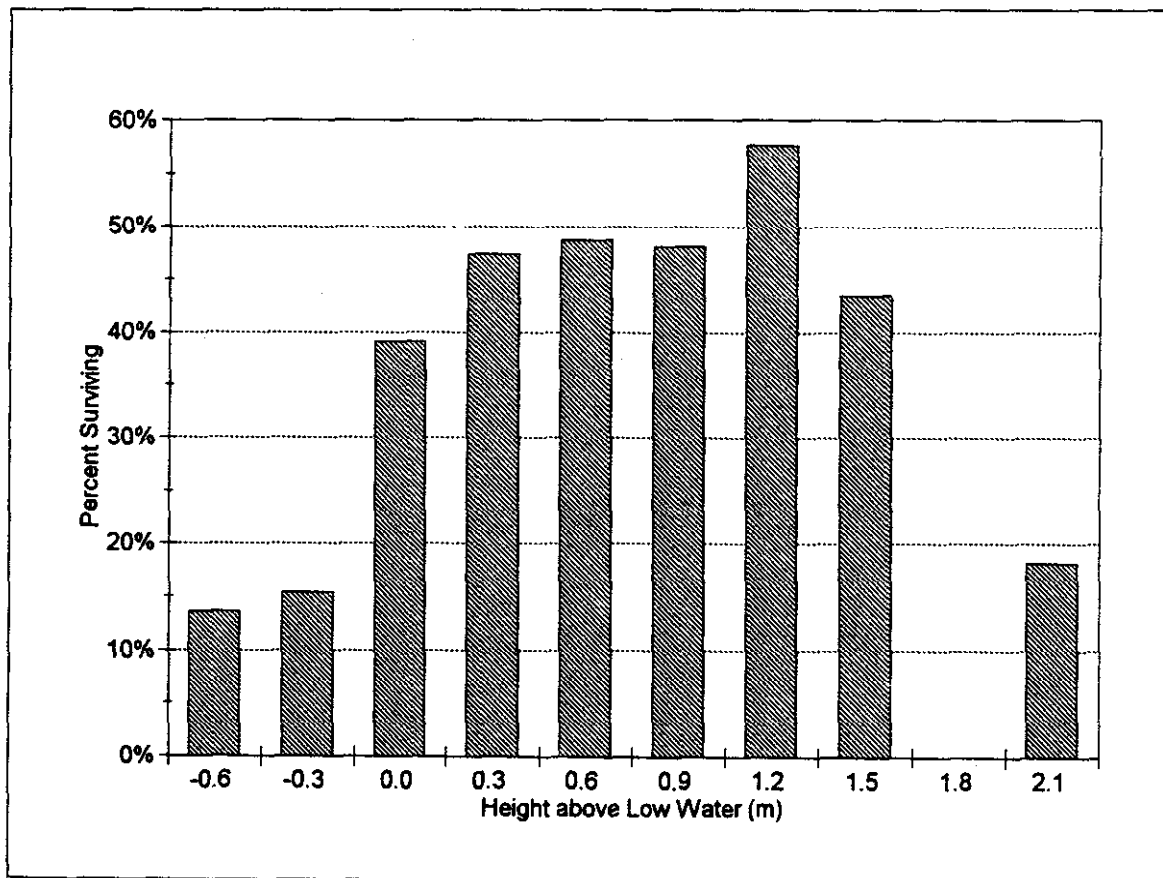


Figure 3 Post Elevation Above Water Surface Versus Percent Surviving (Oct. 1994)

Table 1 Summary of Willow Post Sampling

* Cross Section #	Bank L/R	No. Rows	% Survival	
			Oct. 94	Aug. 95
305	L	2	64	64
457	L	4	31	39
1067	R	5	23	23
1219	R	5	39	31
1676	R	5	29	17
1829	R	3	14	19
1829	L	3	71	86
1981	L	2	57	86
2134	L	2	79	71
2286	R	3	67	43
2438	R	5	31	43
2743	R	3	63	33

overall

43

41

* Measured from the downstream boundary of the study segment.

A TALE OF TWO STREAMS: RESTORATION STRATEGIES COMPARED

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Abstract: Many stream ecosystems are severely limited by damaged physical habitat. Channelization and associated accelerated erosion is a primary cause of damages in agricultural watersheds. Cost-effective strategies are needed to address erosion problems and restore stream corridor habitats. Detailed studies of restoration outcomes are rare. Herein we present a case study of two small streams (watershed size = 12 and 14 km²) damaged by channel straightening and incision. One stream was stabilized using a low drop grade control structure and dormant willow post planting, while the other was treated with a stone weir, stone toe bank protection, and willow sprout planting. Effects of restoration were monitored by collecting physical and biological data for one to two years before restoration and two to three years afterward. Following construction, channel planforms were stable, but up to 1 m of deposition and erosion occurred along the thalweg profile. Willow planting was not successful, so canopy, bank vegetation and woody debris density were unchanged. Pool habitat area increased from less than 5% to more than 30%. Fish species richness was unchanged, but species composition shifted away from cyprinids that occur in shallow, sandy runs toward pool-dwelling types (catostomids and centrarchids). Response to restoration was more modest than for two nearby restoration projects. Potential causes include less ambitious restoration design, greater initial degradation, and isolation from less-degraded sites which could serve as sources of colonists.

INTRODUCTION

Ideally, stream corridors in the agricultural landscape are multi-functional components, supplying visual amenities and recreational opportunities, trapping and processing contaminants, providing floral and faunal habitats, and conveying water and sediment downstream. In fact, previous watershed and channel management practices have emphasized conveyance functions at the expense of others, and have frequently accelerated channel erosion. For example, straightened, incised streams draining agricultural watersheds in Mississippi display shortages of pool habitats, stable bed material, woody debris, and woody riparian vegetation (Shields et al., 1994). Flood peaks tend to be sharper than for nonincised channels, and overbank flow, with its ecologically important exchange of nutrients and carbon with the floodplain, occurs rarely or not at all.

Interest in stream restoration, here defined as activities intended to restore lost functional values to stream corridors, has increased in recent years. At least 12 federal agencies administer programs with stream restoration activities (not including research) (Water Policy Branch, 1995) and many state agencies are also involved. Despite a wealth of available restoration techniques (Brookes, 1988), little information is available about the effectiveness of a given technique in a given setting. Scientific studies of warmwater stream restoration projects are scarce. Planning and design of restoration projects often proceeds by guesswork, intuition, political compromise, or with only single functions targeted. Herein we assess short-term effects of two restoration projects on channel stability and on fish and their habitats. The period of observation includes up to two years prior to construction and up to three years following construction of restoration measures. Effects are compared with other stream corridor restoration projects in agricultural watersheds in the same region. The objective of this study is to advance the state of restoration science pertaining to deeply incised, low-order warmwater streams draining agricultural watersheds.

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STUDY SITES

Martin Dale Creek (C) and Martin Dale Tributary (T), are incised streams draining adjacent agricultural watersheds of 12.3 and 13.6 km², respectively, in the upper Yazoo River basin in northwest Mississippi (Figure 1). Watershed relief is 50 to 60 m, and geological conditions are characterized by deep (up to 6 m) loess deposits overlying sand and clay (Simons, Li and Associates, 1987). No bedrock is exposed, and the only geologic controls along channels are outcrops of consolidated clay or cemented sand. During this study, valley bottoms (Falaya or Collins silt loam) were cultivated for cotton and soybeans, while hillslopes (silty and sandy soils) were wooded or in pasture (Huddleston, 1967). Both channels were nearly straight, with depths of about 5 m and top widths of about 20 m, and resembled stage II of the conceptual model of incised channel evolution presented by Schumm et al. (1984). Numerical simulation suggested that the channel capacity of C exceeded the 100-year flow (Simons, Li, and Associates, 1987). Bed material was sand with outcrops of hard clay. Along both channels, left banks were extremely steep, but right banks were benched. Thalweg slopes were about 0.002 to 0.003. At base flow, water widths ranged from about 1 to 7 m and median depths were usually less than 10 cm. Base flows were generally in the range of 0.005 to 0.050 m³s⁻¹. Woody bank vegetation was scarce, except along the lower reaches of T where there were some large trees on top bank. The exotic vine, kudzu (*Pueraria lobata*) covered much of both banks of both channels. Woody debris was scarce.

Streams C and T were treated using different stabilization measures. A low-drop grade control structure ("drop structure", Little and Murphey, 1982) was constructed on C (Figure 2), while a stone weir was placed in T, and a ridge of stone was placed along the toe of concave banks (Figures 3 and 4). The drop structure and stone weir created pools upstream that were up to 1 m deep. Downstream of the stone weir in T, dumped construction rubble (broken concrete, etc.) created a ~10% slope instead of a scour hole (Figure 3). In addition, about 3,000 willow sprouts (cuttings of no minimum size manually thrust 30 to 36 cm into soil) were planted landward of the stone toe along T, while 4,282 dormant willow posts 1.5 m long by 8-30 cm diameter were planted 1.2 m deep in both banks along 500 m of C (Figure 5). Construction dates were August 1992 through February 1993 and June 1991 through August 1991 for C and T, respectively. Costs for restoration measures are summarized in Table 1.

Table 1. Costs for Restoration Measures

Stream	Components	Cost (1991-1992) ¹	Cost per km ² of upstream drainage area
C	low drop grade control structure	\$178,467	
	4,282 dormant willow posts	\$17,128	
	TOTAL	\$195,595	\$15,902
T	stone weir, approximately 700 m of stone toe bank protection, and planting about 3,000 willow sprouts	\$165,000	\$12,132

¹ Personal communication, Mr. Phil Haskins, U.S. Army Corps of Engineers, Vicksburg, MS.

METHODS

Effects of restoration on channel boundaries were quantified by surveying channel thalwegs and cross-sections during 1991-92 and again in 1995. Bed sediments were grab-sampled at the center and quarterpoints of 12 transects across the baseflow channel of each stream during 1991 and again in 1994. Physical habitat data were measured semiannually (spring and fall) for four years using methods described by Shields et al. (1993b, 1994, 1995a). Counts of living and dead willow posts were made one year after planting. Survival of willow sprouts was assessed visually.

Fish were collected in spring and fall concurrently with physical data using a backpack-mounted electroshocker. Four 100-m stream subreaches within each 1 km study reach were fished for approximately 6 minutes (mean = 388 sec, std dev = 168 sec) of electric field application. Fishes longer than about 15 cm were identified, measured for total length, and released. Smaller fish, and fish that could not be identified in the field were preserved in 10 percent formalin solution and transported to the laboratory for identification and measurement. Water temperature, pH, dissolved oxygen, and conductivity were measured when fish were sampled.

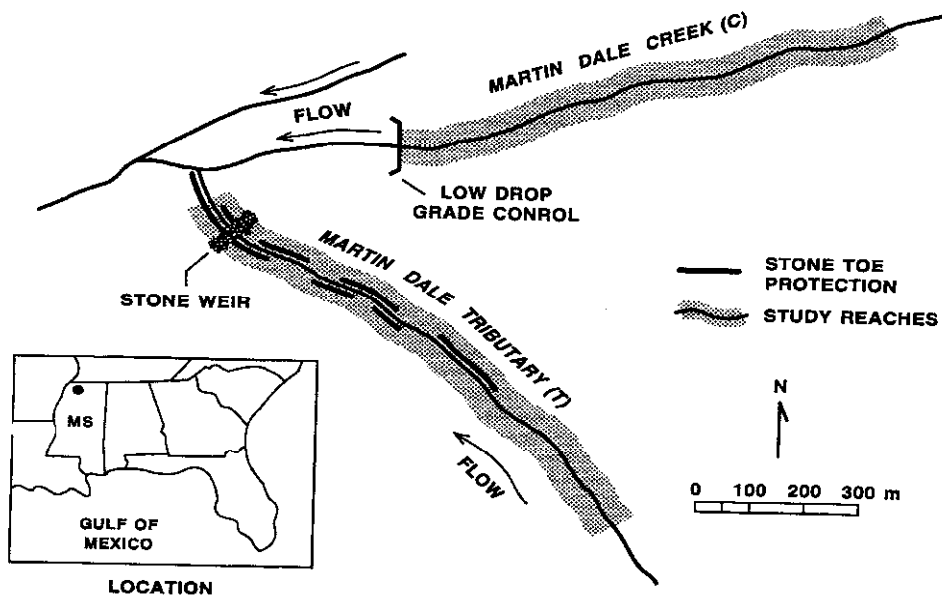


Figure 1. Study site locations.



Figure 2. Drop structure, C.



Figure 3. Stone weir, T.



Figure 4. Stone toe, T.



Figure 5. Willow posts, C.

RESULTS

Thalwegs were horizontally stable during the periods between channel surveys. However, thalweg profiles were dynamic, with up to 1 m of aggradation occurring upstream of the C drop structure (Figure 6). Comparison of 1995 and 1985 (Simons, Li and Associates, 1987) C thalwegs revealed that average slope was reduced from 0.0027 to 0.0021 by deposition above the drop structure and erosion further upstream. Aggradation upstream of the stone weir on T also extended about 400 m upstream, but was less uniform, ranging from 0 to 60 cm (Figure 7). In the aggraded reach, thalweg slope was reduced from about 0.002 to about 0.001, but no change was observed in thalweg elevations or slope upstream of the aggraded reach.

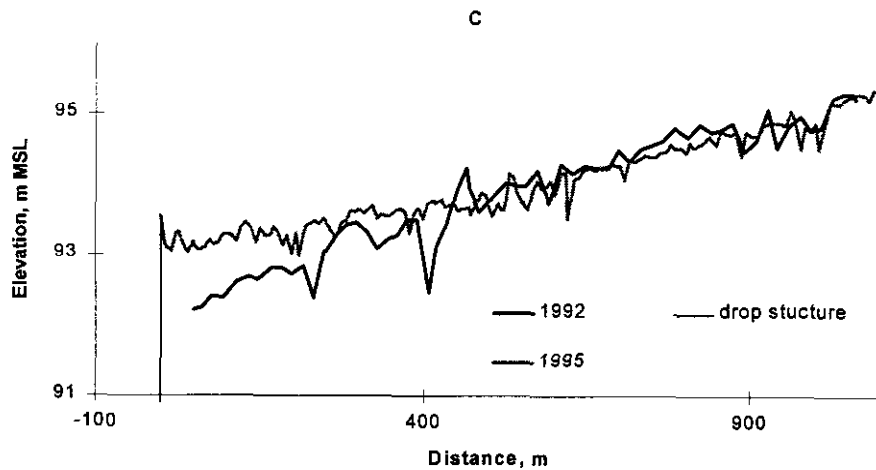


Figure 6. Thalweg profile, site C. Drop structure placed late in 1992.

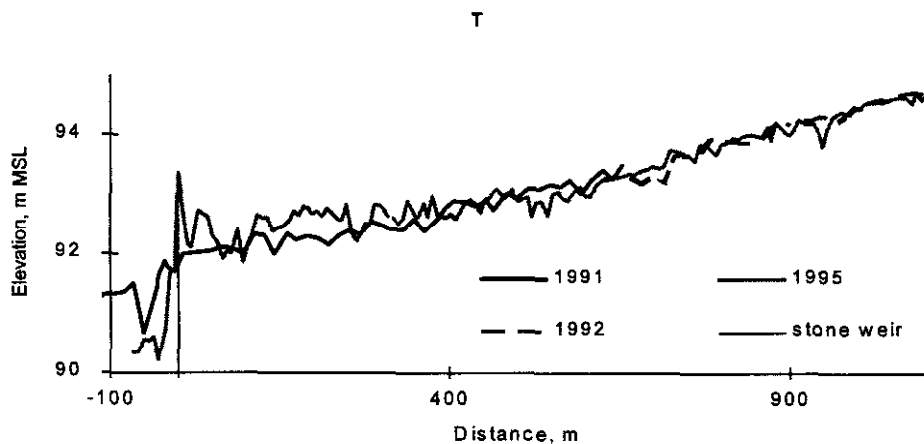


Figure 7. Thalweg profile, site T. Stone weir placed in summer, 1991.

Bed material size was unchanged following restoration: in both streams, median size ranged from about 0.1 to 0.8 mm, and hard clay outcrops were common. Gravel was absent.

Seven of the eight measured baseflows in T were greater (3 to 32 times) than those in C. Discharges in T ranged from 0.025 to 0.060 $\text{m}^3 \text{s}^{-1}$, while those in C were only 0.002 to 0.056 $\text{m}^3 \text{s}^{-1}$. Water width, depth, and velocity

measurements were not normally distributed, so median values for each site were compared using a Kruskal-Wallis one way ANOVA on ranks. Time was used as the independent variable, and differences in medians were found to be significant in all cases ($p < 0.0001$) except for T width ($p = 0.53$). Median water surface widths increased from 4.5 m to 6 m for C but fluctuated from 3 m to 6 m without apparent trend along T. Water depths in both channels were extremely shallow, but median values generally increased over the period of observation from less than 10 cm to more than 20 cm. Velocities declined over the same period, with median values decreasing from about 4 to 6 cm s^{-1} to 2 to 3 cm s^{-1} . T velocities were always greater than C velocities, as would be expected from the differences in discharge. The percent of habitat classified as "pool" (arbitrarily defined as depth > 30 cm and velocity $< 10 \text{ cm s}^{-1}$) increased from less than 5% to more than 30%, with the greatest increase along T (Figure 8). Most pool additions occurred upstream of the weirs and adjacent to stone toe.

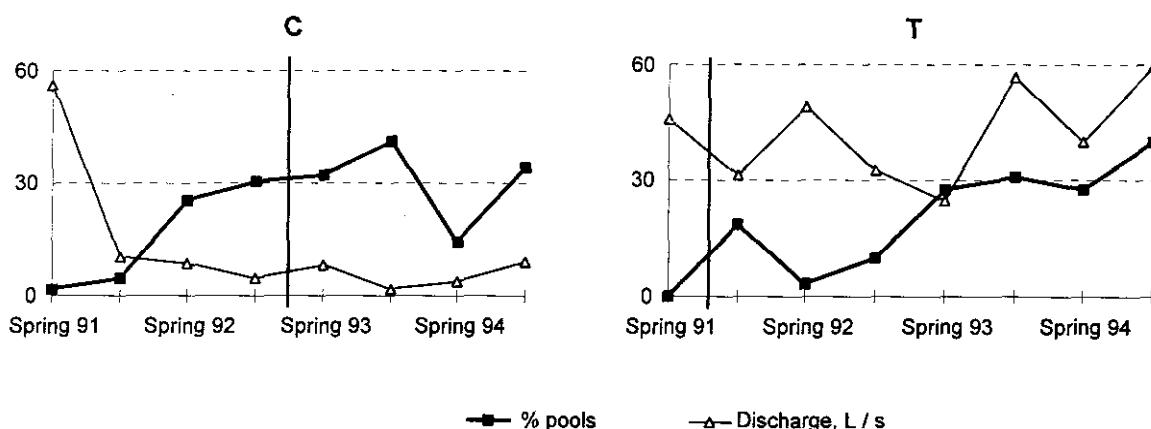


Figure 8. Discharge and availability of pool habitat versus time. Vertical lines indicate dates construction was completed. Fall 92 pool data for C reflect influence of temporary diversion dam placed during construction.

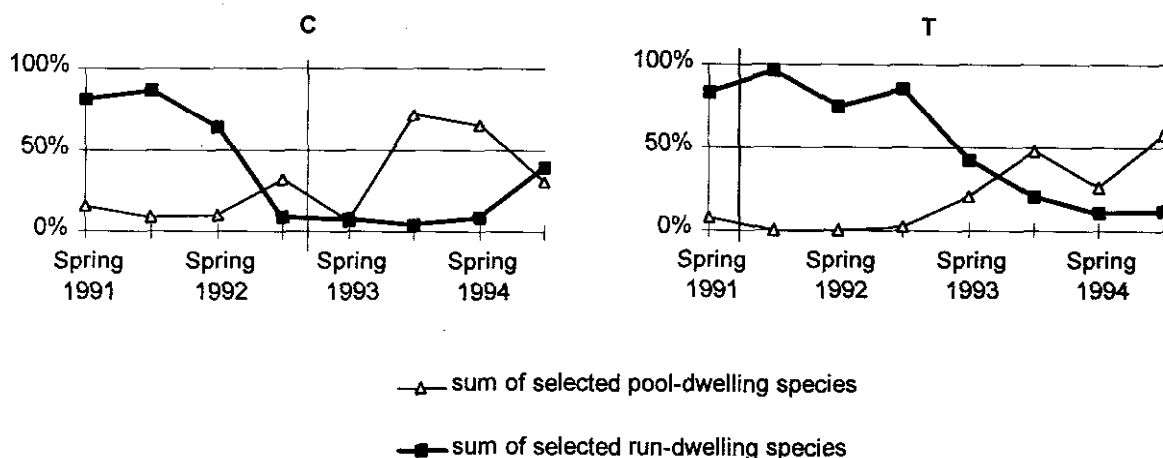


Figure 9. Relative abundance of selected fish species versus time. Pool species are bluegill (*L. macrochirus*), green sunfish (*L. cyanellus*), creek chubsucker (*E. oblongus*), and yellow bullhead (*A. natalis*). Run species are Yazoo shiner (*N. rafinesquei*), blacktail shiner (*C. venusta*), and bluntface shiner (*C. camura*). Vertical lines indicate dates construction was completed.

The frequency distribution of bed types varied with current and antecedent conditions throughout the period of observation but was always dominated by sand (~50 to 70%). Clay comprised ~30 to 40% of the bed surface. However, following restoration, stone riprap comprised 2 to 10% of the subaqueous bed along T, and detritus

increased from about 1% to a range of 3 to 7%. Detritus and woody debris became slightly more common along T. Changes were less pronounced along C, with sand becoming slightly more common (increasing from about 60% to 65%) and clay less common (declining from 39% to 31%).

Woody debris was scarce along both streams, but increased along C after placement of willow posts. The row of posts closest to the stream was quickly surrounded by water as the base flow channel migrated, and the posts furnished limited debris habitat. Fewer than 10% of posts survived 12 months due to infertile soils and competition by kudzu. Survival of willow sprouts on T was even lower, perhaps because they were planted during the summer. One to two beaver dams 50-75 cm high were found along both channels during periods of prolonged low flow. Shade canopy was nearly absent along C during the entire study, and declined slightly along T where trees were removed from the top of about 200 m of bankline prior to stone toe emplacement. Water quality data indicated conditions were suitable for aquatic life. Stream C tended to be about 2 Centigrade degrees warmer than T.

During the study, 10,056 fishes representing 23 species and six families were collected. Biomass ranged from about 0.4 to 4 kg per collection and averaged about 2 kg. Smaller fishes dominated all collections, with no individual longer than 24 cm or weighing more than 107 gm. Catch per unit effort (CPUE) declined from about 30 to 80 fish min^{-1} the first year to 10 to 20 fish min^{-1} the fourth year, reflecting reduced sampling efficiency as depths increased, and a shift in community structure away from large numbers of cyprinids. However, correlations of CPUE with median depth were not statistically significant ($p > 0.14$). The variance in CPUE between streams was not significant, but variance with time was significant ($p = 0.01$, two-way ANOVA). Biomass per unit effort did not vary significantly between streams or through time.

Species richness (number of species per collection) varied from 8 to 11 for T and from 5 to 11 for C without any trend. Differences between streams and sampling dates were not significant ($p > 0.56$). However, pronounced trends were observed in relative abundance at the family level. Cyprinids, principally the Yazoo shiner (*Notropis rafinesquei*) which prefers shallow, sandy habitats (Suttkus, 1991) dominated all collections, but became much less dominant in later years. Pool-dwelling catostomids and centrarchids increased in relative abundance as run-dwelling cyprinids declined (Figure 9). These changes were more pronounced for T than C. Median lengths of selected species known to grow longer than 10 cm were plotted versus time; no trends were evident.

DISCUSSION AND CONCLUSIONS

The difference in base flows and temperatures for C and T reflects greater groundwater inflows to T, where artesian inflows ("sand boils") were often observed in the bed. Aquicludes underlying many northwest Mississippi watersheds are not conformable with surface topography, leading to groundwater transfer and widely varying baseflows between adjacent watersheds (Grissinger et al. 1982). The similarity of the geometry of the two channels confirms that channel shape and size are defined by geomorphic and hydraulic factors independent of groundwater contributions.

The development of pool habitats along both streams is a positive sign, and attendant shifts in fish species composition may presage more positive biological developments. On the other hand, more than half of the increase in pool habitat occurred immediately upstream of the weirs, and these pools are rapidly being eliminated by sedimentation.

Previous restoration studies of warmwater streams damaged by channel incision have illustrated the positive effects of increasing pool habitat availability, riparian vegetation, and stony substrates (Shields et al. 1995a, 1995b, 1993a, 1993b). Results of these efforts are encapsulated in Table 2, and results of this study are presented in parallel form. The project described above was less successful than the others. We hypothesize that this was due to a combination of factors. First, initial habitat conditions at C and T were worse than for the other sites, as was evidenced by the low fish species richness and the dominance of Yazoo shiners (Shields et al., In Press). The absence of gravel substrate and the scarcity of woody riparian vegetation, woody debris, and stream-floodplain interaction evidently created stressful habitat conditions. Second, the study reaches are distant from habitats that could provide colonizing organisms. Third, restoration efforts were primarily standard channel stabilization practices applied

without consideration of habitat goals. The primary habitat feature, planting willow posts along C, was limited to a 500 m reach and was unsuccessful. Previous studies have highlighted the superiority of discrete structures like spur dikes over continuous structures like stone toe for aquatic habitat restoration (Shields et al., 1995c, but see Shields et al. 1995d). Recovery of habitat resources in deeply incised low-order warmwater streams draining agricultural watersheds is likely to be slow and minimal without a more aggressive approach to addressing environmental goals in channel stabilization design. Bed stabilization using standard approaches does not yield dramatic improvements in upstream habitat quality.

Table 2. Results of Restoration of One km Reaches of Incised, Warmwater Streams in Northwest Mississippi

Stream	H	G	T	C
Reference	(Shields et al., 1993a, 1995a)	(Shields et al., 1995b)	this study	this study
Watershed, km ²	91	21	14	12
Restoration measures	16 stone spur dikes, 3,400 willow posts ¹	18 stone weirs, 1,400 willow posts ¹	1 stone weir, stone toe protection, 3,000 willow sprouts	1 drop structure, 4,300 willow posts
length of observation before/after restoration (yr)	1/2	2/<1	1/3	2/2
Physical response, change in pool habitat ²	increased from 3 to 14%	increased from 11 to 61%	increased from 0 to 40%	increased from 5 to 34%
Biological (fish) response	species richness increased from 14 to 19, median fish length (all species) increased from 3.6 to 5.5 cm	Cyprinids declined from 74% to 32% of numerical catch, median length of five selected species increased	Cyprinids declined from >90% to <50% of numerical catch. ³	Cyprinids declined from >80% to <60% of numerical catch. ³

¹ Study areas were immediately upstream from low drop structures placed prior to restoration.

² Arbitrarily defined as all area with depth >30 cm (> 20 cm for H) and velocity < 10 cm s⁻¹

³ Species richness, abundance, and fish size apparently unaffected

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IMPACT OF IN-CHANNEL ORGANIC DEBRIS ON FLUVIAL PROCESS AND CHANNEL FORM IN UNSTABLE CHANNEL ENVIRONMENTS

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Abstract In this paper, degrading, unstable channels in northern Mississippi are examined to assess whether the input of Large Woody Debris (LWD) and formation of debris jams is significant in terms of channel stability management. US Army Corps of Engineers Demonstration Erosion Control data-sets have been utilised to identify significant debris-jams in planform and long profile at 23 river reaches so that changes in debris input volume, jam stability and the associated scour, backwater sedimentation and bank erosion/ accretion can be monitored through time.

Results indicate that the impact of debris jams varies primarily with jam orientation relative to the main flow direction. Impacts change from depth adjustment through bed scouring in small creeks, to width adjustment through lateral erosion in medium size streams, to negligible effects in the largest creeks. Processes, therefore, appear to be watershed-scale dependent and evidence suggests that the ratio of average riparian tree height to channel width can be used as an indicator of the likely impact that a jam will have on channel morphology and sediment routing.

The relationships between LWD formations and channel processes have been incorporated into a Drainage Basin Debris Management computer program, written in C++. Input data take the form of those variables found to be significant in terms of debris-channel interactions including channel width functions, average tree height/species parameters and sediment type. The output data given is based upon the relationships found in the current research and consists of recommendations, with explanatory notes, for debris removal, retention, relocation or input depending on the type of management strategy desired in a particular catchment or channel reach.

INTRODUCTION

Numerous papers have been written on the subject of Large Woody Debris (LWD) concerning input processes, spatial location within the channel network, and impact upon channel morphology, flow and sediment routing. The majority of studies have, however, been carried out in stable gravel-bed rivers, especially in isolated headwater reaches, although one or two studies, such as that by Gregory, Davis and Tooth (1993), have dealt with watershed-wide processes. Consequently management issues have only been addressed in headwater environments. The aim of this research is to obtain a better understanding of the watershed-wide impact of debris jams in stable and unstable channel environments. This is important because debris management is currently carried out using engineers' judgement on an ad hoc basis. For a coherent basin-wide debris management strategy decision makers need to know how important debris jams are in terms of inhibiting or promoting bed and bank scour, sediment transport and deposition. Prediction of debris build-up rates at run-of-the-river structures is also vitally important for correct operations and maintenance procedures. Debris can, for example, be a major problem at bridges where it can cause excessive pier scour, localised flooding and increase the pressure force on piers. Parola et al. (1994) investigated bridge failures after the 1993 Mississippi River Basin flood and found that debris induced lateral loading and scour and was a contributing factor to many of the failures. Debris can also cause maintenance problems at other structures such as locks, dams and weirs where, for instance, sluice gates can

become jammed open and the hydraulic capacity of spillways and conduits can be impaired. Large debris can also be a hazard to boats.

ANALYSIS

Method The study area is in the US Army Corps of Engineers Demonstration Erosion Control watersheds in the Yazoo Basin, northern Mississippi. Twenty-three channel reaches, from 4000 to 12000 feet long, with a range of watershed area of between 3 and 150 square miles have been surveyed. The reaches fall into several categories which are being compared with respect to LWD dynamics. These categories include stable/unstable reaches, straight/meandering reaches and reaches which have either a predominantly agricultural or wooded riparian strip. Thalwegs and cross-sections are surveyed through each reach once every 6 months. Debris jams in each reach are surveyed with the thalweg data to monitor their stability and the changes in associated bed scour and sedimentation. The survey data is plotted in graphical form and subsequent data-sets overlaid so that changes in debris jam position and thalweg topography are apparent.

Survey Results From field observations it is apparent that the main LWD input mechanism in these channels is tree topple due to bank failure. Also, in November 1993, over the period of one or two days, a heavy frost caused branches to tear off a large number of trees in the northern half on the DEC survey area causing a sudden influx of new debris material into many catchments. It appears, however, that because much of this load is composed of only limbs, rather than whole trees, it has been moved by high flows to previously established debris jams, rather than forming new sites of obstruction.

On a catchment-wide scale it is evident that major debris input regions and jam concentrations are to be found in laterally unstable reaches, especially downstream of knickpoints and knickzones. It is also apparent that the input of debris from the outside of actively migrating meander bends from both stable and unstable channels is significant as a large proportion of the total number of jams surveyed can be found at the apex of bends, while significant debris input in straight channels is limited to those reaches which are highly unstable. Meander apexes are also a preferential site for deposition of debris which has been floated from upstream. This is likely to be due to the propelling of debris to the outside of bends by centrifugal force and outward flowing secondary currents at the water surface. During high flow events debris then becomes snagged in vegetation or is pinned to the bank and deposited at its base as high flows recede.

In channels with a catchment area greater than 50 square miles, coherent jams appear unable to form as even the "key-debris" (whole mature trees) can be transported at the higher flows without becoming stuck in the channel. It appears, therefore, that there is a limiting catchment size (channel width) from which larger debris is made available to downstream areas. This has important management implications for controlling debris at "run-of-the-river" structures such as bridges, locks and weirs and dams, because at-source debris management (riparian vegetation management) can be limited to channels above a basin given size.

It is evident from the thalweg plots that debris-filled reaches have far more irregular bed topographies than those which are completely debris-free. Therefore reaches, with their debris induced pools and shallows and abundant nutrient supply from the decomposing woody material, will offer a more diverse habitat for aquatic flora and fauna than debris-free uniform reaches (see

Bilby & Likens, 1980). Localised bed scour around debris-jams appears to predominate where the sediment load is mainly sand grade or finer, while there appears to be more backwater and bar sedimentation around jams in reaches that have a gravel component to the sediment load.

Geomorphic field reconnaissance from the current research and previous studies (Wallerstein & Thorne, 1994) indicates that the impact of debris jams varies primarily with jam orientation relative to the main flow direction. Impacts change from depth adjustment through scouring in small creeks, to width adjustment through lateral erosion in medium size streams, to negligible effects in the largest creeks. Processes therefore appear to be watershed-scale dependant and field evidence suggests that the ratio of average riparian tree height (potential debris) to channel width can be used as an indicator of the likely impact that a jam will have on channel morphology and sediment routing.

A debris classification system, modified from Robinson and Beschta (1989), has been used to describe the observed geomorphic impact of debris jams throughout the drainage network. The progression of jam types is as follows:

Underflow jams; in small catchments where fallen trees span the channel at bank-full level. Local bed scour may occur under debris at high flows, otherwise the in-channel geomorphic impact of the LWD is minimal.

Dam jams : in channels which the average tree height to channel width ratio is rough equal to one, so that debris completely spans the channel cross-section. This type of jam causes significant local bank erosion and bed scour due to flow constriction, and backwater effects will cause sediment deposition in the lower energy environment upstream. Bars may also form immediately downstream of the jam.

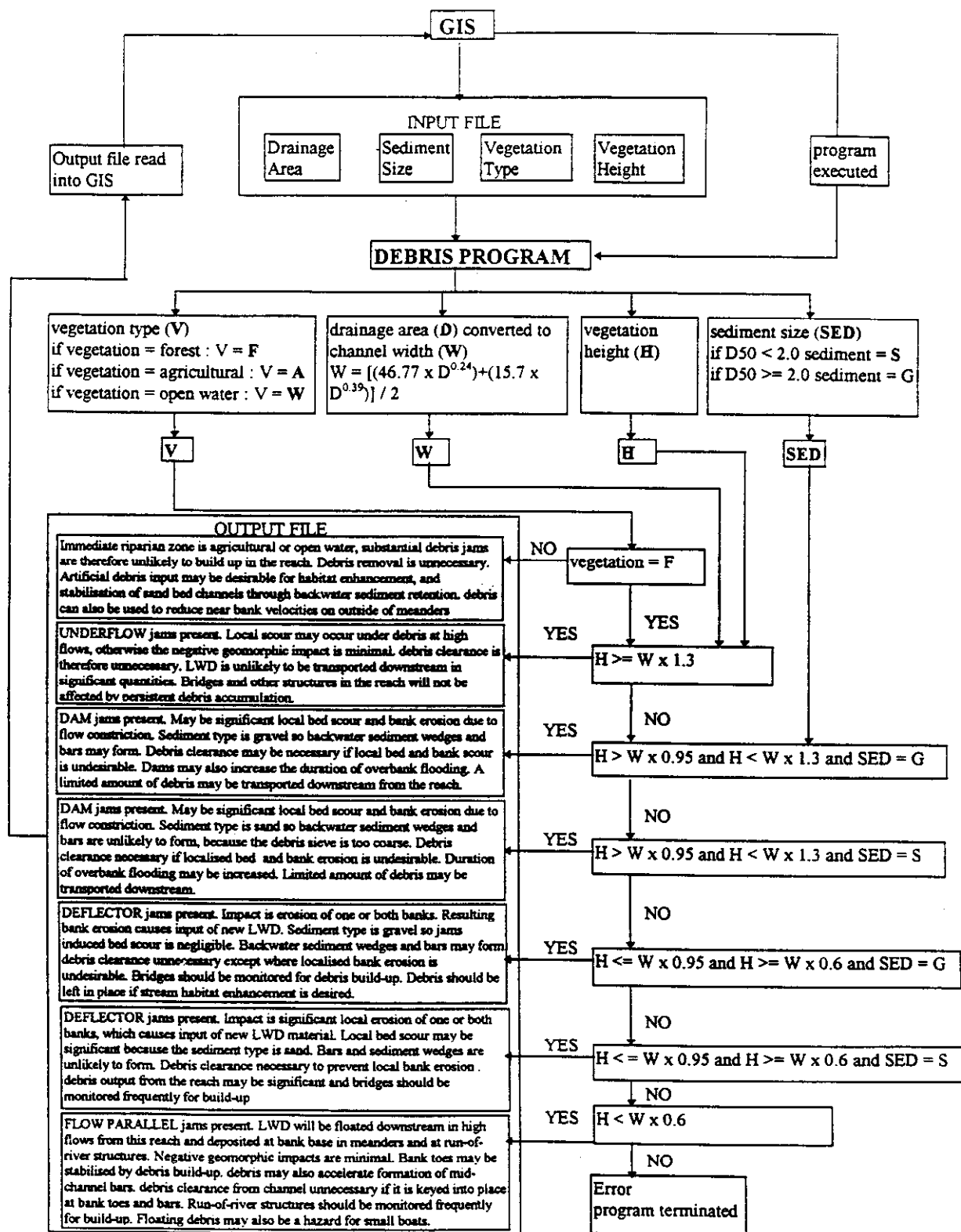
Deflector jams : found where input debris does not quite span the channel so that flow is deflected against one or both of the banks causing localised bed scour and bank erosion. Subsequent bank failure results in the input of new LWD material to the reach so that the jam builds up further. Backwater sediment wedges and downstream bars may form at this type of jam provided that stream power is dissipated by the jam below the critical level for the bed load and suspended sediment transport.

Flow Parallel jams : found where channel width is significantly greater than the key-debris length, and flows are competent enough to rotate debris so that it lies parallel to the flow. Debris is also transported downstream in high flows and deposited against the bank-base on the outside of meander bends or at channel obstructions such as man-made structures. Related bank erosion and bed scour will be minimal, and bank toes may even be stabilised by debris build-up. Flow parallel debris may also initiate or accelerate the formation of mid-channel and lateral bars.

LWD MANAGEMENT PROGRAM

Relationships from the field data have been used to create a computer program that has been coded

Figure 1. LWD management program flow diagram



Stream Erosion System

1. Title: Stream Erosion System

2. Author: [illegible]

3. Date: [illegible]

4. Location: [illegible]

5. Purpose: [illegible]

6. Method: [illegible]

7. Results: [illegible]

8. Conclusions: [illegible]

9. References: [illegible]

10. Appendix: [illegible]

11. Notes: [illegible]

12. Summary: [illegible]

13. Acknowledgments: [illegible]

14. Distribution: [illegible]

15. Comments: [illegible]

16. Revision: [illegible]

17. Approval: [illegible]

18. Date: [illegible]

19. Signature: [illegible]

20. Title: [illegible]

21. Author: [illegible]

22. Date: [illegible]

23. Location: [illegible]

24. Purpose: [illegible]

25. Method: [illegible]

26. Results: [illegible]

27. Conclusions: [illegible]

28. References: [illegible]

29. Appendix: [illegible]

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37. Date: [illegible]

38. Signature: [illegible]

39. Title: [illegible]

40. Author: [illegible]

41. Date: [illegible]

42. Location: [illegible]

43. Purpose: [illegible]

44. Method: [illegible]

45. Results: [illegible]

46. Conclusions: [illegible]

47. References: [illegible]

48. Appendix: [illegible]

49. Notes: [illegible]

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51. Acknowledgments: [illegible]

52. Distribution: [illegible]

53. Comments: [illegible]

54. Revision: [illegible]

55. Approval: [illegible]

56. Date: [illegible]

57. Signature: [illegible]

Key

Fine Sand (0.25mm)

Coarse Sand (15mm)

Gravel (2mm)

Scale 1:8205

Feet

1. Title: Stream Erosion System

2. Author: [illegible]

3. Date: [illegible]

4. Location: [illegible]

5. Purpose: [illegible]

6. Method: [illegible]

7. Results: [illegible]

8. Conclusions: [illegible]

9. References: [illegible]

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11. Notes: [illegible]

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14. Distribution: [illegible]

15. Comments: [illegible]

16. Revision: [illegible]

17. Approval: [illegible]

18. Date: [illegible]

19. Signature: [illegible]

20. Title: [illegible]

21. Author: [illegible]

22. Date: [illegible]

using C++. Input variables are riparian landuse type, average riparian vegetation height, channel width (determined from function of drainage area), and channel sediment type. The ratio of tree height to channel width is used to determine the debris jam type. The precise limits of each jam type have been determined from empirical and observational evidence from the survey sites. The magnitude of sediment retention and bed/bank scour at each jam is determined from the jam type and the D_{50} of the sediment present.

This model has been coupled with an GIS front end to demonstrate its potential use as a tool for integrated river basin management. The GIS has been created for the Abiaca Watershed, which is located within the DEC study area in northern Mississippi, and has a drainage basin area at its downstream end of about 150 square miles. This watershed was selected because it encloses four debris jam survey sites so that the model can be calibrated against actual field results. The program model outline is shown in Figure 1. The GIS has been created in the UNIX version of ARC INFO and comprises four layers; The drainage network (vector data); The road network : used to determine bridge locations (vector data); The landcover type : agricultural, open water or wooded (raster data); Soil type : used to determine primary channel sediment size (raster data); Digital terrain model : used to calculate drainage basin area (raster data). The GIS has been built with a tool bar so that layers can be displayed and analysis performed by simply using the computers' mouse. On-line help files are also included. Figure 2 shows a screen shot of the Abiaca Creek GIS displaying the drainage network, roads and landuse layer, the tool bar (top left) and a help text box.

To perform analysis using the GIS, for example to determine the best management strategy for woody debris at a particular bridge location, the following steps are taken. First zoom into the area of interest and use the mouse to mark the point of interest on the channel network. Next the GIS extracts the relevant values from its database for that location and passes then to an input file, the analysis program is then automatically activated and reads the input file, calculates the result, and produces an output text file. Finally the output file is automatically read into the GIS and displayed on the screen. The output file displays the parameters that were selected, the calculated channel width, the expected geomorphic impact of LWD jams at the selected location, and the suggested debris management strategy with warnings about debris impacts at structures in the reach. Figure 3 shows a screen shot of the GIS displaying a Debris Management Output file returned from the analysis program.

A second version of this program has also been written which calculates scour at bridge piers due to debris build-up. The calculations are based upon a model suggested by Melville and Dongol (1992). Data input for this program is via the keyboard, and the various parameter values are taken from the bridge site of interest.

CONCLUSIONS

The impact of LWD varies spatially across the drainage basin, and is also dependant on channel stability sinuosity, riparian land type (debris availability), and channel sediment properties. Integrated basin management could be enhanced by using secondary GIS data and predictive programs such as the one discussed in this paper to provide instant preliminary information about channel geomorphic and hydraulic conditions for a large number of channel reaches. Site specific surveys could therefore be conducted in a more informed manner. Modifications will be made to this

LWD analysis system to incorporate channel sinuosity, the degree of channel stability, and also the type of run-of-the-river structure encountered. The program will also take into account debris source and sink areas upstream of the selected site, so that estimates can be made of the potential volume of debris that could arrive at in-channel structures.

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DETERMINATION OF STABLE CHANNEL SEDIMENT LOADS IN DEMONSTRATION EROSION CONTROL PROJECT WATERSHEDS

Nolan K. Raphelt¹, Daniel Gessler², Chester C. Watson³

Abstract

The U.S. Army Corps of Engineers, Demonstration Erosion Control project (DEC) is actively involved in the stabilization of many incising streams in central and northern Mississippi. The objective of the project is to stabilize the channels by reducing their sediment transport capacity. One decision that must be made when designing such a project is what the target sediment transport rate should be. This paper presents a methodology for computing stable channel sediment loads that has been developed for streams in central and northern Mississippi.

Introduction

The Demonstration Erosion Control Project (DEC) provides for the development of a system for the control of sediment, erosion, and flooding in the foothills area of the Yazoo Basin, Mississippi. Structural features that are utilized in developing rehabilitation plans for the DEC water sheds include: grade control structures, pipe drop structures, bank stabilization which includes longitudinal riprap and transverse dykes, bio-engineering technologies, and combination of detention and retention reservoirs. In addition, other features such as levees, pumping plants, land treatments, and developing technologies may be utilized.

Evaluation of the performance of these features can contribute to the improvement and development of design guidance. Most of the previous Yazoo basin evaluations were limited to a single visit and data collection effort, with no comprehensive long term monitoring of the structures or their impact on channel stability. In December 1992, field monitoring of 20 DEC study sites was begun with a field data collection effort that has been repeated annually through 1995. In general the climate, geologic and geomorphic features of the streams are similar. Each monitoring site is at least 4000 feet in length with cross sections surveyed approximately every 500 feet. Sediment samples have been collected and analyzed at each site. These data allow for the computation of hydraulic and sediment transport characteristics of the stream using HEC-2 (USACE, 1982) and SAM (Thomas et al., 1994) respectively. In addition, geomorphologic data are logged and the reaches are classified in accordance with the channel evolution model from Schumm et al. (1984).

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Channel Evolution Model

The Channel Evolution Model (Schumm et al., 1984) was proposed as a conceptual model to describe the evolution of incised channels. Though the model is applicable only in a system wide context in which both lateral and vertical channel adjustment are possible, it has helped develop an understanding of the DEC watersheds. The model was originally used to describe the erosion evolution of Oaklimer Creek, MS, a tributary of the Tippah River in Northern Mississippi. Location for time substitution was used to generate a five reach type, incised channel evolution sequence for streams of the Yazoo Basin as shown in Figure 1. In each reach of an idealized channel, types 1 through 5 occur in series and, at a given location, will occur in the channel through time. The CEM describes the channel systematic response of a channel to base level lowering, and encompasses conditions that range from disequilibrium to a new state of dynamic equilibrium. The following paragraphs characterize the conceptual reach types. It should be recognized that these categories are only conceptual and variation may be encountered in the field.

Type 1 reaches have a sediment transport capacity that exceeds the sediment supply, resulting in scour of the bed. The bank height, h is less than the critical bank height, h_c , and the reach typically contains small precursor knickpoints.

Type 2 reaches are located immediately downstream of the primary knickpoint and are characterized by: a sediment transport capacity that exceeds the sediment supply, a bank height that is greater than the critical bank height, ($h > h_c$), little or no bed sediment deposits, a lower slope than the Type 1 reach and lower width to depth ratio than the Type 1 reach because the depth has increased but the banks have not failed.

Type 3 reaches are located downstream of Type 2 reaches and are characterized by a sediment transport capacity that is highly variable with respect to the sediment supply. The bank height is greater than the critical bank height, ($h > h_c$), erosion is primarily due to slab failure (Bradford and Priest, 1980), bank loss rates are at a maximum, and bed sediment accumulation is generally less than two feet, but can locally be greater due to local erosion sources. The channel depth is somewhat less than in Type 2. Channel widening is occurring due to continued bank failure.

Type 4 reaches are downstream of Type 3 reaches and are characterized by a sediment supply that exceeds the sediment transport capacity. The result is aggradation of the channel bed, a bank height that approaches the critical bank height with a rate of bank failure that is lower than that of Type 3 reaches, a nearly trapezoidal cross-section shape, and a width to depth ratio no less than that of Type 2 reaches. Bank failures have increased the channel width and in some reaches the beginnings of berms along the margins of an effective discharge channel can be observed. These berms are the initiation of natural levee deposits that form in aggraded reaches that were over-widened during earlier degradational phases. Bradford and Priest (1980) observed that in the later phases of evolution, the mode of bank failure changes from circular arc to slab-type failures.

INCISED CHANNEL EVOLUTION PHASES

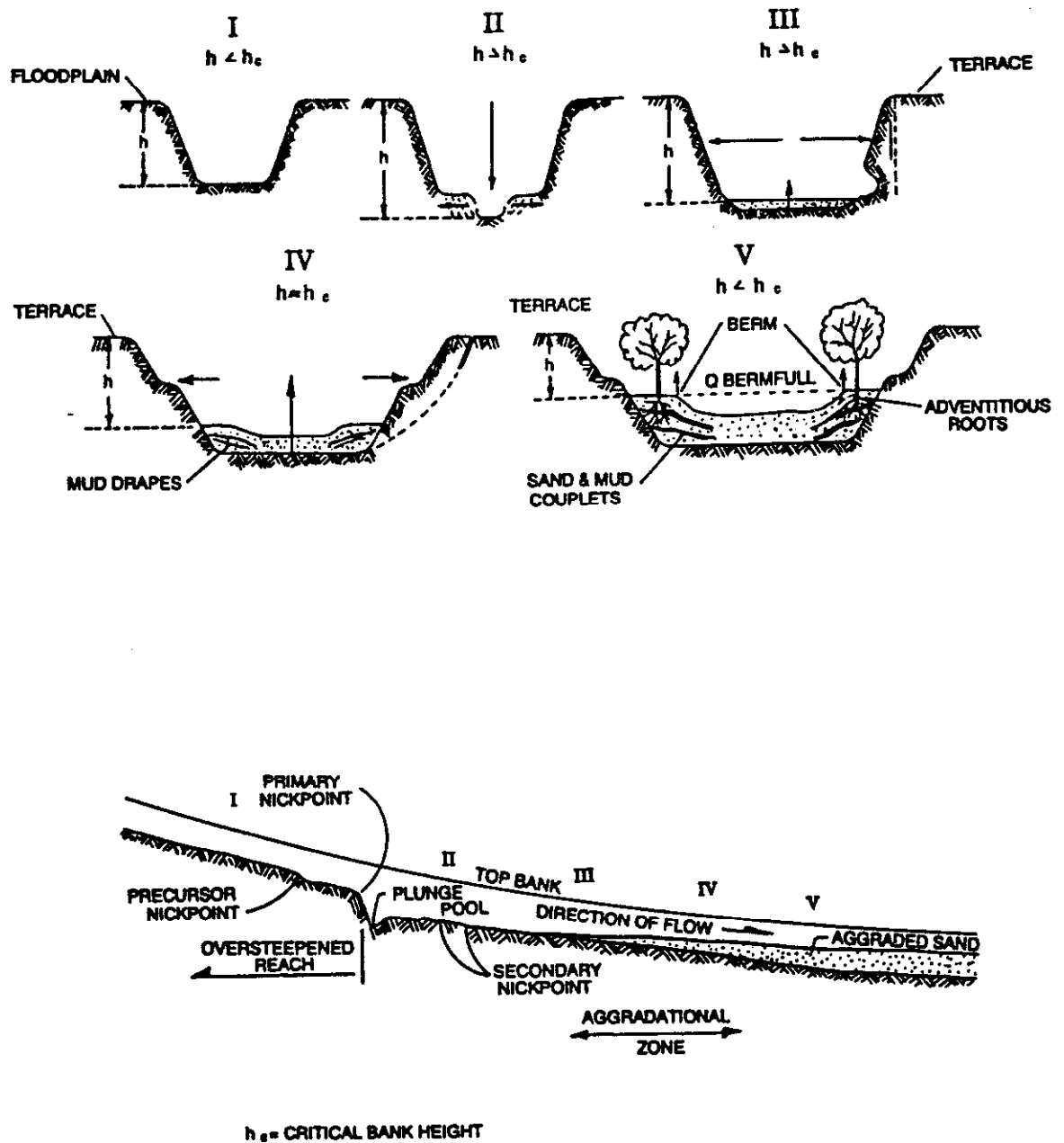


Figure 1: Incised Channel Evolution Sequence (after Schumm et al., 1984).

Type 5 reaches are located downstream of Type 4 reaches and are characterized by a dynamic balance between sediment transport capacity and sediment supply for the effective discharge channel. The bank height is less than the critical bank height, there is colonization by riparian vegetation, accumulated bed sediment depth generally exceeds three feet, the width-depth ratio that exceeds that of the Type 4 reach, and generally a compound channel is formed within a newly formed floodplain. The channel is in dynamic equilibrium. Bank angles have been reduced by accumulation of failed bank materials at the toe of the slope and by accumulation of berm materials.

The sequence of channel evolution is based on the assumption that the observed changes in channel morphology are due to the passage of time in response to a single base level lowering without changes in the upstream land use and sediment supply from the watershed. Application of the sequence assumes that the materials forming the channel perimeter are erodible and all degrees of channel adjustment are possible. The sequence is applicable only in a system wide context. Local erosion such as that caused by bends or deflection of flow by debris, may cause difficulty in application of the sequence.

The primary value of the sequence is to determine the evolutionary state of a channel from field reconnaissance. The morphometric characteristics of the channel reach types can also be correlated with hydraulic and geotechnical parameters (Watson, et al., 1988). The channel evolution model assists in the understanding that reaches of a stream may differ in appearance, but channel form is related to other reaches by an evolving process. Form, process, and time relate dissimilar reaches of the stream.

SAM

The Hydraulic Design Package for Channels is a computer program that was developed by the U.S. Army Corps of Engineers (Thomas et al., 1994). The purpose of the program is to provide the design engineer with a tool to help in sediment transport calculations and the design of stable flood control channels. The program is organized in modules. Sam can compute the effective width and depth of for either a single cross section or for a channel reach. The reach averaged effective width and depth are computed are computed from a HEC-2 output file. The sediment transport capacity for the effective width and depth of the channel can be computed from 1 of 13 different sediment transport formulas. An expert system within SAM helps the user select the appropriate sediment transport functions. Each of the 20 DEC stability study reaches was subdivided into segments of constant slope and fairly uniform cross sectional geometry. For each segment the CEM model type was determined from field observations. The sediment transport capacity for each segment was determined using SAM. A comparison of the CEM model type with the sediment transport rate for each segment is shown in Figure 2. The maximum, minimum and average sediment concentrations are shown for each reach type. The average sediment concentration in the Type 5 streams is approximately 1000 mg/l at the lesser value of the two year or bankfull discharge.

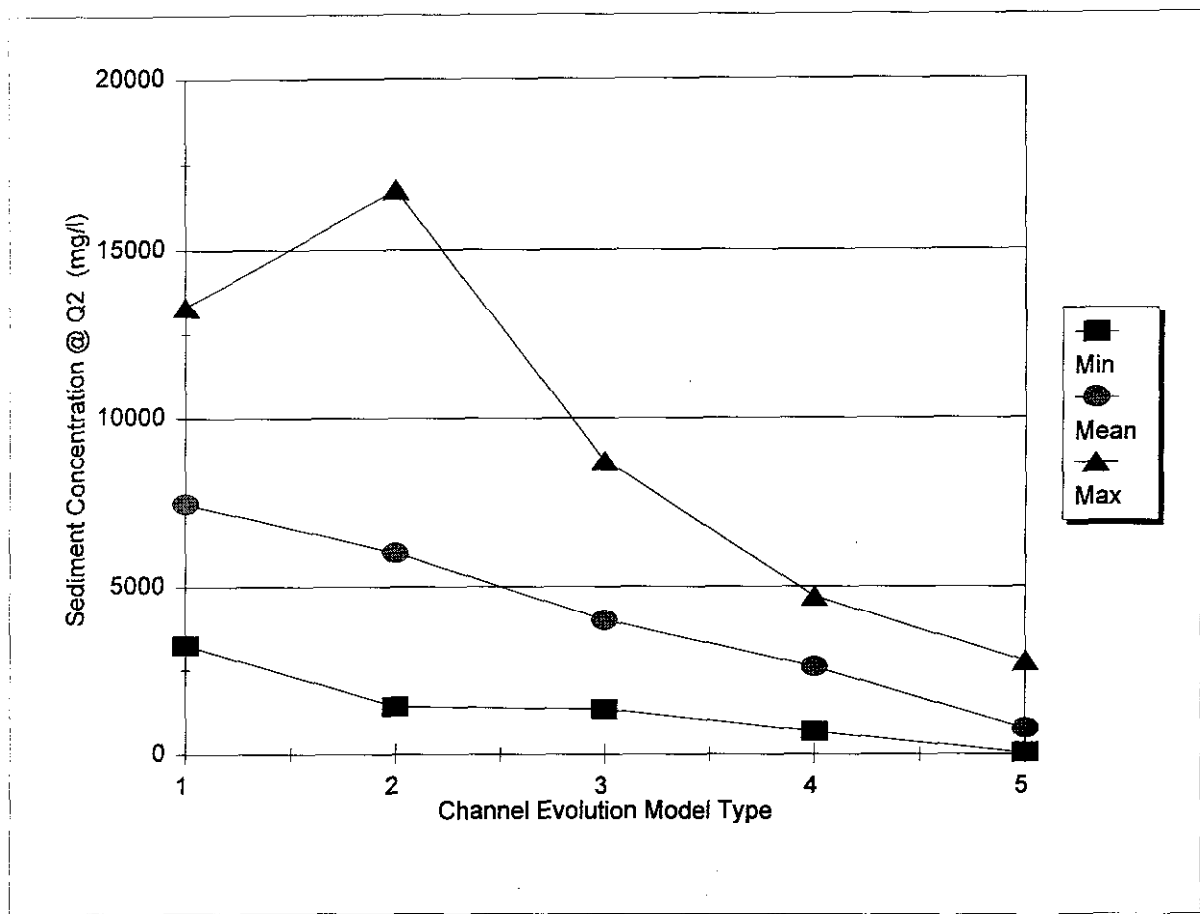


Figure 2: Sediment concentration as a function of CEM type.

Conclusions

A comparison of sediment concentrations compute by SAM to field observed CEM types was made. It was shown that for the 20 DEC stability sites analized the stable channel sediment load is approximately 1000 mg/l at the two year discharge.

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DEVELOPMENT OF DESIGN TOOLS FOR DETERMINING BANK STABILIZATION REQUIREMENTS

Nolan K. Raphelt¹, Chester C. Watson², Dan Gessler³

Abstract

The U. S. Army Corps of Engineers, Demonstration Erosion Control project (DEC) is actively involved in the stabilization of many channelized streams in central and northern Mississippi. When a stream is stabilized, the objective is to stop further incision, reduce the sediment transport capacity of the stream and prevent further bank failure. This paper demonstrates the use of two development computer programs that can be used cooperatively to aid designers of channel stabilization measures. The computer program SAM is used to compute width and slope combinations that satisfy a specific sediment transport rate. The computer program BURBANK is used to determine bank stability in the study reach, as a function of bank height, bank angle and the soil characteristics in the region. An example application of the procedure is presented.

Introduction

The U.S. Army Corps of Engineers is currently involved in a Demonstration Erosion Control (DEC) project. The objective of the project is to develop and implement methods for reducing erosion in actively incising streams. Many of the streams being studied were channelized straightened during the early part of this century. This resulted in an increased channel slope and sediment transport capacity. Without an increase in the sediment supply from the surrounding land, the streams incised. Decreases in bed elevation on the order of 10 to 15 feet have resulted in wide spread bank instability and failure. Most of the streams have base flows of less than 200 cfs. The two year event is typically on the order of 10 to 30 times greater than the base flow. The following is a summary of two computer programs, SAM and BURBANK, a presentation of a flow chart that illustrates the interaction between the programs, and an example application of the method to Red Banks Creek, a severely incised stream in Mississippi.

SAM

The Hydraulic Design Package for Flood Control Channels is a computer program that was developed by the U.S. Army Corps of Engineers (USACE, 1993). The purpose of the program is to provide the design engineer with a tool to help in sediment transport calculations and the design of stable flood control channels. The program is comprised

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calculations and the design of stable flood control channels. The program is comprised of three major modules, each performing a part of the calculations. SAM can compute the normal depth of flow for a specific channel geometry, or utilize the output file from a HEC-2 output file, the average effective width and depth is computed for all of the cross sections in the deck. The Stable Channel Design module solves Brownlie's roughness equation and sediment transport function simultaneously, generating a table of width and slope combinations that satisfy both equations. The bank angle and roughness must be specified for these calculations. The Sediment Transport Module of SAM computes sediment rating curves using one of 13 sediment transport equations. An expert system is included in SAM to aid the user in the selection of the appropriate sediment transport function.

BURBANK

The BURBANK computer program was developed at Colorado State University under contract to the USACE to compute bank stability (Burgi, 1995). BURBANK is designed to read a HEC-2 input deck and compute the left and right bank stability at each cross section. The program requires the user to specify the soil characteristics. Each cross section is checked for slab and rotational failure. If the factor of safety by either method is less than one, the bank is denoted as being at risk, otherwise, the smaller of the two factors of safety computed is output. The user can also specify a general lowering of the bed elevation and compute the bank stability for the new channel geometry. When the user specifies a general lowering in the bed elevation, the rate at which the bottom width incases as a function of the amount of degradation must be specified.

Design Flow Chart

The computer programs, HEC-2, SAM, and BURBANK can be used cooperatively to design a channel restoration or stabilization plan. The interaction between the programs is best illustrated with the flow chart shown in Figure 1. The following is a description of each item in the flow chart:

Data Collection -

1. Survey to create a HEC-2 input file
2. Sample bed sediment to determine the grain size distribution for use in sediment transport calculations
3. Determine friction angle, specific weight, and cohesion of the bank material
4. Develop design discharge

HEC-2 - HEC-2 is run for the channel to compute the existing hydraulic characteristics for the reach being analyzed.

SAM- The output file from the HEC-2 analysis is used by SAM to compute the existing sediment transport rate. Stable channel dimensions, width and

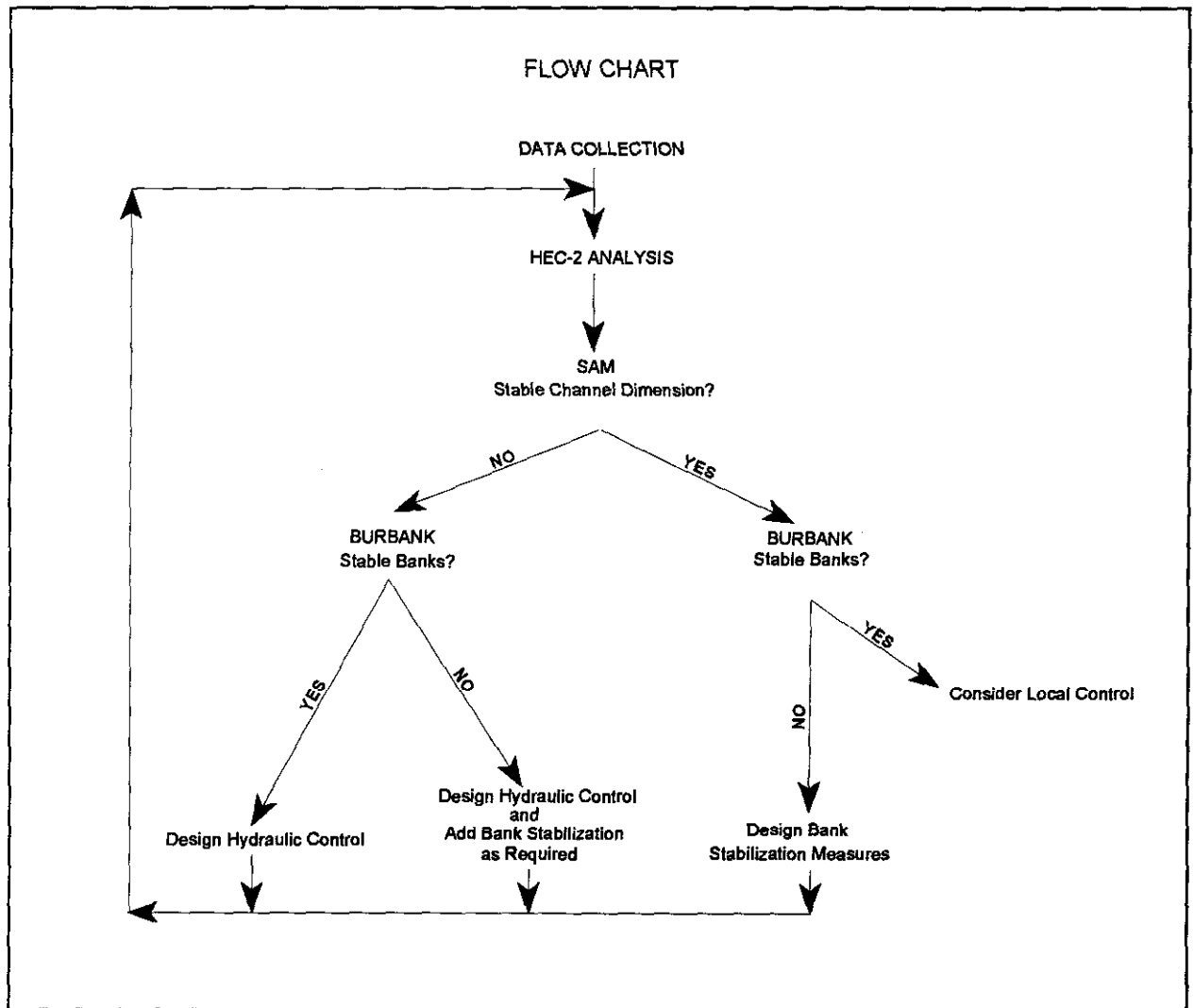


Figure 1: Flow Chart for use of HEC-2, SAM, and BURBANK in the design of stable channels.

slope, are computed for the existing channel at the target sediment transport rate.

BURBANK - The input file for the HEC-2 analysis is used by BURBANK to develop bank stability along the reach.

Based on the results of the analysis using SAM and BURBANK, one of four design scenarios should result: design of grade control, design of grade control and reach wide bank stabilization, design of reach wide bank stabilization, or design of local bank stabilization to counter the effects of local scour. After designing the channel stabilization measures, the new channel is re-analyzed in the same manner as the original channel was analyzed. Modifications are made to the design until it meets the design objectives.

Example

Red Banks Creek is located in northern Mississippi in the Coldwater River basin. The channel bottom width varies from approximately 75 to 150 feet. As a result of channelization, the stream is actively incising. It is typically at least 15 feet from the top bank elevation to the thalweg. The basin is generally comprised of silts and clays overlaying fine sand. The mean particle diameter of the bed material is approximately 0.5 mm. The stiff silts and clays that comprise the bank material have a cohesion of 356 lbs/ft² (March et al., 1993), resulting in very steep banks. The reach of interest is 5000 feet in length (Watson et al., 1995).

A target sediment concentration of 1000 mg/l during two year event was selected as the design concentration. The two year event has a magnitude of 3950 cfs. A HEC-2 analysis was used to compute the depth of flow followed by a SAM analysis to compute stable channel dimensions during the two year event at the design concentration. The stable channel dimensions are shown along with the existing width and slope in Figure 2. The plot shows that the stream is 50 percent steeper than the predicted stable channel. A drop structure is designed for the downstream extent of the reach to reduce the hydraulic slope to equal the computed stable channel slope by adjusting the bottom width of the structure. Figure 2 shows the decrease in water surface slope that is obtained as the bottom width of the structure is reduced. The final design for the structure is a two foot drop with 3 to 1 side slopes and a bottom width of 50 feet.

Once the necessary structures have been designed, BURBANK is used to check for bank stability. After the bed aggrades due to the construction of the structure, the average factor of safety for rotational failure of the banks in the study reach is projected to be 1.3 along the left bank and 1.1 along the right bank. If it is felt that these factors of safety are too small, the design engineer has two options: the height of the structure can be increased by several feet to cause a reach wide increase in the bed elevation, or toe riprap or bioengineering methods can be used along the reach to increase the stability of the banks. If bank stabilization methods are used that change the roughness of the banks or the width of the channel, the HEC-2 and SAM analysis should be repeated to insure the channel remains stable.

This example has illustrated the effectiveness of using HEC-2, SAM, and BURBANK to design modifications of a channel to stop incision and increase bank stability. The example also illustrates the value of designing grade control structures as hydraulic control, not just a bed stabilizer. Finally, it should be noted that by stabilizing the study reach, the sediment supply to downstream reaches of the channel has been decreased. This may cause downstream channel incision.

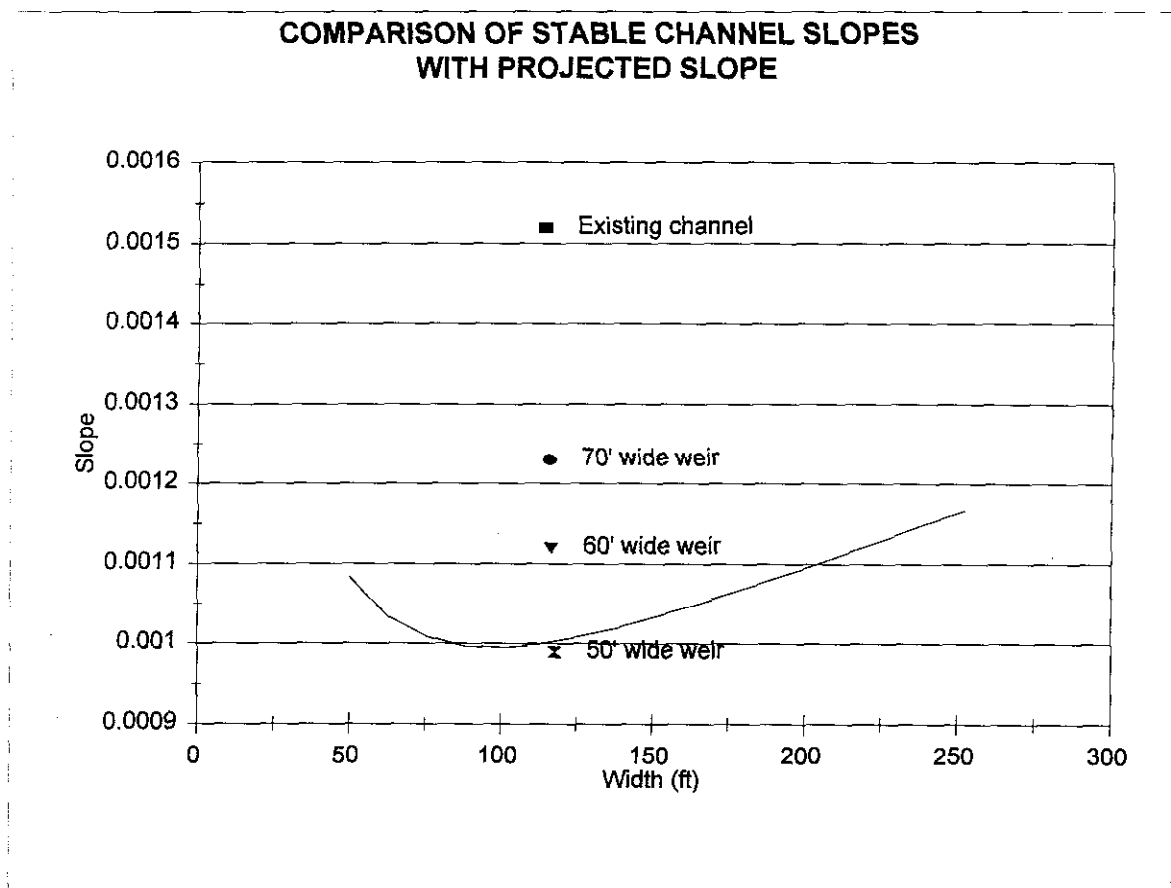


Figure 2: Stable channel dimensions shown with proposed weir widths.

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VALIDATION OF EXTREMAL HYPOTHESIS FOR STABLE CHANNEL DESIGN

Chester C. Watson ¹, Dan Gessler ², Steven R. Abt ³, and Nolan K. Raphael ⁴

Introduction

The Demonstration Erosion Control (DEC) project provides for the control of sediment, erosion, and flooding in the hill area of the Yazoo Basin, Mississippi. Many of the Yazoo Basin streams are deeply incised, unstable channels resulting from channelization that began during the early part of this century. Structural features that are utilized in developing rehabilitation plans for the DEC watersheds include: grade control structures, pipe drop inlet structures, riprap and bioengineering bank stabilization, levees, land treatments, and reservoirs. A series of 25 sites have been monitored beginning in 1991 with field surveys, sediment sampling, structure evaluation, and data analysis for four years.

One of the objectives of the DEC project has been to develop an understanding of incised channel evolution and response of these channels to stabilization measures. Ultimately, the purpose of understanding channel response for engineers is to develop a useable design procedure to estimate stable channel morphology under given conditions of discharge and sediment concentration.

Background

Schumm, et al. (1984) provided a review of basin geology. The upper Yazoo Basin lies within the Gulf Coastal Plain Physiographic Province. A major subdivision of the Province is the Loess Hills, which is comprised of loess-covered hills rising 100 feet or more above the Mississippi River alluvial plain. The loess cap is on the order of 90 feet in thickness and gradually, but irregularly thins to the east. Underlying the loess cap are sands and gravels of possibly Pleistocene age. The spatial distribution of the gravels is irregular, and in places the loess cap directly overlies the Tertiary Claiborne group. The combination of weak and highly erodible bank and bed material, extensive channelization, and precipitation of 100 mm annually has resulted in cycles of headcutting and erosion that have continued since the original channelization during the 1900's. An extensive period of local drainage district channelization was followed, in many watersheds, by spatially intensive watershed channelization during the 1960's.

The primary difference between the extremal theories for predicting stable channel morphology and previous channel design relationships is that earlier work was generally based on empirical data analysis resulting in some type of regime equation. Regime equations would apply only

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to conditions similar to those for which the empirical equations were derived. Various extremal theories are based on the minimization or maximization of certain variables that are used in the formulation of the balance between sediment transport, roughness, sediment characteristics, channel morphology, and discharge.

Davis (1899) originally presented the concept of a graded river, which states "...streams proceed to entrench themselves in the slanting plain, and in a geologically brief period, while they are yet young, they will cut their valleys so close to base level that they cannot for the time being cut them any deeper, that is, the streams will, of their own accord, reduce their valley lines to such a grade that their capacity to do work shall be just equal to the work they have to do." As the gradient decreases, an equilibrium will result between the capacity to do work and the work to be done. At that equilibrium, the gradient has minimized for the imposed conditions of morphology, sediment and water discharge, and roughness. The original concept of the graded stream is, therefore, an early statement of an extremal hypothesis.

Langbein (1964) provided a hypothesis that the changes in hydraulic geometry (Leopold and Maddock, 1953) should be a minimum for a stable channel, i.e., the sum of the squares of the hydraulic geometry exponents is a minimum that is consistent with local restrictions. Yang (1987) suggests that Langbein's primary contribution is the introduction of the concept of minimization to the study of river morphology. Yang and Song (1986) proposed the theory of minimum energy and energy dissipation rate as a hypothesis governing the equilibrium of a closed, dissipative system. Simply stated, this hypothesis is that a system is not in equilibrium if the energy in that system which is compatible with the constraints applied is not at a minimum. For an alluvial river transporting water and sediment, this may be interpreted as the channel morphology and roughness must adjust to transport the imposed water and sediment load at a minimum energy for the river to be in equilibrium.

Three applications of the extremal hypothesis are: minimum stream power (Chang, 1979), minimum unit stream power (Yang, 1976), and maximum sediment discharge (White, et al., 1982). Minimum stream power, expressed in power per unit channel length, is equivalent to the product of discharge and slope (QS). Yang (1987) explains that for steady uniform flow and neglecting energy to transport sediment, the rate of energy dissipation or stream power is:

$$\phi = \gamma QH$$

in which γ = specific weight of water; Q = water discharge; and H = head difference between the upstream and downstream ends of the study reach. The length and slope product, LS , can be substituted for head, H , and for a given γ and a given L , minimization of ϕ results in minimization of QS . Minimization of the QS product defines minimum stream power per unit channel length. Yang (1987) states that for steady uniform open channel flow where the velocity does not vary through a cross-section with constant width, stream power can be further simplified to:

$$\Phi = VS$$

in which V = velocity, and Φ = stream power expressed per unit weight of water.

White et al. (1982) hypothesized that the rate of sediment transport should be maximized as a condition for reaching an equilibrium. They also demonstrated that maximization of sediment discharge and minimization of stream power, or slope for a given water discharge, will give the same result.

Other variations of extremal hypotheses are also found in the literature. The primary value is to provide a rational theory for the solution of channel morphology as a function of the imposed water and sediment load, and for a given roughness.

The Hydraulic Design Package For Flood Control Channels (SAM) is a computer program that was developed by the US Army Corps of Engineers (Thomas, et al., 1994). The purpose of the program is to provide the design engineer with a tool for sediment transport calculations and the design of stable flood control channels.

The conceptual incised channel evolution sequence has been of value in developing an understanding of incised channel dynamics, and in characterizing stable reaches of these channels. The sequence was originally used to describe the erosional evolution of Oaklimiter Creek, a tributary of Tippah River in northern Mississippi. The channel evolution sequence can be used to identify and define a stable channel.

Location-for-time substitution was used to generate a five-reach type, incised channel evolution sequence for stream of the Yazoo Basin (Schumm et al., 1981, 1984). In each reach of an idealized channel, Types I and V occur in series and, at a given location, will occur in the channel through time. The Oaklimiter Sequence describes the systematic response of a channel to base level lowering, and encompasses conditions that range from disequilibrium to a new state of dynamic equilibrium, or stable channel, in Type V.

The primary value of the sequence is to determine the evolutionary state of the channel from a field reconnaissance, a quasi-equilibrium, or stable, channel being Type V. The morphometric characteristics of the channel reach types can also be correlated with hydraulic, geotechnical, and sediment transport parameters (Watson et al., 1988). An understanding that reaches of a stream may differ in appearance, but channel form is associated with other reaches by an evolving process. Form, process, and time relate dissimilar reaches of the stream.

ANALYSIS

For a given discharge, sediment transport rate, bank angle and bank roughness, combinations of channel width and slope can be computed that satisfy both requirements. The width and slope combinations are referred to as stable channel dimensions. SAM can compute stable channel dimensions using Brownlie's bed roughness predictor (Brownlie, 1983) to compute the bed roughness and Brownlie's sediment transport function (Brownlie, 1983) to compute the sediment transport rate. For an existing channel, SAM can be used to compute the stable channel dimensions, given the existing bank angle and the existing bank roughness. The grain size distribution of the bed material must also be specified. A typical stable channel curve along with the existing channel dimension is shown in Figure 1.

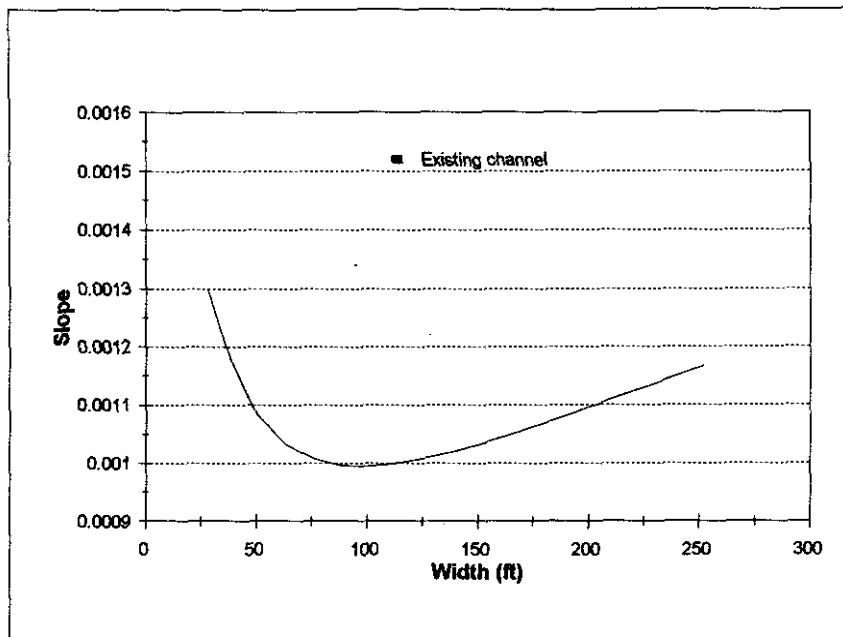


Figure 1. A typical stable channel curve

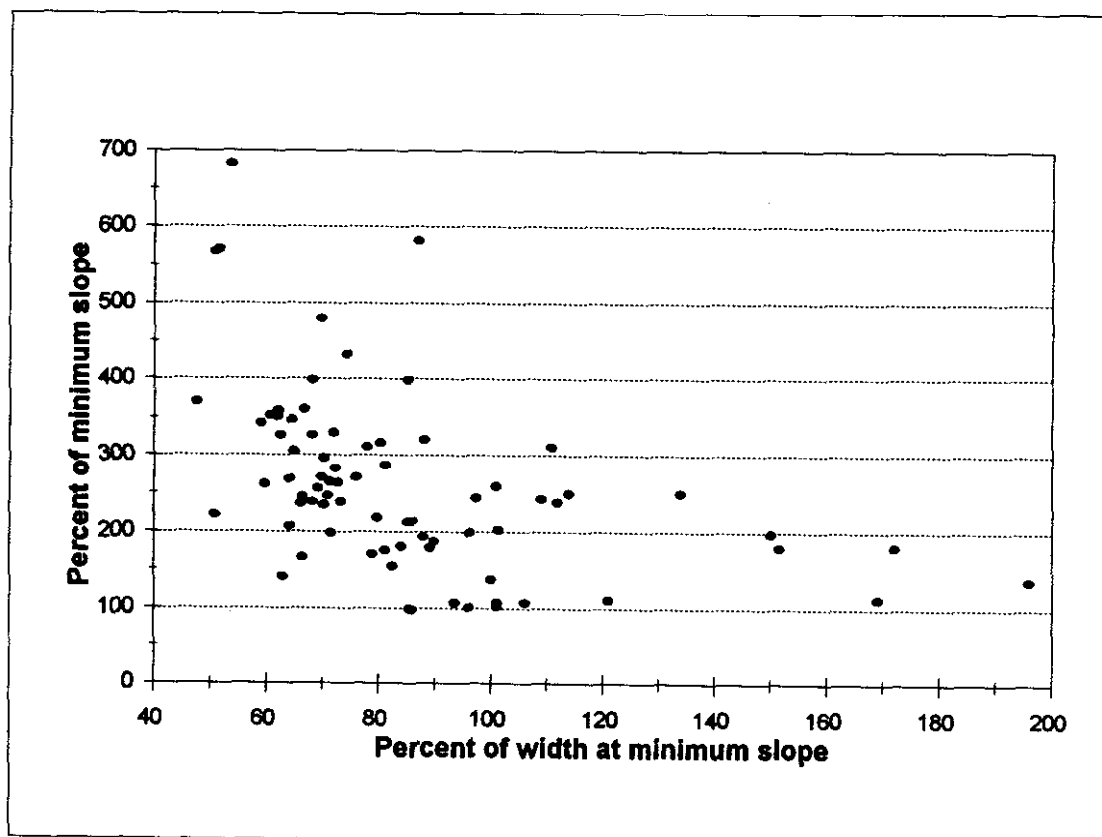


Figure 2. A dimensionless DEC data set

To compare the relative stability of many channel reaches, the existing channel width is divided by the minimum stable channel width and the existing channel slope is divided by the minimum stable channel slope. These two values are referred to as the channel stability indicators.

Using the SAM computer program, the stable channel curve was computed for each segment of the 25 DEC streams. A sediment concentration of 1000 mg/l and the approximate 2-year discharge was used for all segments, based on the assumptions that the presently stable (Type V) channel segments have an effective discharge of the 2-year discharge, and that the present sediment supply to those reaches is 1000 mg/l. The dimensionless stable channel curve is shown in Figure 2.

For each segment, the actual sediment transport capacity was also computed, and the segments were then grouped according to the actual sediment transport capacity of the segment. The median values of the sediment transport capacity in mg/l for each group are shown in Figure 3. Figure 3 indicates that segments with a median sediment transport capacity of 1000 mg/l have channel dimensions that match the minimum slope of the dimensionless stable channel curve. In addition, the channel segments were field-evaluated to determine the channel evolution model types. Figure 4 is the same data as Figure 3, and in Figure 4 the data has been grouped by the channel evolution model type. The value shown is the median value for each group.

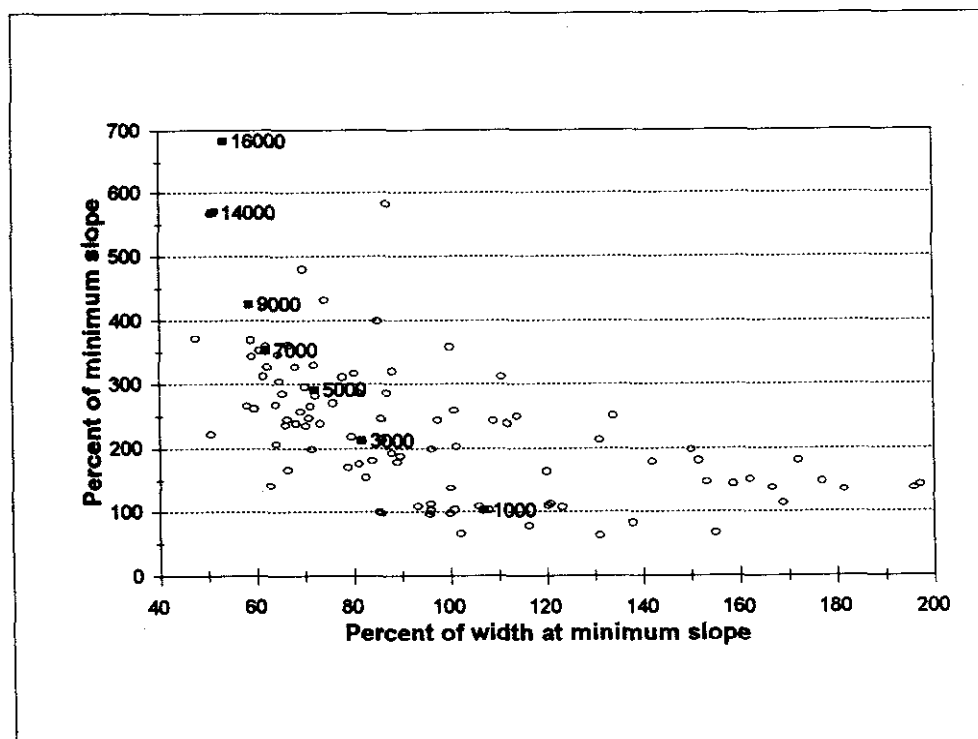


Figure 3. The dimensionless DEC data set grouped by transport capacity

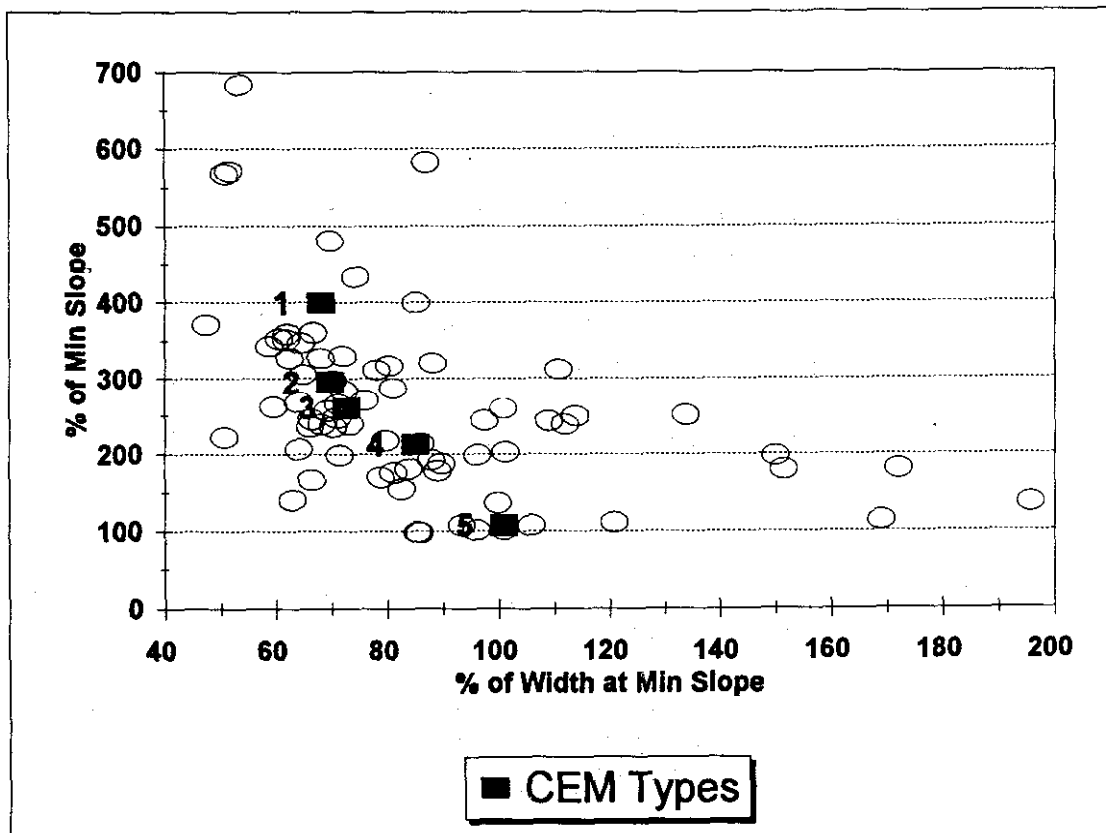


Figure 4. Dimensionless stable channel data grouped by CEM type

Conclusion

A comparison of computed channel morphology to field-measured channel dimensions was made. It was shown that for the 25 DEC stability sites analyzed, the stable channel dimensions of width and slope, and the observed stable channel morphology converge at the point of minimum slope. These data provide strong evidence to support the extremal hypothesis concepts.

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COMPARISON OF NPS MODELING PROTOCOLS, OR, WHAT'S ONE PERSON'S VERIFICATION IS ANOTHER'S VALIDATION

Russell Shepherd and Frank Geter*

ABSTRACT

The recent increased use of agricultural nonpoint-source water quality computer models necessitates the use of a consistent, comprehensive protocol for evaluating their effectiveness and utility in meeting specific modeling goals. We compared a standard protocol for evaluating ground-water models (Anderson, et. al., 1993) with protocols used for a watershed-scale and two field-scale NPS models, including code and model verification, model calibration and validation, and predictive simulation. Results indicate that model calibration and validation are two important steps that will require additional emphasis in future NPS modeling. The study resulted in a suggestion for a more detailed, comprehensive modeling protocol scheme.

INTRODUCTION

Computer models of ground-water flow and transport have been in existence for many years (see Prickett, 1975, Prickett and Lonquist, 1971, and Wang and Anderson, 1982 for historical summaries). Along with their development, a protocol of model development and evaluation has been established. This protocol has been well accepted, and generally employed by many hydrogeologists (Anderson et. al., 1993). In contrast, and although computer modeling of surface-water nonpoint-source (NPS) pollution has been done for many years also, a similar modeling protocol is less well documented and employed, and especially in the area of agricultural nonpoint source models. Thus, the agricultural nonpoint-source literature contains numerous discussions and presentations of model and/or code validations, verifications, calibrations, and evaluations, but these are not used consistently or uniformly, either in practice or in the literature. What is one modeler's verification may be another modeler's validation, which does not tend to promote optimum communication. The Natural Resources Conservation Service has recently released and supports technology for NRCS versions of the Agricultural Research Service (ARS) NPS models AGNPS, GLEAMS, EPIC, SWRRB-WQ, and NLEAP. Communication of modeling protocol is required among model users, developers, and SCS technicians. The purpose of this brief paper is to discuss modeling protocol issues, emphasizing those used in ground-water modeling in relation to analogous protocols used in agricultural nonpoint-source modeling. We will provide examples from protocols of ARS developers of three excellent and well-reviewed models (AGNPS, GLEAMS, and NLEAP) that are well known, accepted, and becoming widely used in NPS simulation.

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MODELING PROTOCOLS

In a recently published Handbook of Hydrology (Maidment, 1993), Anderson and others (Chapter 22) defined a modeling protocol as "an accepted procedure for applying models". They identified eight basic steps in this procedure, normally used in ground-water modeling, and presented here with some interpretation as applied to NPS modeling efforts, as follows:

1) Define Purpose. This is more important than may be recognized at first. For example, a model may be used only to characterize the components and/or operation of a system. The purpose of many agricultural NPS pollution models (e.g. AGNPS) is to compare management scenarios rather than to simulate chemistry and predict specific levels of potential contaminants. Another purpose of modeling may be to simulate the processes and components of the system under expected future conditions to predict quantitative levels of pollutants. Defining a purpose well may change the significance of later steps in the protocol.

2) Define the Physical/Chemical System. This step is commonly the overriding limiting step because here the lack of basic data will become apparent. For example, our pollutant loading models GLEAMS and NLEAP are designed to model ground-water loading of pesticides or/and nitrates, yet data characterizing the vadose zone and the ground-water system are commonly inadequate to allow the models to achieve our goals. In agricultural modeling we commonly have adequate data only through soil survey depth (5 feet is standard). Many of our models are two dimensional but are necessarily applied to a 3-D system. Our watershed-scale model, AGNPS, is cell-based, with standard recommended cell sizes of 4 and 16 hectares (10 and 40 acres). Assignment of a slope, a channel geometry, a soil erodibility, and other singular factors for a 16-hectare area of land constitutes rigorous system definition that will have a controlling effect on model results regardless of the quantitative methods used.

3) Code Selection and Verification. Here the model that is thought will best simulate the system for the intended purpose is chosen, and the code checked against analytical solutions to be sure it will work. For example, if we have a vadose zone problem, we may select one or more of the 89 vadose zone models listed by van der Heidje (1992) as appropriate. However, because of the large number of variables, processes, and system elements in agricultural NPS modeling, it is often found that only one or two applicable models are available, and the model selection and code verification process becomes reversed. That is, we select which model we are going to use before we know much if anything about our physical/chemical system.

4) Model Calibration. This step is basically adjusting or "tweaking" one or more variables of the model to obtain a match to a known set of field measurements that constitutes a calibration target. Recent methods use parameter estimation statistics to evaluate error and optimize model inputs (e.g. Hill, 1993). Interestingly, some agricultural NPS models are not calibratable, for example GLEAMS (Knisel, 1993, personal communication) and AGNPS (Young, 1994, personal communication) because they contain so many variables that interact so complexly. Such models require a high level of empirical knowledge and skill to apply effectively. In contrast, input parameters for the nitrate-loading model NLEAP are adjusted to produce output matching known levels of nitrate in soil profiles before simulations are made (Shaffer, 1994, pers. comm.).

5) Model Verification. In this step the calibrated model is used in a simulation in which results are compared to an independent set of measured field data. For example, in ground-water modeling steady state heads may be used for calibration, and the model is verified by comparison to changing heads produced during a pumping test (transient data). Models may also be calibrated with transient data (Shepherd, 1987). In agricultural NPS modeling, this step is frequently referred to as model validation (Shaffer, 1994, personal communication).

6) Predictive Simulation. This is the final goal of many models, when the calibrated, verified model is run to predict the effects of stresses on the system in the future. For example, what will be the result of long-term ground-water withdrawals? What will be the predictions of long-term pesticide usage or nitrogen applications? Models may be parameterized to predict individual events of possibly very short time periods, or of very limited physical systems, and should not be used outside their boundary conditions. This step (and the next one) can present unique problems for models used to compare scenarios rather than to predict pollutant loads at a watershed outlet, for example.

7) Model validation. According to Anderson et. al. (1993), a model is validated if a predictive simulation is found to be accurate after a "postaudit" is performed on the system after a significant time period has passed since the simulation and the calibrated conditions have changed. In agricultural NPS modeling, this definition of model validation is generally not used (Shaffer, 1994, personal communication).

DISCUSSION

It is apparent that some differences exist between the ground-water and the agricultural nonpoint-source modeling protocols, and indeed that the NPS protocols are not consistently or uniformly applied. This is not to be unexpected, mainly because of the temporal and spatial constraints initially placed on the agricultural system to be modeled (Rojas, 1994, personal communication). The modeling of runoff and/or sediment yield on agricultural watersheds, which are generally the first processes to be considered in NPS modeling, and the modeling of ground-water flow and heads, are temporally disparate. That is, ground-water flow rates and changes in ground-water head distribution, recharge, and discharge rates commonly (but not always) are slow compared to runoff events. Therefore, the model validation done for long-term changes in a ground-water system is not possible with NPS models that simulate a storm event, (for example, AGNPS). Nevertheless, the calibration of a runoff model is possible and recommended, as is its verification (validation in NPS circles). A possible temporal hierarchy of calibration for NPS modeling is 1) runoff/infiltration, 2) erosion, 3) sediment transport, 4) soil moisture, 5) plant growth and water and nutrient uptake, and 6) pollutant sorption/desorption and transport.

CONCLUSIONS

The comparison of protocols used in ground-water modeling (Anderson et. al., 1993) with those used in agricultural NPS modeling indicates that significant differences exist having important practical ramifications. However, the most apparent issues are semantically based and derive from temporal and spatial differences in the physical systems modeled.

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VALIDATION OF THE GRASS-TOPAZ-SWAT SEDIMENT YIELD SCHEME USING MEASUREMENTS FROM THE GOODWIN CREEK WATERSHED

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Abstract: A validation of the integration of GIS, topographic analysis tools and watershed models is presented. This scheme involves the use of GRASS GIS layers, the digital elevation analysis model, TOPAZ, and the watershed model, SWAT. The GRASS-TOPAZ-SWAT scheme is validated using data collected by the USDA-ARS, National Sedimentation Laboratory at the Goodwin Creek Watershed (GCW) in northern Mississippi. Using a combination of GRASS GIS layers digitized for GCW and measured input parameters, the SWAT model's results were compared to the measured runoff and sediment yield collected over ten years from fourteen instream measuring stations. Results show that the SWAT model runoff and sediment yield predictions follow the relative trends described by the measured data from various locations within the watershed. The GRASS-TOPAZ-SWAT scheme provides a relatively accurate, integrated and simplified approach to develop information needed for evaluations of erosion control measures on watersheds with minimal model user involvement.

INTRODUCTION

Erosion models are often needed to evaluate erosion control practices on watershed systems. The widespread use of these models on large watersheds has been restricted in the past because of computer limitations and the tedious task of developing manually a large model database. Recently, techniques have been developed which utilize geographical information systems (GIS) to obtain many of the model input parameters (Srinivasan and Arnold, 1994). By combining GIS layers with relationships to determine model input parameters, complex systems can be easily simulated. Obtaining GIS layers with sufficient accuracy to reliably simulate the watershed processes still represents a major problem.

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The SWAT (Soil and Water Assessment Tool) model (Arnold et al., 1993) was developed to simulate large watersheds by subdividing the area into many subbasins. SWAT can utilize information from GRASS (Geographic Resources Analysis Support System) (Shapiro et al., 1992) GIS layers, which describe the elevation, soils, and landuse of a watershed. The responsibility of ensuring the accuracy of the GIS layers is mostly left to the model users. One tool that can be used to automatically extract topographic parameters from digital elevation models (DEM) (e.g., slope, slope length, subbasin boundaries, and subbasin routing sequence) is the model, TOPAZ (formerly DEDNM) (Garbrecht and Martz, 1995). TOPAZ provides relatively accurate channel representations within a watershed and the resulting subbasin boundaries. DEM's typically require some rectifications of depressions, flat areas, and other surface features, which can be easily resolved with TOPAZ.

The integration of these tools is important in evaluating large watershed systems. The Goodwin Creek Watershed (GCW), located in northern Mississippi, was used to test the capabilities of this scheme to provide input information to SWAT from the subbasin boundaries and routing sequence generated by TOPAZ, the use of a GRASS input interface for the other parameters, and measured parameters collected from GCW. Rosenthal et al (1995) used a similar approach to evaluate a GRASS/SWAT scheme on the hydrology of a Texas watershed. They found that the scheme worked well for their conditions. The addition of TOPAZ in this scheme allows a fine level of control on the use of DEM's in developing subbasin boundaries and the routing of runoff and sediment among them. The objective of this paper is to compare measurements of runoff and fine sediment yield from the Goodwin Creek Watershed to values computed with the GRASS-TOPAZ-SWAT scheme over a ten year period.

WATERSHED DESCRIPTION AND INPUT PARAMETERS

A 21.3 km² experimental watershed in northern Mississippi, Goodwin Creek Watershed, was used to test the GRASS-TOPAZ-SWAT modeling scheme (Figure 1). GCW is divided into fourteen subbasins with a fully-instrumented flume located at each subbasin drainage outlet to continuously monitor runoff and sediment yield. The drainage areas above the outlets of each subbasin range from 0.05 to 21.3 km². Watershed terrain elevation range from 71 to 128 m above mean sea level, producing an average channel slope of 4m/km. Within GCW, 32 recording raingages provide areal distributions of rainfall storm events. The average annual rainfall during 1982-1991 was 1460 mm. This rainfall produced an average annual runoff of 537 mm as measured at the outlet of GCW.

The GRASS elevation layer was produced from USGS 7.5 minute DEM's at a scale of 30 m by 30 m with one meter elevation increments. A soils layer was produced by digitizing information from the Natural Resources Conservation Service (NRCS, formerly SCS) county soils maps for the GCW area. Initial GIS landuse information was obtained from a

1987 Landsat thermal mapper satellite image, which was able to divide landuse into forest, pasture, and crop land. Bingner (1995) described a similar simulation process using SWAT to evaluate the runoff predicted at each of GCW's fourteen instream measuring stations using fourteen subbasins to define the watershed.

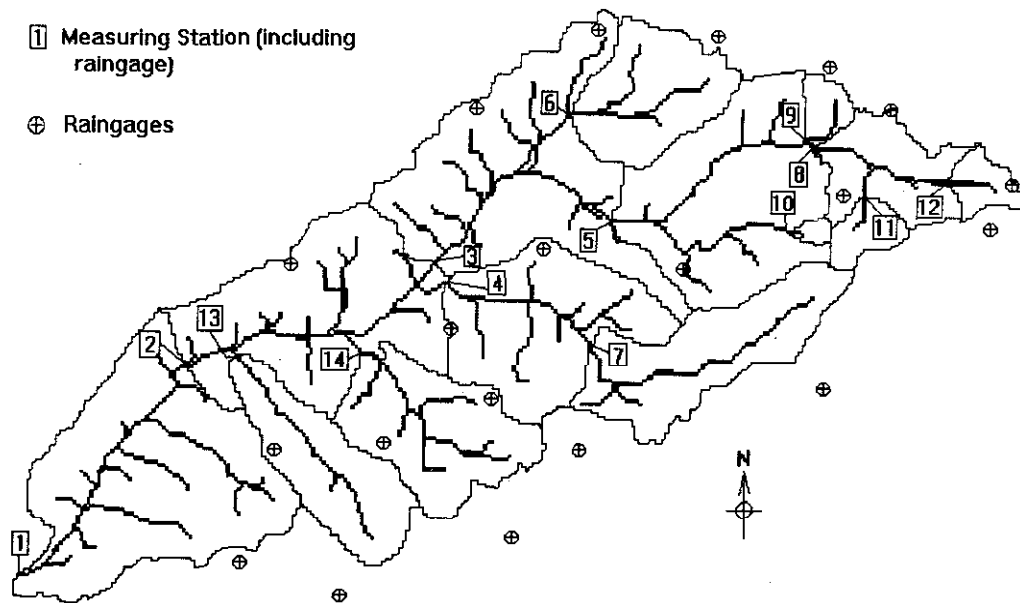


Figure 1 - Goodwin Creek Watershed drainage subbasins defined from the measuring stations along with TOPAZ generated channels and raingage locations.

TOPAZ was used with the GCW DEM by specifying parameters for critical source area and minimum source channel length as seven hectares and 130 meters, respectively, to produce the subbasin boundaries. The output from TOPAZ was used to combine the source area, the left and right subbasins along a channel, to produce 138 subbasins. The channel network produced from this simulation of 138 subbasins is illustrated in Figure 1. Runoff and sediment yield from individual subbasins were routed to points in the watershed that would closely correspond to the location of the GCW instream measuring stations. These points could then be used to compare measured runoff and fine (< 0.062 mm) sediment yield for each rainfall storm event.

A GRASS/SWAT interface (Srinivasan and Arnold, 1994) was used with the GIS layers to determine predominate subbasin properties. For example, the soil type from the GIS soils layer that had the greatest area within a subbasin was used for that entire subbasin. The GIS landuse layer was used to provide initial cropping information for each subbasin by the interface to SWAT. SWAT has the capability to vary the simulated landuse practices each

year within each subbasin. Annual landuse surveys performed on GCW of all the individual fields were used, after the interface provided the general landuse information, to determine the variation of landuse practices within each subbasin. Landuse throughout the simulation period varied from 23% cultivated, 11% idle, 40% pasture, and 26% forested in 1982 to 13% cultivated, 10% idle, 50% pasture, and 27% forested in 1991. Once the information for the subbasins was developed, observed daily rainfall and temperatures for 1982-1991 were used for the simulation. SWAT input parameters were used as determined by the GRASS/SWAT interface, except where the interface could not determine appropriate values, such as Manning's n . Measured input parameters were used when available and calibration was not performed on the other input parameters.

RESULTS AND DISCUSSION

SWAT (version 93.1) simulated each of the 138 subbasins and routed the flow of water and sediment through the channel system for each storm event during 1982-1991. There were 913 storm events simulated during the ten years of simulation. A storm event was defined as a period of rainfall greater than or equal to 2.5 mm separated from other rain periods by more than six hours. Each measuring station was used to validate the capability of SWAT to simulate the runoff and sediment yield from subbasins of various sizes and locations. The average measured and simulated daily runoff during these years, based on drainage area, is shown in Figure 2.

The simulated total runoff of each year between 1982-1991 is within 50% of measured values for all subbasins and typically within 80%. Regressions of measured versus simulated runoff for the storms in each year resulted in r^2 values typically above 0.80. Simulations of runoff produced at measuring stations 8, 9, and 12 typically produced average annual runoff within 85% of measured and individual storm event r^2 values within 0.95. Generally, the subbasins in the upland areas were simulated better than the areas further downstream.

The yearly measured and simulated runoff determined at the main outlet of GCW, located at measuring station 1 is illustrated in Figure 3. SWAT simulations were very good during 1984-1988 and less accurate for the other years. During 1982, 1983, and 1989-1991, when rainfall was above normal, SWAT consistently underpredicted the runoff at all locations of GCW. Although, during those years the simulated trend of average annual flow was similar compared to the measured trend. Rosenthal et al (1995) found that SWAT underestimated runoff for extreme events for a watershed in Texas. The results from the simulations of GCW suggests that during years of many extreme events, such as for the above normal rainfall years compared to the below normal rainfall years, SWAT also underestimated runoff volume.

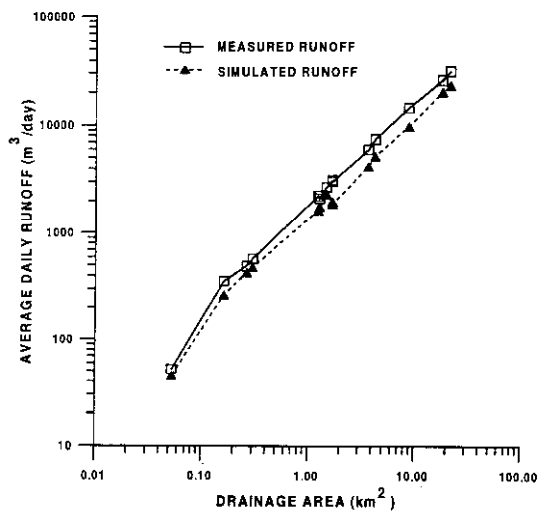


Figure 2 - Average daily runoff as a function of drainage area during 1982-1991.

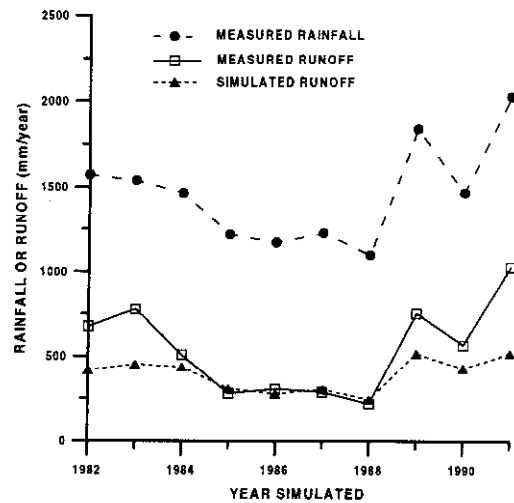


Figure 3 - Yearly measured rainfall and runoff compared to simulated runoff as determined from measuring station 1.

Ten-year averages of measured and simulated daily sediment yields plotted against subbasin drainage area are presented in Figure 4. Simulated sediment yields were 2 to 55% of the measured sediment yield for all subbasins and typically within 35%. From Figure 5, simulated results at measuring station 1 of GCW show that 36% of the measured fines are produced in the overland areas and thus 74% come from other sources in GCW, such as channels and gullies. These results agree with those of Grissinger et al (1991) who estimated, from measurements of the channel cross-sections of GCW from 1982-1987, 78% of the fine sediment yield determined at measuring station 1 originated from channel or gully sources. Total load measurements from measuring stations 1 and 2 are shown in Figure 4. Simulated fines represent 20% of the measured total load, which generally agrees with the 15% estimated by Grissinger et al (1991).

Measured and simulated fines represent only sediments finer than 0.062 mm. Other sizes of transported sediments were also measured at a few measuring stations, but with less sampling frequency than the fine sediment sampling method. Since the most available measured data was less than 0.062 mm for all stations, only that portion of SWAT simulated sediment yield was used for comparison with measured values. SWAT can simulate the sediment yield of fine sand-sized particles from individual subbasins, but this portion was not included in this analysis.

Measured and simulated sediment yield are shown on an average annual basis for measuring station 1 in Figure 5. Reduced runoff during 1985-1988 resulted in significantly reduced average annual sediment yield compared to the other years. Simulated sediment

yield demonstrated similar yearly trends as the measured sediment yield. Generally, r^2 values of measured versus predicted at all stations were higher during the years of above normal rainfall, typically near 0.75. During the below normal years, r^2 values were reduced to near 0.50. This suggests that overland sources of fine sediment yield does not describe very well the variation in total fine sediment yield of individual storm events at the measuring stations. Other erosion processes, such as gully erosion, bed and bank erosion, and bank failure, may be occurring that SWAT does not account for.

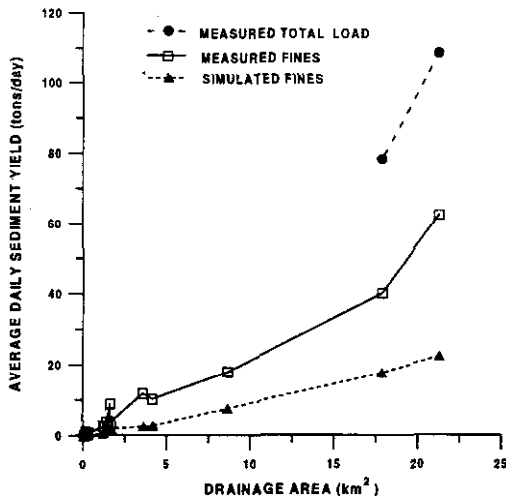


Figure 4 - Average daily sediment yield from overland sources and measured total load, as a function of drainage area during 1982-1991.

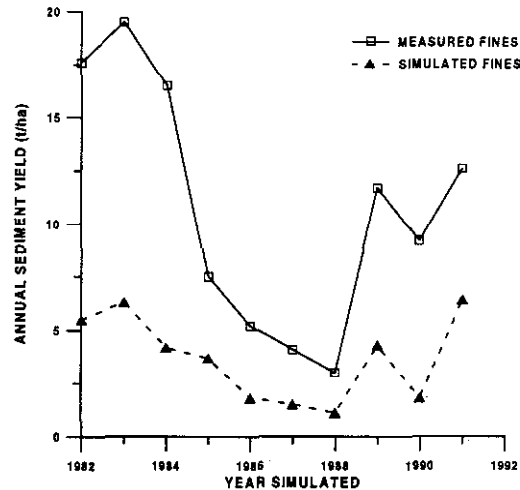


Figure 5 - Yearly measured and simulated sediment yield as determined from measuring station 1.

Land use changes have been shown by Kuhnle et al. (1995) to affect the fine sediment concentration on GCW. Sediment concentration at the outlet of GCW decreased during 1982-1991. Kuhnle et al. (1995) showed that 26% of this sediment concentration decrease was attributable to a decrease of cultivated land use, from 23% to 11% of GCW. The rest of the sediment concentration decrease was attributable to changes in the production of channel erosion within the watershed during this period, such as construction of bank stabilization measures that reduced channel bank erosion. The reduction of cultivated land use may partially explain why the fine sediment yield was less during 1989-1991, a time of less cultivated land use, than during 1982-1984, even though rainfall and runoff was equal to or greater than 1982-1984.

CONCLUSIONS

The integration of GRASS-TOPAZ-SWAT technology was validated using runoff and sediment measurements over ten years from the highly instrumented USDA-ARS, Goodwin Creek Watershed. Simulated runoff values exhibited reasonable trends associated with drainage area compared to measured values. The simulation of sediment yield demonstrated that 36% of the total fine sediment yield and 20% of the total load can be attributed to overland fine sediment producing sources. This agrees with earlier studies attributing 75% of the fine sediment from channel and concentrated flow areas in GCW.

This scheme has been demonstrated to be applicable to a wide range of subbasin size areas, land use management practices, and spatial location of the subbasin within a larger watershed. This scheme can be used to efficiently develop effective long term conservation management plans for large watershed systems. This scheme also reduces the effort required in the tedious task of developing a large and accurate database of input parameters needed by the simulation model.

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VARIATIONS IN BED MATERIAL SIZE ON GOODWIN CREEK

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Abstract: The Goodwin Creek Research Watershed drains a 21.3 km² area in the bluffline hills just east of the Mississippi River flood plain in the north central part of the state of Mississippi. In the spring of 1994 bed material was sampled in 14 reaches of the watershed. The mean length of the reaches was about 1000 m. Median grain diameter of the bed material ranged from 0.5 - 7.4 mm over the watershed. The bed material in the upper reaches of the watershed is predominantly composed of sand ($D_{50} = 0.5$ mm). In the central parts of the watershed there are many sources of gravel and the size of the bed material coarsens. In the last 3 km of the main channel there are no major tributaries and the median size of the bed material decreases from 7 mm to 1 mm. This decrease in size is attributed to selective deposition of the coarse material in the bars on the inside of several bendways. These size variations are likely related to the mechanisms by which gravel is transported through the watershed.

INTRODUCTION

Knowledge of the size distribution of the bed material in the channels of a watershed is critical information for understanding the movement of sediment and water through the watershed and for making determinations of the stability of the channel. This knowledge is an important part of the comprehensive set of flow and sediment transport data, which has been collected on the Goodwin Creek Experimental Watershed (GCEW). Understanding the movement and storage of the gravel is critical for understanding the long-term stability of the bed and banks of the channels. The knowledge gained from this study will also be useful for understanding the movement and storage of gravel in other watersheds with incising channels. The bed material of Goodwin Creek is composed of a mixture of sand and gravel and shows large variations in median size over the watershed. The median size of the bed material is in the sand sizes in the upstream part of the watershed, coarsens in the central part of the watershed, and fines in the downstream 3 km of the watershed.

The main channels of the watershed were divided into a series of reaches with a mean length of 1000 m, for the purposes of sampling the bed material sediment. The size distribution of the bed material varies spatially and probably temporally as well. The data contained in this report came from samples collected in the spring of 1994 and thus represent the mean state of the bed material at that time. This report contains a summary of the data on the spatial variability of the bed material size distribution of the GCEW and offers some initial determinations as to the causes of this variability.

STUDY AREA

The 21.3 km² Goodwin Creek Experimental Watershed is located in the bluffline hills of the Yazoo

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River basin in the north central part of the state of Mississippi (Fig. 1). The elevation on the watershed ranges from 71 to 128 m above mean sea level, with an average channel slope of 0.004. The region is characterized by steep slopes and a wide variety of erosion and sedimentation processes. Reach slopes of the bed range from about 0.001 to 0.01, and bed load transport rates of 1 kg/s m have been measured on the channels of the watershed. The average annual rainfall of the watershed over the period of record was 1460 mm, with the greatest amounts of rainfall occurring during the spring months of the year. Most of the channels of the watershed are ephemeral, with perennial flows occurring only in the lower reaches of the main channel. The bed material sediment ranges in size from 0.1 to 64 mm. Sources of sand and gravel to the channels originate from gullying in some of the upland parts of the watershed and bank erosion in the channels. The watershed includes 14 gauging stations in the form of concrete "V" -shaped supercritical-flow flumes. These structures serve dual functions as grade control and gauging structures, but were designed primarily to facilitate measurement of flow and sediment transport rates (Bowie and Sansom., 1986; Willis et al., 1986). Collection of flow and sediment transport data began in 1982 and continues to the present.

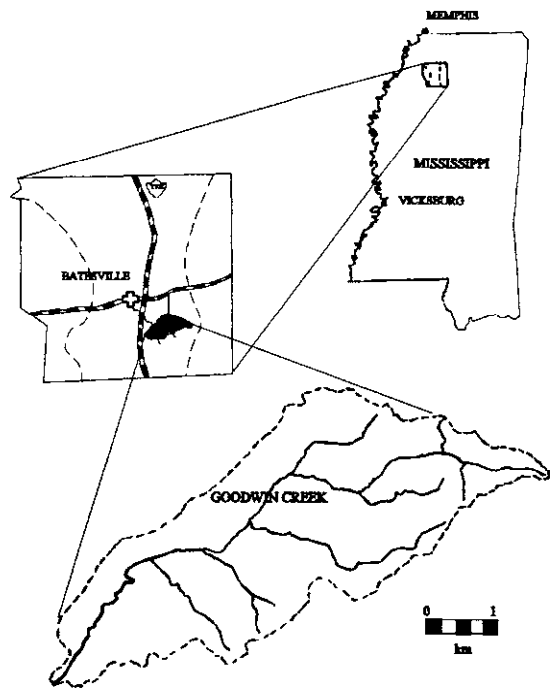


Figure 1. Location Map of Goodwin Creek Experimental Watershed.

SAMPLING TECHNIQUE

The procedure for sampling was designed to measure the mean size distribution of the bed material sediment that is available for transport on a yearly basis. First, the channels of the watershed were divided into 19 reaches. Each reach was defined as the length of channel between major channel junctions. The mean length of the reaches was 1059 m. Because of time and accessibility constraints, 8 reaches were sampled over their entire length and 6 reaches were partially sampled. With 3 exceptions because of local conditions, reaches were sampled on a grid with a flow parallel spacing of 30 m. At each location either 2 or 4 subsamples were collected depending on the width of the channel. Each subsample was collected about 75 mm below the surface and had a mass of approximately 1 kg.

The total sampled mass for each reach was at least 70 kg, which corresponded to medium accuracy according to the gravel sampling guide of Church et al. (1987). All of the gravel collected from each reach was sieved at 0.25 ϕ intervals ($\phi = -\log_2 d$, where d is the grain diameter in mm). The subsamples of each reach were combined into groups of approximately 20 kg for efficient size

analysis and to yield information on the variability of the size distribution in each reach. The sand for each reach was combined and a representative split was sieved. The size distribution of the sand fraction of each 20 kg group was assumed to be the same as that for the entire reach.

RESULTS

The median size of the bed material of the 14 sampled reaches ranged from 0.5 to 7.4 mm (Fig. 2).

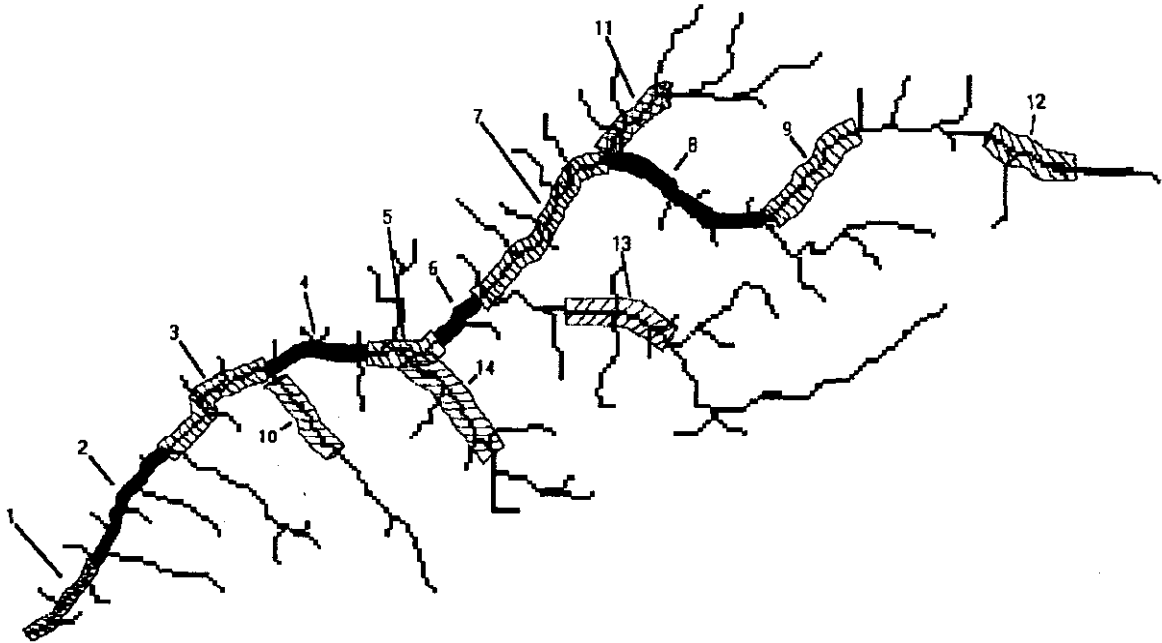


Figure 2. The 14 reaches for which bed material samples were collected on the GCEW. Bed material median sizes for the reaches are: 1- 1.0 mm, 2 - 1.4 mm, 3 - 4.4 mm, 4 - 6.9 mm, 5 - 3.5 mm, 6 - 6.7 mm, 7 - 3.9 mm, 8 - 1.1 mm, 9 - 1.4 mm, 10 - 7.3 mm, 11 - 1.4 mm, 12 - 0.5 mm, 13 - 2.4 mm, 14 - 7.4 mm. The hatching and shading is used to highlight the different reaches.

Median sizes are generally fine in the upstream (northeast) portions of the watershed, coarsen towards the central part of the watershed, and become finer again towards the outlet of the watershed. Following the main channel of the watershed from reach 12 to reach 1 illustrates this pattern (Fig. 3). The changes in median size of the bed material are a function of the location and supply of gravel to the channels and selective transport of the gravel sizes.

Over most of the channels of the watershed it is difficult to separate the effect of gravel supply from that of selective transport on the resulting size of the bed material. The main channel from the confluence of stream 13 (reach 10 on Fig. 2) to the outlet of the watershed is different, however. The last major source of gravel to the main channel of the watershed occurs at the confluence of stream 13. Downstream of the stream 13 (reach 10) confluence the median size of the bed material begins to decrease rapidly. In a distance of about 3.5 km the median size of the bed material decreases from 6.9 to 1.0 mm. The changes in median grain size over this 3.5 km distance are well represented by

an exponential relation (Fig. 4). Changes in other percentile measures over this reach are also shown in Figure 4.

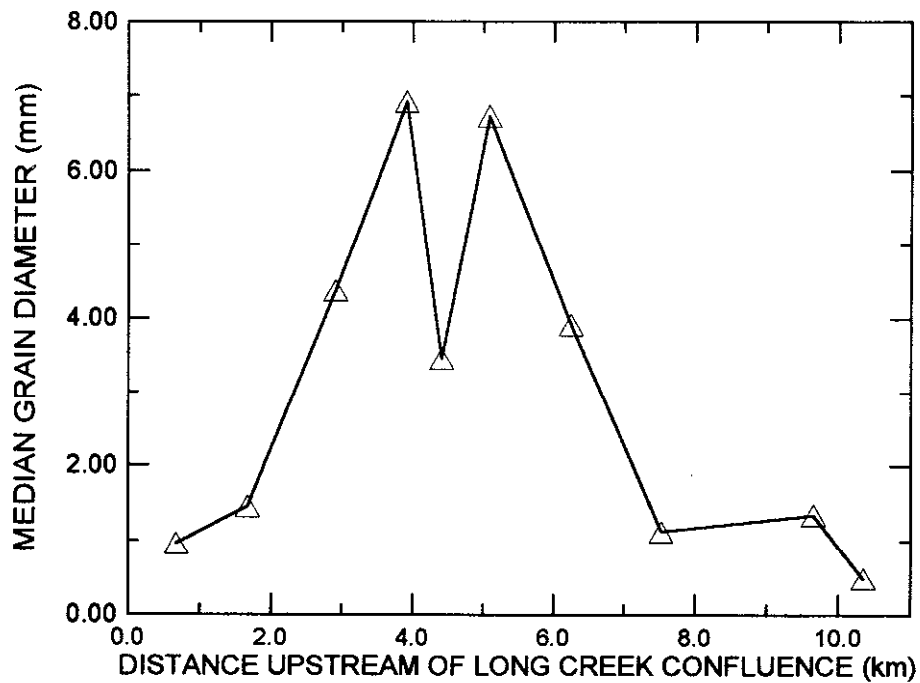


Figure 3. Median grain size by reach for the main channel of Goodwin Creek. Goodwin Creek flows into Long Creek and their confluence is used as the zero point of the channels.

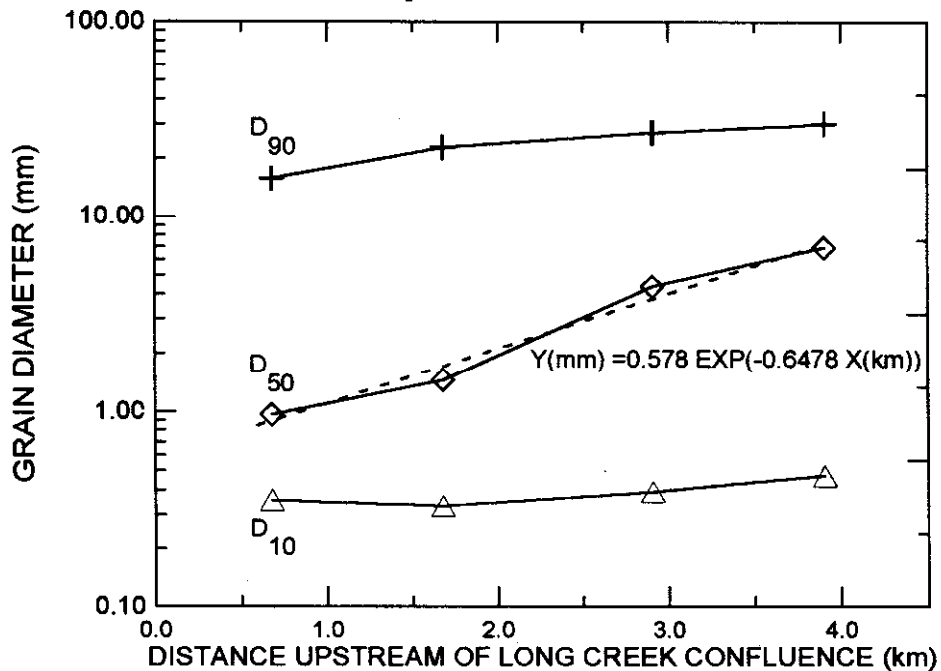


Figure 4. Changes in grain size of the bed material for the lowermost 3.5 km of Goodwin Creek watershed. D_x is the grain diameter for which $x\%$ is finer than.

Grissinger and Murphey (1989) studied bank and bed material erosion and deposition on a 4.7 km long reach that included the 3.5 km reach discussed above for a period of about 11 years. Data collected from repeated cross section surveys of this reach allowed them to determine that erosion of in situ bank material averaged about 10,500 m³ per year, and bar aggradation averaged about 3,300 m³ per year from this channel length. This erosion and deposition occurred while the mean elevation of the thalweg of this reach was stable.

Sediment transport data has been collected at several of the main gauging stations of the watershed over a number of years. The sediment transport data collected at stations 1 and 2 along with the data collected by Grissinger and Murphey (1989) from reaches 1, 2 and 3 (Fig. 2) allowed an approximate budget of the supply and deposition of sand and gravel to be made for the channel between stations 1 and 2 (reaches 1 and 2, Fig. 2). Calculations of sand load based on collected samples indicate that sand transport into and out of the 3.5 km reach may be considered to a first approximation to be equal for the period 1983 -1987. Calculations of gravel load from transport samples show that the mean volume of gravel being transported through station 2 of the watershed was from 240 -720 m³ yr⁻¹ over the same period. The size distribution of the bed material in reach 1 is much finer than in reach 2 (Fig. 2). Because of this change in the bed material size of the two reaches, it was assumed that all of the gravel is being deposited between stations 1 and 2. Also it follows that any sand or gravel that is added to the channel from bank erosion must be deposited in the bars of this reach. Two samples of the bank material from the lower end of the watershed have an average of 25% sand, and 0% gravel. If the banks are assumed to be 25% sand and all of it is assumed to be deposited in the bars of the reach then it follows from using the volumes of bank erosion from Grissinger and Murphey (1989), that 65% of the bar deposits are sand and the remaining 35% is gravel. Working from the calculated volume of gravel that has been transported through station 2 from 1983 to 1987, and assuming that all of the gravel is deposited in the bars of the reach, yields that from 11 to 32% of the bars should be composed of gravel and from 68 to 89% sand. It also follows that the eroded banks must be composed of from 26 to 34% sand. The results of these two calculations indicates that the percentage of sand in the bars likely ranges from 65 to 89%. A future study will sample the sediment in the deposited bars between stations 1 and 2 to close the loop on the sediment budget of this reach. Determining the sources and volumes of the sediment deposited in the bars of the reach between stations 1 and 2 will yield information on the rates of channel change in this and on similar reaches of channel.

The variability of the size distribution of the bed material within the reaches was assessed by analyzing groups of subsamples (each about 20 kg) for each reach. The reaches of the main channel of the watershed (1,2,3,4,5,6,7,8,9,12, Fig. 2) show a somewhat cyclic variation in the median grain size (Fig. 5). The median size of the bed material varies from sand to gravel dominated areas at a mean distance of about 1 km on the main channel of the watershed. These coarse to fine variations are likely related to the transport of gravel through the watershed. Large changes in grain size with time have been measured in the bed load over 30 minute periods during a period of nearly steady flow on the GCEW (Kuhnle et al., 1989). It is not likely, however, that the coarse regions shown in Figure 5 migrate at a rate sufficient to cause the observed changes in bed load grain size distribution.

Smaller scale features likely were the cause for the 30 minute fluctuations in bed load size. Possibly these 1 km coarse/fine alternations may cause a longer term fluctuation in the rate and size of the bed load.

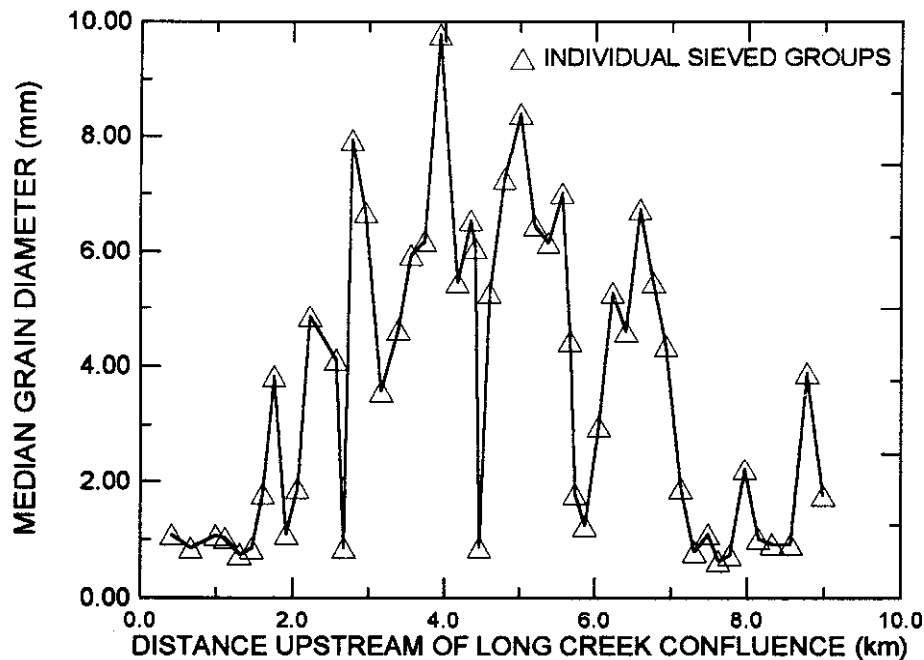


Figure 5. Variations in median grain diameter for bed material of the main channel, GCEW.

SUMMARY AND CONCLUSIONS

A sampling program of the bed material of Goodwin Creek Experimental Watershed conducted during the spring of 1994 revealed that the median sediment size varied from 0.5 to 7 mm. The median size of the bed material on the watershed was predominantly composed of sand in the upstream portions of the main channel (Fig. 3) had increasing amounts of gravel in the central portion, and had progressively less gravel and a finer median size in the downstream part of the watershed. Superimposed on this the median size of the bed material varied from coarse to fine over the main channel with a spacing of about 1 km (Fig. 5). This is probably related to the mechanisms by which gravel is transported through the watershed. The median size of the bed material changes from 7 to 1 mm in the downstream 3.5 km of the watershed. As the elevation of the thalweg of this reach is stable and the bed material is composed predominantly of abrasion resistant chert, this change in size was attributed to preferential deposition of the gravel in bars lateral to the channel thalweg.

ACKNOWLEDGEMENTS

The bed material data used in this study was collected by John Cox, Richard Matsufuji, Cedric Shelby, Paul Seay, and Danny Brewer. This study would not have been possible without their dedicated efforts to collect the samples despite difficult conditions in the field.

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DISTRIBUTED FLOW ROUTING IN THE GOODWIN CREEK, MISSISSIPPI

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Abstract: A channel flow routing model is presented that together with watershed and sediment routing models predicts the response of control structures to reduce watershed and channel erosion on sediment yields. The model is aimed at long-term simulation of runoff from storm events in channel networks in the DEC watersheds in Mississippi. The model is tested for Goodwin Creek Watershed in northern Mississippi. Comparison of measured and computed values for the period 1982 through 1991 shows that the model satisfactorily predicts long-term statistics of the flow in Goodwin Creek. Peak discharges of storm events, which were among the highest two percent of all storms, were predicted within 7.5 percent. Discrepancies exist between measured and predicted runoff from single storm events due to the simplified assumptions involving hydrographs of lateral flow into the channel network.

INTRODUCTION

The Demonstration Erosion Control (DEC) Project is an interagency cooperative effort among the Corps of Engineers, Natural Resources Conservation Service (NRCS) and the Agricultural Research Service (ARS). DEC provides for a system of control structures to reduce watershed and channel erosion in the foothills of the Yazoo Basin in Mississippi. The control structures installed by the Corps of Engineers and NRCS are low- and high-drop structures, bank stabilization works, levees, flood retarding structures, and riser pipe structures. The USDA-ARS National Sedimentation Laboratory (NSL) in Oxford, MS, has been monitoring the Goodwin Creek watershed which is 21.3 km² in area and is located in the Long Creek DEC watershed in northern Mississippi. Fourteen instream gaging structures measure discharges, stages, and sediment yields. This data set provides an excellent means to develop numerical models that can at first simulate the measured response of the above control structures on the sediment yields, and then be extended to examine scenarios of measures to reduce or control sediment losses in watersheds. The NSL in cooperation with the Center for Computational Hydroscience and Engineering at the University of Mississippi have developed flow and sediment routing computer models that simulate the long-term effects of structures on flow and sediment yields.

This paper presents features of the flow routing model DWAVNET (Diffusion WAVE model for channel NETworks), which together with the sediment routing model BEAMS (Bed and Bank Erosion Analysis Model for Streams) and watershed model SWAT (Soil and Water Assessment Tool) (see accompanying papers of Li et al. (1996), and Bingner et al. (1996)) are aimed at the long term simulation of sediment yields and the evaluation of the effects of control structures on these sediment yields in the DEC watersheds. Results of applying the model to Goodwin Creek are shown for the period January 1, 1982 through December 31, 1991.

MODEL DESCRIPTION

The one-dimensional flow model DWAVNET has been developed to route flood waves through networks of channels of compound cross-section. It is based on the model WROUT developed by Ponce (1982) for the ARS. Because DWAVNET will be applied to long-term continuous simulation of storm events (10 to 50 years) an efficient numerical algorithm has to be established to reduce the computational burden, while retaining a justifiable accuracy of predicted stages and discharges to obtain reliable sediment yields from BEAMS. A diffusion wave formulation is chosen by omitting the inertia and convective terms in the one-dimensional De St. Venant equations:

$$\begin{aligned}\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} &= q, \\ \frac{\partial h}{\partial x} + S_f - S_0 &= 0,\end{aligned}\tag{1}$$

in which A is flow area, Q is discharge, q is lateral inflow, t is time, x is distance along the channel, h is flow depth, S_f is friction slope, and S_0 is bed slope. Besides reducing the number of operations to solve the governing equations (1), neglecting the inertia and convective terms will also improve the stability of the model. Unreported, extensive tests carried out by the authors show that the effects of this simplification on predicted discharge and stage hydrographs are negligible for the watersheds of interest. The Muskingum-Cunge method, being a diffusion wave model, could have been selected, however, backwater effects are not reproduced by this model. Backwater effects are important when confluences and hydraulic structures are present.

The method of Garbrecht (1990) to represent the hydraulic properties (HP) of channel cross-sections by power functions has been adopted. This method relates the cross-sectional area, top width, wetted perimeter and conveyance to water depth as $HP = \alpha h^\beta$. In this way the HPs can be updated each time step without performing the traditional, tedious interpolation from tables containing the values of the HPs for certain stages at each cross section. Also, derivatives of HPs that occur in the numerical solution method can be easily determined. The power function is an accurate approximation of the actual HP for channels of trapezoidal cross-section. Therefore, compound channel cross-sections are divided in trapezoidal main channel and left and right flood plain sections, each section having its own set of power functions. The channel cross-section is thus described by eight nodes. This schematization simplifies the updating of the cross section when erosion, deposition or bank failure occurs. Also, the coefficient α and exponent β of the power function are easily obtained with this scheme. In general, irregular cross sections are adequately described by eight nodes, as is the case for the 86 surveyed cross sections of Goodwin Creek that have been used to determine the geometry of its channels. Using separate power laws for main channel and flood plain segments enables the model to represent the important features of compound channel flow; in-channel hydrograph attenuation, and flood wave celerity differences in the main channel and the flood plains.

The four-point implicit scheme of Preissmann (1961) is used to discretize the governing equations (1), which are then solved by a double sweep method. A Newton iterative method is used to determine the dependent variables at each new time level. This method is well-established and largely applied to equivalent flow problems. Details of the method can be found in Ponce (1982)

and Cunge et al. (1980). The Preissmann scheme is compact and allows large time steps (time step is not limited by a Courant number condition), which makes it robust and efficient, especially for long-term computations. However, the time step may still be limited by the temporal variation of the flow and the linearization of the discretized equations where it is assumed that changes in the dependent variables are much smaller than the variables themselves. The set of discretized equations is solved link by link. A link is a channel segment between an upstream node and a confluence or structure, a channel segment between confluences or structures, or a single structure. Each link (except structures) usually contains several reaches, which are smaller channel segments between two nodes.

External boundary conditions have to be imposed at upstream nodes and at the outlet, whereas internal boundary conditions are imposed at junctions. A set of equations similar to (1) is derived for hydraulic structures (see accompanying paper of Langendoen et al., 1996), consequently structures are easily implemented in the numerical solution method and do not require internal boundary conditions. At upstream boundaries discharge hydrographs are imposed. At the outlet a looped rating curve corresponding to a diffusion wave is imposed. The compatibility conditions at junctions are conservation of mass and equal free surface elevation of the joining channel links.

The discharge hydrographs and the lateral inflow hydrographs are obtained from the watershed model SWAT (Arnold et al., 1993). SWAT computes daily runoff rates and volumes from upland subbasins into the channel network by employing estimated storm durations and times to peak, thus generating triangular discharge hydrographs. These discharge hydrographs are then added to the flow rates at the nodes of the channel network located at the outlets of the subbasins. Due to the daily time step used by SWAT, the duration of a storm event is always less than one day. Furthermore, the time at which the storm event occurs within the day is not known. The initiation of runoff is assumed to occur at midnight for the day of the storm event since the time at which the simulated storm event occurs within the day is not known.

RESULTS FOR GOODWIN CREEK

The channel flow in Goodwin Creek (GC) was simulated for ten years using DWAVNET. The channel network of GC was generated from a digital elevation map using a digital elevation and drainage network model (TOPAZ) (Garbrecht and Martz, 1995), and is shown in Figure 1. The channels of first Strahler order are omitted in the TOPAZ network employed by DWAVNET (hereafter referred to as the network) in order to combine the source subbasins with the immediate downstream subbasins, thus reducing the number of subbasins and reaches required for simulation. The resulting network consists of 67 reaches and 86 nodes, the nine junctions being represented by three nodes. Geometries of all cross sections are available from surveys conducted in 1977 by the Corps of Engineers. Cross-sectional changes did occur in the channels, which the model currently does not consider, but will when DWAVNET is coupled with BEAMS. Bed slopes vary from 0.035 to 7.9 percent through GC. Due to the large variability in bed slope and HPs along the network, additional nodes were added to satisfactorily perform the simulation. As a result, the number of nodes is 172, the number of reaches is 140, and the number of links is 45. These numbers include the nodes used for hydraulic structures. The reach lengths vary from 30 m to 365 m.

Four culverts are located in the network (see Figure 1). All culverts are concrete boxes, ranging from 2.44 m to 4.88 m in width and 1.83 m to 3.05 m in height. Fourteen instream measuring stations are located throughout the watershed. Each station consists of a supercritical flow flume constructed of reinforced concrete, except for the use of sheet metal at station 10. Ten of the measuring stations, stations 1 through 8, 13, and 14, are located in the stream network used in DWAVNET.

The ten-year simulation from January 1, 1982, through December 31, 1991, includes 913 storm events. Storm events are defined by the rain showers that occur on GC during this period and are separated from other rainfall by more than six hours. The yearly measured rainfall varied from approximately 1050 mm in 1988 to approximately 2000 mm in 1991. The Manning's coefficient was determined for the distributed channel network using information collected by Bowie (1989). A time step of eight minutes was employed resulting in a total number of 176,400 time steps for the simulation.

Figures 2 and 3 plot predicted and measured peak discharges against percentage storm exceedance for various measuring stations as determined from the USGS model, ANNIE (Lumb et al., 1990). The base flow has been subtracted from the storm peak discharges shown. The measured and predicted values are in general agreement, except for small (base flow) and large peak discharges. The geometry of the channel network and the linearization of equations (1) set a lower limit on the base flow that can be used in DWAVNET. This base flow is much larger than the actual base flow in Goodwin Creek, for example at station one the base flow rate of DWAVNET is $0.54 \text{ m}^3/\text{s}$, while the measured annual base flow is about $0.008 \text{ m}^3/\text{s}$. The volumes of simulated overland runoff added at the upstream nodes are therefore slightly larger than measured. This effect is especially noticeable for very small storm events with negligible runoff. However, these small storm events are not important to the morphology of the channel network. The discrepancy at large peak discharges is mainly due to the fact that some measuring stations become submerged, e.g. stations 2 (not shown), 3, 7, and 13. The measured flow at those large rates are then inaccurate. Also, the large peak discharges occur less frequently, producing less accurate percentage storm exceedances. The flow at station 13 is consistently underpredicted. This station is at the downstream end of a long channel link for which the overland runoff is added to the flow rate at the confluence just downstream of the station. Consequently, the discharges in this channel link are underpredicted.

Figure 4 shows the predicted and measured discharge hydrographs at station 1 for three storm events. These storm events are characterized by single-peak hydrographs, and therefore are more suitable to be used to compare measured and predicted hydrographs. Storms one and two have approximately a one-year return period, while storm three has approximately a four- to five-year return period. It is observed that the difference between predicted and measured peak discharge is approximately 15 percent. Furthermore, the simulated volume is smaller than that measured. This is partly caused by SWAT that generally underpredicts the runoff volumes in Goodwin Creek for single storm events, but, more importantly, satisfactorily predicts the long-term trends in runoff from GC (see Bingner, 1995). Experiments with smaller time steps have shown that predicted peak discharges will be larger. The triangular discharge hydrographs from the subbasin outlets that are routed through the channel network will distort when using large time steps. Reducing the time step will more accurately approximate their shape. However, the computational

time will then become too large.

Figures 5 and 6 compare predicted peak stage profiles between stations 1 and 2 with those measured by Murphey and Grissinger (1983) for two storm events in the second half of 1982. The storm of Figure 5 is moderate, the measured peak discharge at station 1 has a percentage storm exceedance of approximately eight. The measured peak discharge at station 1 for the storm event of Figure 6 has a percentage storm exceedance of approximately one. The stage profiles compare well for storm one. The stage profiles compare less well for storm two. Both Figure 5 and Figure 6 show a local change in the predicted free surface slope approximately 1450 m upstream of station 1. This change is not observed in the measured free surface slope. This deviation seems to contribute largely to the differences between measured and predicted peak stages. This location coincides with the upstream end of Kathryn Leigh's Bendway, a large meander loop that is not represented in DWAVNET's network.

CONCLUSIONS

DWAVNET can adequately simulate the long-term behavior of the flow in Goodwin Creek which includes the effects of hydraulic structures. In general, individual storm events were less satisfactorily predicted. The inadequate temporal resolution of storm events and ensuing upland runoff computed by SWAT inhibited a better determination of single discharge and stage hydrographs. However, as shown above, DWAVNET is able to acceptably simulate the discharge hydrographs in Goodwin Creek for well-defined single-peak storm events.

The main purpose of the model is to predict the long-term response of control structures to reduce watershed and channel erosion on sediment yields. Therefore, the above findings support the procedures adopted in DWAVNET to increase its computational efficiency without reducing its accuracy.

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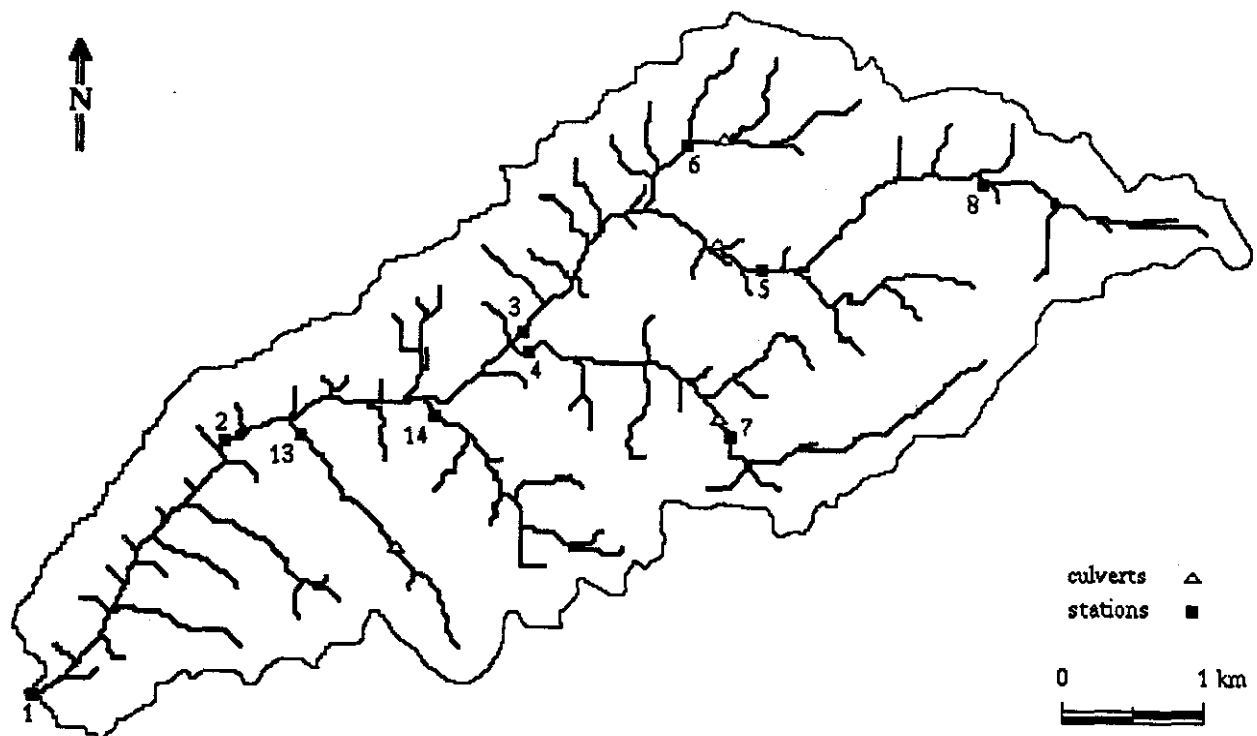


Figure 1 Channel network and structures of Goodwin Creek Watershed used in DWAVNET.

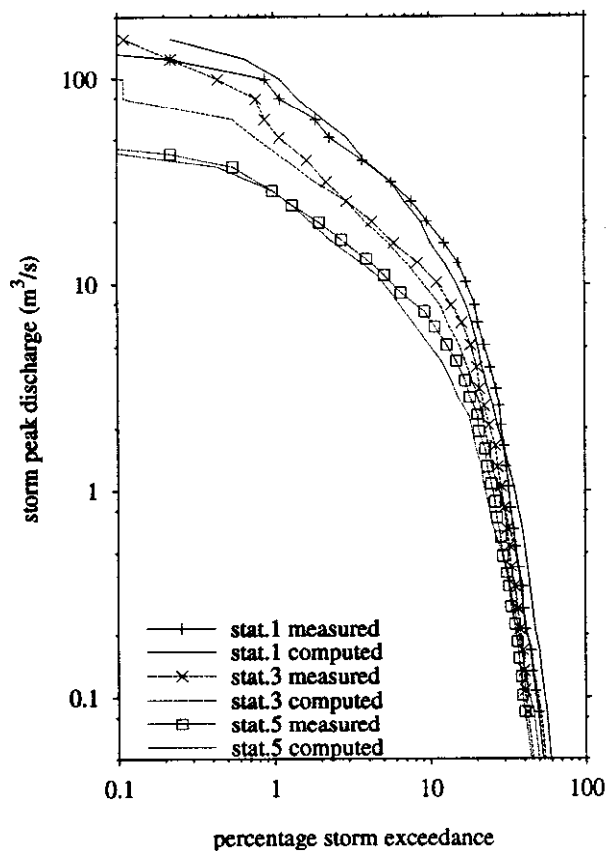


Figure 2 Predicted and measured percentage storm exceedance of storm peak discharge.

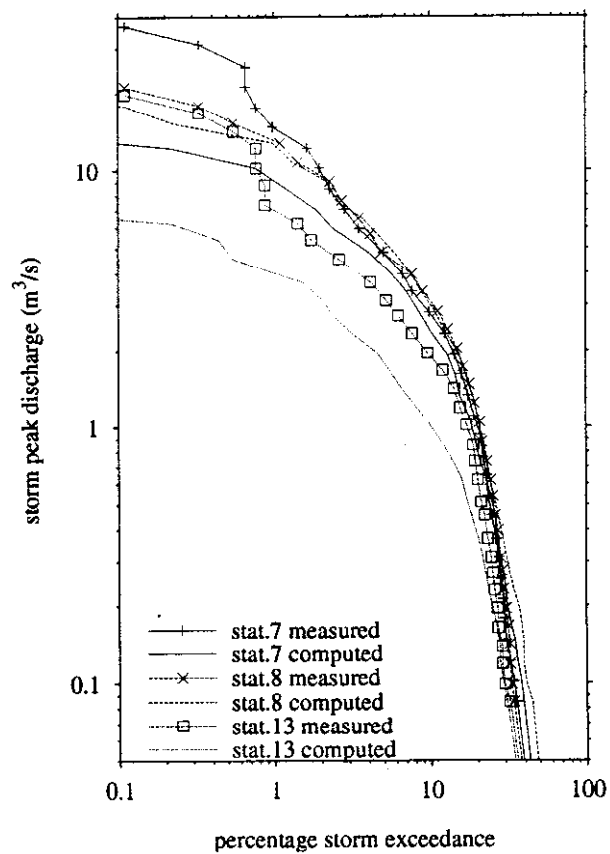


Figure 3 Predicted and measured percentage storm exceedance of storm peak discharge.

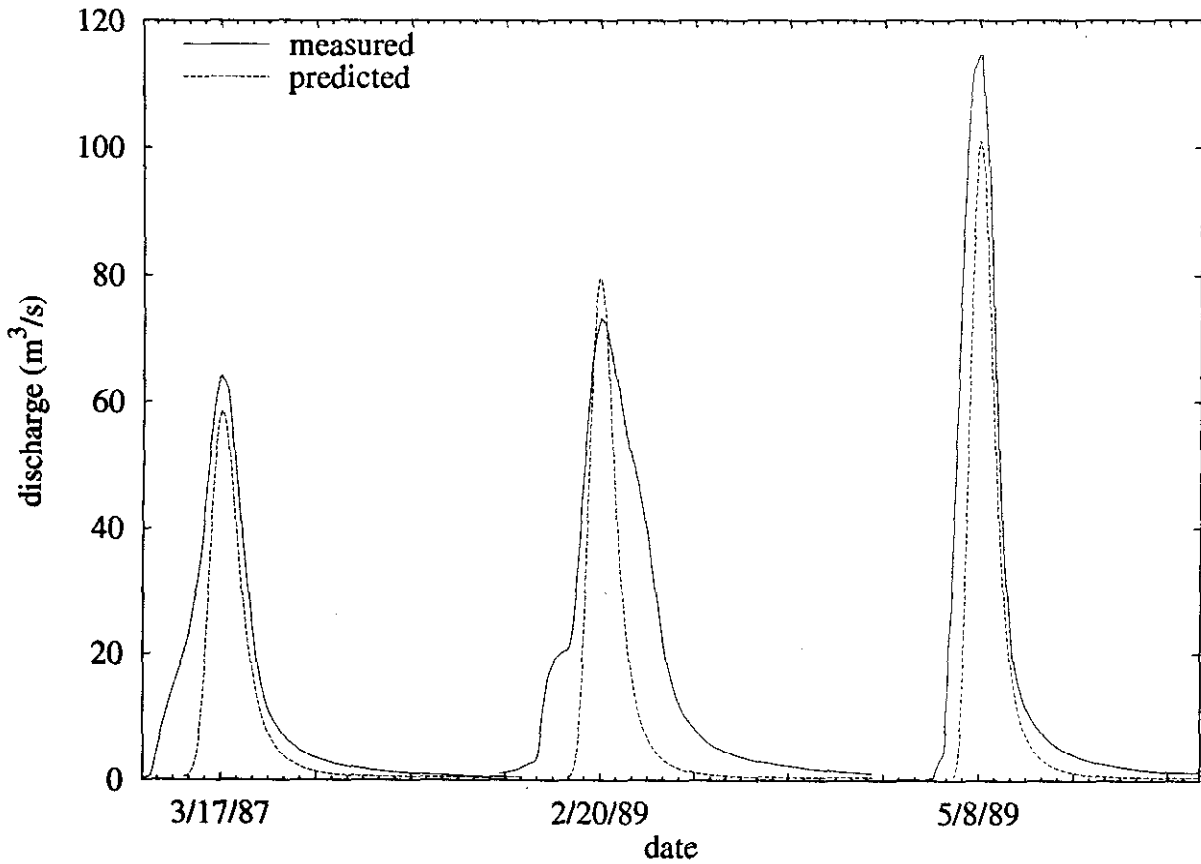


Figure 4 Comparison of predicted and measured discharge hydrographs at station 1 on March 17, 1987, February 20, 1989, and May 8, 1989. The time between minor tickmarks is one hour.

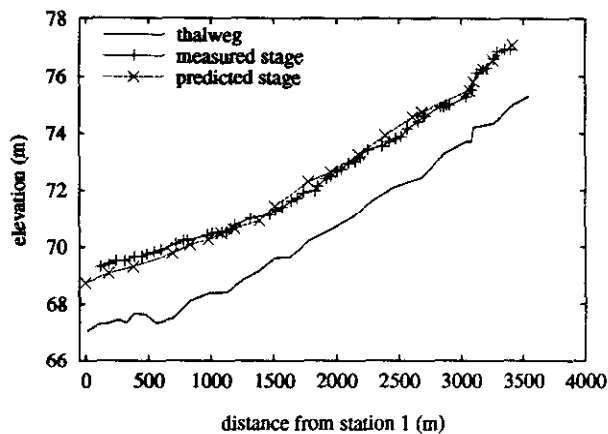


Figure 5 Predicted and measured peak stages between stations 1 and 2 on August 28, 1982.

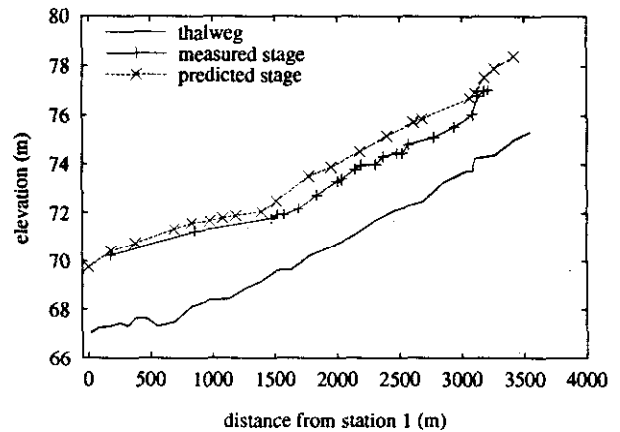


Figure 6 Predicted and measured peak stages between stations 1 and 2 on December 4, 1982.

EFFECTS OF STRUCTURES ON THE PROPAGATION OF FLOOD WAVES IN GOODWIN CREEK, MISSISSIPPI

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Abstract: The inclusion of hydraulic structures such as culverts, bridges, and drop structures, in a flow routing model employing Preissmann's scheme is presented. Application of the model to long-term simulation of the flow in the Goodwin Creek Watershed in northern Mississippi shows the influences of hydraulic structures on discharge and stage hydrographs. Discharge hydrographs are only slightly affected by structures. Stage hydrographs, however, are shown to be largely influenced by structures.

INTRODUCTION

Hydraulic structures are usually not included in conventional flow routing models. However, structures can act as controls or transitions. Consequently, local rating curves of the structure are fixed, producing a change in the state of flow compared to the structure not being located at that point. Since sediment transport is directly related to the local velocities, structures will not only affect discharge and stage hydrographs but also sediment movement in stream channels. If the effects of structures are not properly taken into account, incorrect responses will be predicted when routing floods and sediments.

The USDA-ARS, National Sedimentation Laboratory (NSL) in Oxford, MS, and the Center for Computational Hydrosience and Engineering at the University of Mississippi, are jointly developing computer models to predict the impact of control structures on reduction of watershed and channel erosion in the DEC (Demonstration Erosion Control) Watersheds in the Yazoo Basin in Mississippi. A proper implementation of these control structures, including low- and high-drop structures, bank stabilization works, and levees, is necessary in the computational modeling scheme. Other hydraulic structures generally present in channels are culverts and bridge crossings.

This paper presents the inclusion of hydraulic structures in the flow routing model DWAVNET (Diffusion WAVE model for channel NETworks, see accompanying paper by Langendoen et al., 1996). The model was applied to simulate the flow in the DEC watershed Goodwin Creek in northern Mississippi, which is operated by the NSL. Fourteen instream gaging structures measure discharges and stages. Because these measuring stations affect the flow, they also were evaluated using DWAVNET. Influences of hydraulic structures located in Goodwin Creek on the predicted stage and discharge hydrographs are shown.

MODEL DESCRIPTION

The flow routing model DWAVNET is based on the model WROUT developed by Ponce (1982). It employs the four-point implicit scheme of Preissmann (1961), and the discretized equations

solved by a Newton iteration method (see accompanying paper by Langendoen et al., 1996). To maintain the efficiency of the solution method, discharge-stage relations for structures are incorporated such that structures are an intrinsic part of the numerical algorithm. Analogous to regular flow in channels two equations are formulated to describe the flow through/over structures:

$$\text{continuity,} \quad Q_{\text{up}} = Q_{\text{down}}; \quad (1)$$

$$\text{dynamic,} \quad Q = f(y_{\text{up}}, y_{\text{down}}). \quad (2)$$

Here Q is discharge, y is stage, and the subscripts 'up' and 'down' denote computational nodes upstream and downstream of the structure. The dynamic equation (2) can be discretized by performing a series expansion up to first order as:

$$Q^{n+1} = Q^n + \frac{\partial f}{\partial y_{\text{up}}} \Delta h_{\text{up}} + \frac{\partial f}{\partial y_{\text{down}}} \Delta h_{\text{down}}, \quad (3)$$

in which n denotes time level, and Δh is the change in flow depth between time level n and $n+1$. As a result, the set of discretized equations is similar to that formulated for regular channel reaches, and is easily incorporated in the solution algorithm.

The following hydraulic structures are included: culverts, bridge crossings, drop structures and stream gaging stations. The latter are designed around flumes capable to accommodate the 100-year recurrent peak discharge and to operate in the supercritical regime to prevent deposition of sediment during high flow events. The formulations of the dynamic equations are presented below.

Culverts: Three flow states are distinguished for culverts: unsubmerged and submerged inlet, and drowned outlet. For inlet controlled flow (both submerged and unsubmerged) nomographs of the Federal Highway Administration (FHWA, 1985) were used in combination with the assumption of critical flow at the inlet. These nomographs are also employed in the HEC2 model (USAE, 1982). The nomographs allow the inlet control headwater to be computed for different types of culverts operating under a wide range of flow conditions. Equation (2) can be written as $Q = A\sqrt{gA/T} = f(y_{\text{up}})$, in which A is flow area in culvert, T is top width of flow through culvert, and g is gravitational acceleration. The energy head upstream of the culvert is related to that at the culvert inlet as $H_{\text{up}} = H + D(c_1 \tilde{Q}^2 - c_3 i)$ for an unsubmerged inlet, and $H_{\text{up}} = D(c_4 \tilde{Q}^2 + c_5 - c_3 i)$ for a submerged inlet. Here D is the height of the culvert, $\tilde{Q}$ is the dimensionless discharge defined as $\tilde{Q} = Q/A\sqrt{D}$, i is the slope of the culvert, and c_1 through c_5 are coefficients that depend on the geometry of the culvert's inlet. For a drowned outlet an orifice-type flow is assumed, thus, $Q = A\sqrt{2g\Delta y/c_L}$, in which $\Delta y = y_{\text{up}} - y_{\text{down}}$, and c_L is an energy loss coefficient.

Bridge Crossings: Low flow classes A and B are distinguished (Henderson, 1966). For low flow class A, Yarnell's (1934) equation is reformulated as $Q = A_{\text{down}} \sqrt{g h_{\text{down}} c_1 + \sqrt{c_1^2 + c_2 \Delta y / h_{\text{down}}}}$, where $c_1 = 0.1(K-0.6)$ with K being a pier shape coefficient, and $c_2 = 1/[5K(\alpha+15\alpha^4)]$ with α being the contraction ratio of the flow. For low flow class B, the flow through the bridge cross-section is critical and is similar to that for an inlet controlled culvert. The upstream energy head

is related to that at the bridge as $H_{up} = H + c_L(Q/A)^2/2g$.

Drop structures: The flow at the brink of a drop structure is critical for an unsubmerged drop structure. In this case the flow over the drop structure is evaluated similar to inlet controlled culverts and bridge low flow class B. When the drop structure is submerged an orifice-type flow is assumed similar to that of a culvert with drowned outlet.

Measuring Stations: For measuring stations, discharge-stage rating curves, $Q = f(h)$, are available that can be incorporated in the numerical solution method of DWAVNET.

RESULTS

Using DWAVNET the effects of hydraulic structures on stage and discharge hydrographs were examined in Goodwin Creek Watershed in northern Mississippi over a ten-year period from January 1, 1982 through December 31, 1991. The schematized channel network includes four culverts and ten measuring stations (Figure 1). All culverts are concrete box types, ranging from 2.44 m to 4.88 m in width and 1.83 m to 3.05 m in height. Simulations without and with these structures were performed.

Figure 2 shows the long-term effects of the structures on discharge hydrographs at measuring stations along the main stem of Goodwin Creek. It can be observed that the structures have little effect on the storm exceedance of storm peak discharges. The peak discharges are slightly smaller for the simulation with structures.

Figures 3 and 4 show predicted peak stage profiles between stations 1 and 2 (Fig. 1) for storms on August 28, 1982, and December 4, 1982. As a reference, the peak stages measured by Murphey and Grissinger (1983) are also shown. Generally, minor differences can be observed between the computed peak stages in the case with structures and the case without structures. For the storm event on August 28, which was a moderate storm compared to that on December 4, the computed peak stages just upstream of station 1 without structures are quite different from those computed with structures (Fig. 3). A backwater exists upstream of station 1 for the simulation with structures. A backwater is absent for the storm event of December 4. For large flow rates (as on December 4) the flow depths at measuring stations increase only slightly when discharges increase, whereas the opposite occurs for small flow rates. Consequently, larger backwater effects will result when discharge is small.

Figures 5 and 6 compare predicted peak stages for the channel link in which station 5 is located, (Fig. 1), for the same storm events as above. Station 5 is located approximately 1070 m upstream of the outlet of the channel link. This channel link also includes a simulation of a culvert approximately 400 m downstream of station 5. Backwater effects can be clearly observed upstream of both culvert and station 5. This should affect the sediment transport, because water velocities will be smaller in the backwaters. It can be observed that the thalweg upstream of both culvert and measuring station is nearly horizontal, denoting deposition at these locations.

Figure 2 showed that peak discharges in Goodwin Creek were only slightly affected by the

structures. Figures 7 and 8 show similar results for single discharge hydrographs just upstream of station 1 of the beforementioned storm events. The computed peak discharges of the simulation without structures are slightly larger than those of the simulation with structures. Also, the times to peak are quite similar. Figure 7 shows that the peak of the flood wave on August 28 arrives a little bit later at the outlet in the case when structures are included in the simulation. Figures 9 and 10 show the discharge hydrographs near the outlet of the channel link containing station 5. Differences in time to peak can be observed. Structures appear to slow down the flood waves.

CONCLUSIONS

The effects of hydraulic structures, here culverts and measuring stations, on flood wave propagation in Goodwin Creek Watershed were examined for the period January 1, 1982 through December 31, 1991. Structures had a minor effect on discharge hydrographs, reducing the peak values and the times to peak slightly. On the contrary, stage hydrographs are significantly affected by structures. Backwater occurred upstream of the structures, especially culverts. As a consequence, water velocities will be reduced upstream of structures, which will affect the sediment transport at these locations. Therefore, hydraulic structures have to be included in flow and sediment routing models to correctly predict sedimentation and erosion processes in channels.

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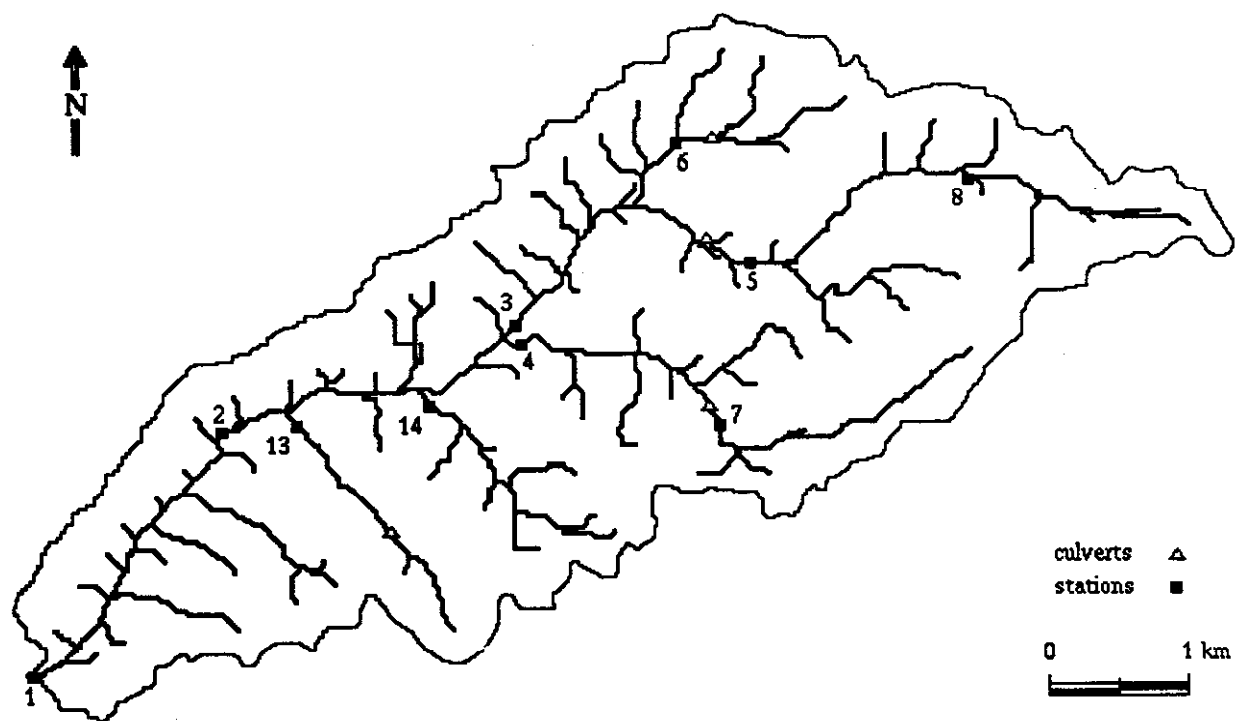


Figure 1 Channel network and structures of Goodwin Creek Watershed.

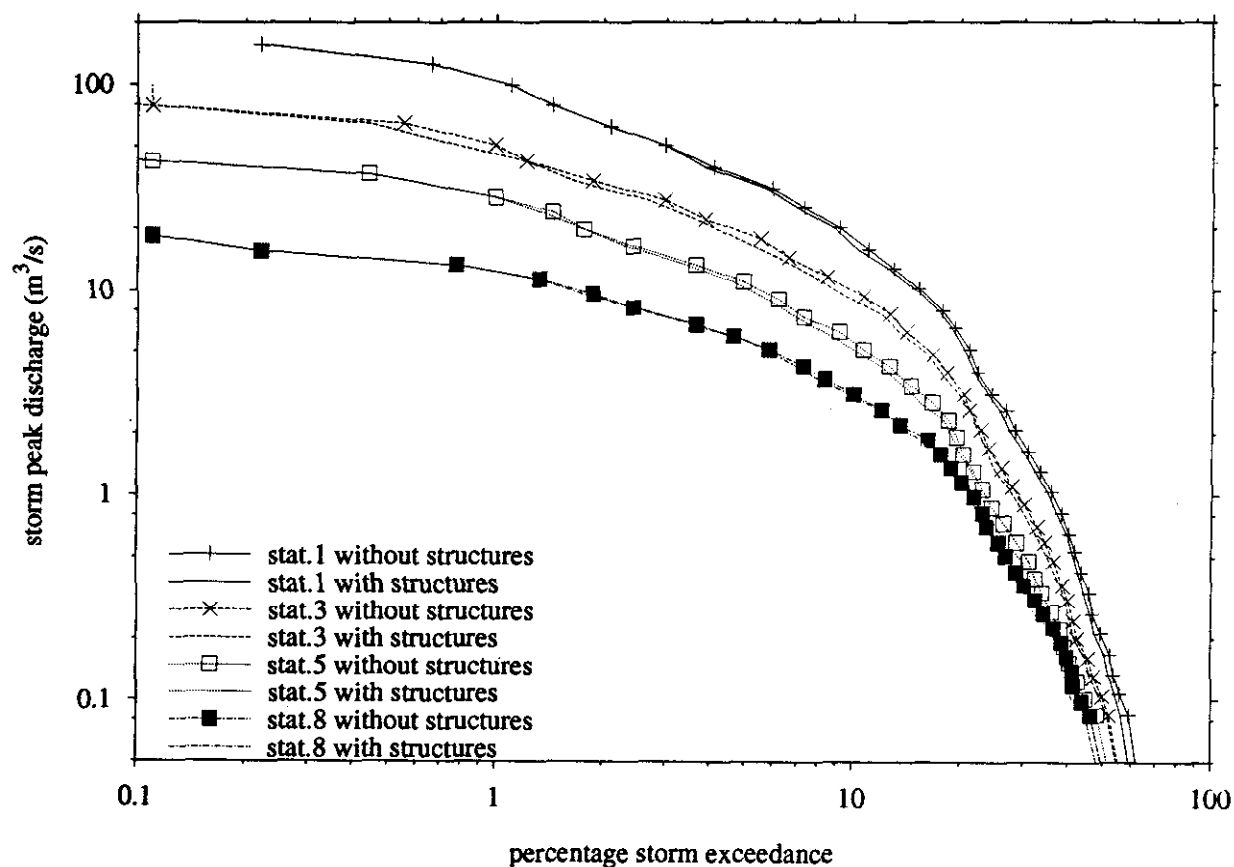


Figure 2 Predicted and measured percentages storm exceedance for the simulations without and with structures along the main branch of Goodwin Creek.

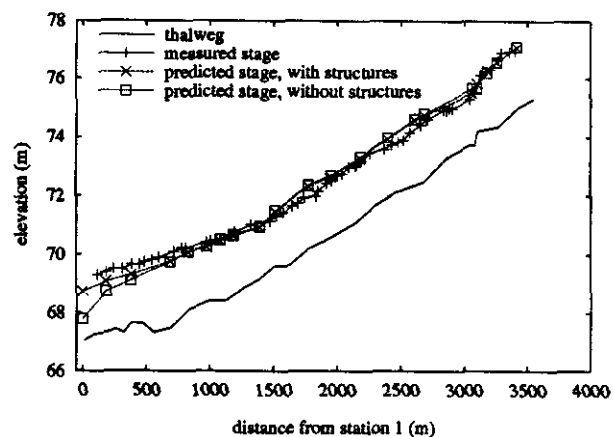


Figure 3 Predicted and measured peak stages between stations 1 and 2 on August 28, 1982.

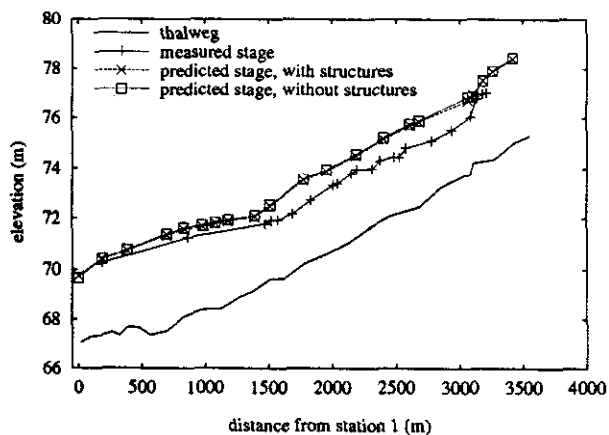


Figure 4 Predicted and measured peak stages between stations 1 and 2 on December 4, 1982.

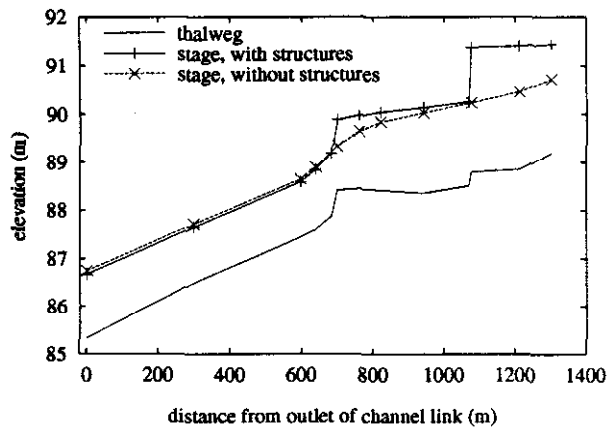


Figure 5 Computed peak stages near station 5 on August 28, 1982.

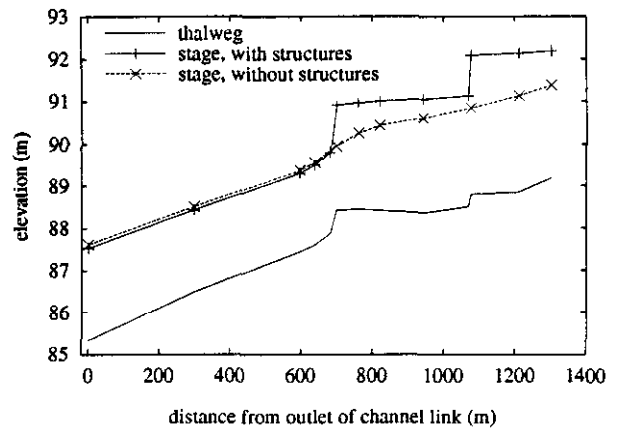


Figure 6 Computed peak stages near station 5 on December 4, 1982.

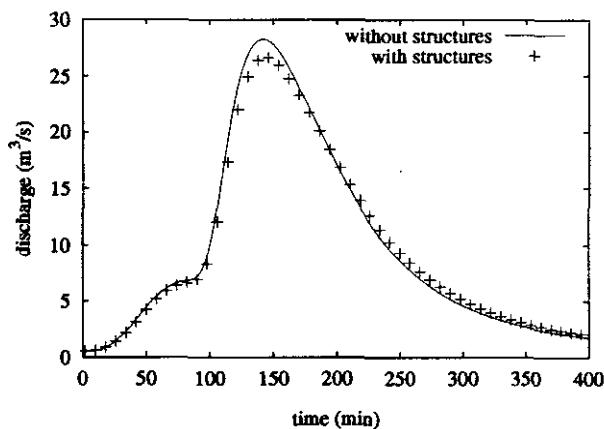


Figure 7 Computed discharge hydrographs just upstream of station 1 on August 28, 1982.

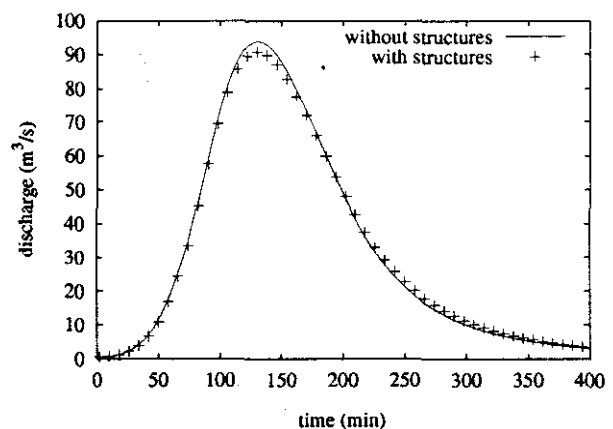


Figure 8 Computed discharge hydrographs just upstream of station 1 on December 4, 1982.

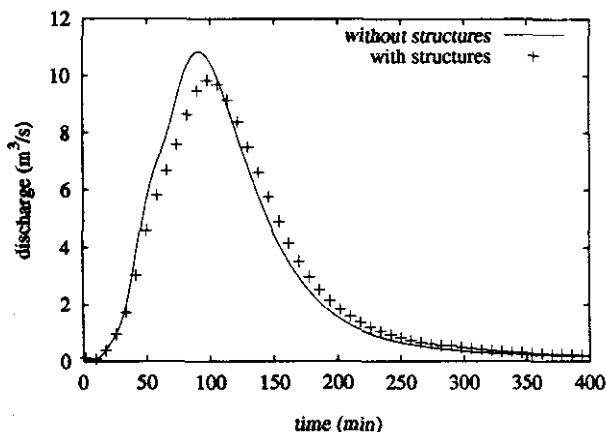


Figure 9 Computed discharge hydrographs near the outlet of the channel link that contains station 5 on August 28, 1982.

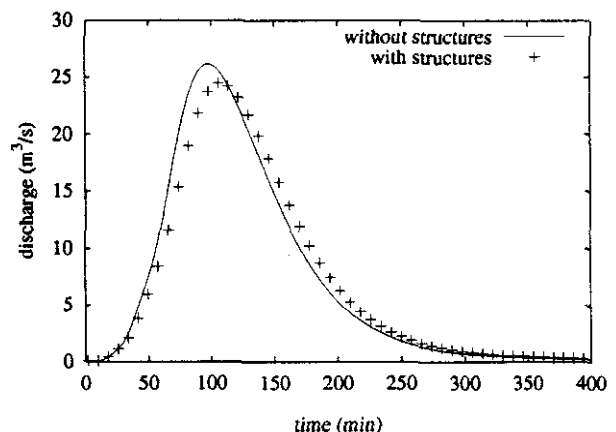


Figure 10 Computed discharge hydrographs near the outlet of the channel link that contains station 5 on December 4, 1982.

A MODELING APPROACH TO PREDICT LOCAL SCOUR AROUND SPUR DIKE-LIKE STRUCTURES

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Abstract: This paper presents a brief report outlining the major findings of the research project with the objective of applying a computational model to simulate a time-dependent phenomena of a local scour hole development around the tip of a spur dike or groyne in an open channel with an erodible bed. By the inclusion of effects of bed shear, turbulence intensity, and vortex strength and direction into a series of systematic computational experimentations, together with other refinements, reasonable results in agreement with physical model measurement have been obtained.

INTRODUCTION

River training hydraulic structures such as spur dikes, groynes, etc. are often installed along river banks to maintain navigation safety, control bank erosion and failure, enhance aquatic habitats, among others purposes. Since such a structure blocks the flow path near the bank and forces the incoming flow to change direction, it intensifies the turbulence and generate additional vortices. Strong bed erosion or local scouring therefore occurs around the tip of the dike and eventually it may undermine the integrity of the dike. Similar phenomena have been observed around bridge abutments, piers, etc. in river channels. Extensive studies on this problem have been conducted by hydraulic researchers using primarily physical modeling. Applications of numerical simulation to study the flow and the local scouring around the dike or pier have been quite limited. Most of the physical models were conducted under clear-water scour condition (no sediment motion in the channel reach upstream of the dike) in rectangular flumes so that the flow condition was simple and the influence of sediment from upstream would not be involved. The data of local scour experiments were used to generate empirical relations, which could be used to predict the maximum scour depth based on parameters such as water depth, particle size and properties of bed materials, the Froude number and the length of the dike. In natural rivers and streams, the geometric configuration and bed topography are highly irregular, bed materials are of varying types and sizes, plus that flows are much more complex than those generated in the laboratory flumes. For these reasons it is not a easy task to select an appropriate empirical function to predict the depth of a local scour at a specific site with full confidence. Therefore research activities have continued to be intensive at the present.

Due to the complexity of the physical processes, insufficient understanding of particle-fluid interactions, and limited capability of mathematical modeling, the major thrust of the investigations is still in the area of physical modeling with the objective of developing better empirical relations for engineering applications. But the researchers in the field have fully realized that physical modeling to produce empirical relations is costly, time-consuming and tedious. Furthermore, it is highly desirable to conduct some basic research to enhance our understanding of physical processes in scouring phenomena and to produce specific tools for solving these problems, so that we shall have a more general and cost-effective model for performing more reliable predictions of a variety of scouring phenomena in the near future.

A research project has been conducted at the Center for Computational Hydrosience and Engineering (CCHE) of The University of Mississippi. The approach is to verify and validate a computational model constructed by several modules developed at the CCHE for simulating the local scour phenomena. The validated model shall be used as a "computational flume" to simulate a series of well-designed cases of local scour "experimentations" systematically for generating a rich amount of knowledge to advance the state of the art in the field of local scour research. With sufficient amount of knowledge of scour hole development under a variety of flow, bed materials and geometric/topographic conditions, it is hoped that a more general and reliable prediction tools can be developed.

MATHEMATICAL MODEL OF FREE SURFACE FLOW

To simulate the local scour hole development realistically, one needs to select a time-dependent, three dimensional mathematical model describing turbulent free surface flow, sediment transport phenomena and morphological processes. As the first step of the research effort, a simplified set of equations is used.

Governing Equations: Navier-Stokes equations

$$\dot{u}_i + u_j u_{i,j} + p_{,i} / \rho - (v_t(u_{i,j} + u_{j,i}))_{,j} + f_i = 0 \quad (1)$$

the continuity equation

$$u_{i,i} = 0 \quad (2)$$

and, hydrostatic pressure

$$p = \rho g(\eta - z) \quad (3)$$

were applied. Water surface elevation, η , was solved by the free surface kinematic equation:

$$\dot{\eta} + u_\eta \eta_{,x} + v_\eta \eta_{,y} - w_\eta = 0 \quad (4)$$

where u_i ($i=1, 2, 3$) and (u, v, w) are velocity components, ρ is density and g is gravitational acceleration. f_i is the forcing function in i th-direction, x, y, z are coordinates in longitudinal, transversal and vertical directions. u_η, v_η, w_η are values of velocity components at the free surface, and $()_{,x}, ()_{,y}$, and $()_{,z}$ are partial derivatives in respective directions. The summation convention on repeated indices is adopted. The eddy viscosity v_t was modeled by the k - ϵ model

$$v_t = c_\mu k^2 / \epsilon \quad (5)$$

$$\dot{k} + u_i k_{,i} = (v_t / \sigma_k k_{,i})_{,i} + P - \epsilon \quad (6)$$

$$\dot{\epsilon} + u_i \epsilon_{,i} = (v_t / \sigma_\epsilon \epsilon_{,i})_{,i} + c_{1\epsilon} \epsilon / k P - c_{2\epsilon} \epsilon^2 / k \quad (7)$$

$$P = v_t(u_{i,j} + u_{j,i}) u_{i,j} \quad (8)$$

Commonly adopted coefficients of the two equation model or closure scheme were used. The Efficient Element Method (EEM) developed at the CCHE, (Wang and Hu, 1991) was used to solve these equations. Interpolation functions of this method have been improved, so that the finite element operators are more accurate and stable.

Boundary Conditions : The inlet flow was assumed to be fully developed with logarithmic velocity profile in vertical direction. The distribution of the longitudinal velocity along

transversal direction was assumed to be fully developed, which can be obtained by solving the flowfield in the same channel without the dike (CCHE2D - a depth averaged flow model may be used for this purpose). The inlet distribution of k was specified by

$$k = (\overline{u'^2} + \overline{v'^2} + \overline{w'^2})/2 \quad (9)$$

And the turbulence intensity components were calculated by Nezu's formula (1986). The energy dissipation, ϵ , was calculated from equation (5) together with the assumption that v_i follows a parabolic distribution. The boundary conditions near the wall were prescribed by assuming that velocities obey the log law of the wall. k and ϵ values were specified to be consistent with the local equilibrium approximation. At the outlet cross section, all properties have a zero gradient normal to the section ($\partial/\partial n=0$). These boundary conditions have produced reasonable results in similar cases simulated previously (Jia and Wang, 1993).

SEDIMENT TRANSPORT AROUND THE LOCAL SCOUR HOLE

To simulate channel bed change due to local scour or deposition, one usually adopts an empirical function for predicting the bed load according to the flow and bed material properties in that locality and calculate the suspended load by solving the convection-diffusion equation. When the total load is estimated, it is substituted into the sediment continuity equation to compute the bed elevation change. In some simple free surface flows over a mobile bed, the morphological changes on the bed can be estimated with reasonable accuracy; provided that the bed and suspended loads can be accurately predicted. However, applying this approach to simulate the scour hole development near the tip of a dike, or around any obstruction, results have been unsatisfactory. The primary reasons may be that the flow is three dimensional and turbulent; turbulence energy is intensified and vortices and secondary flows generated due to the presence of the obstruction in the flow. Therefore, if one is to use an the empirical function to predict the bed or the total load based on either the bed shear or critical velocity, he has no chance (other than coincidence) to predict the reality correctly.

As Gill (1972) noted in his paper that "the obstruction caused by the spur generates a complicated system of vortices which are believed to be the main cause of erosion. The primary horse shoe vortex impinges on the sand bed immediately in front of the spur and throws up the eroded material which is transported downstream by the main flow." And this observation has been confirmed by other researchers (Kawn and Melville, 1994). According to the measurement by Rajaratnem and Nwachukwa (1983), shear stress at the tip of the dike can be increased up to 4.9 times that of the approaching flow, the area of high shear stress is very small. Even by using the accurate value of shear stress near the dike, the scour prediction was not in agreement with the observations. This fact was verified by the Test Run 6 of the study of Rajaratnem and Nwachukwa (1984).

After having considered all the observed physical processes in and around a scour hole developed on the erodible bed, one should not have any difficulties to see that several other influences other than the bed shear have contributed to the scouring process. In order to achieve a more realistic simulation or prediction of the scour development, one must include the effects of turbulence intensity or energy, the strength and direction of vortices, the shape of the hole, and properties of the bed materials, etc. Hoffmans and Pilarczyk (1995) discussed the importance of the vertical vortices in the development of a scouring hole downstream of engineering structures. And

Fiorotto (1993) observed "A vertical axial flow forms that can remove the sediments from the bottom of the scour hole" somewhat similar to the effect of a tornado. The effects of vertical vortices, turbulence energy and the shear stress on sediment transport are combined into an effective stress term to be used in the simulation of scouring process near the dike:

$$\tau = \tau_s + \tau_v + \tau_t \quad (10)$$

where: τ_s is the shear stress obtained from the wall function which was used in the bed load transport calculation, $\tau_v = \rho u_{v*}^2$ was selected to represent the effect of vortex with u_{v*} being the shear velocity due to vortex, which can be calculated by

$$u_{v*} = k_v D (\omega/\omega_{\max}) \omega_{av} \quad (11)$$

where ω is the vorticity, $\omega_{\max}$ is the maximum value of ω , and ω_{av} is average value of ω near the dike, k_v is an empirical coefficient, and D is the length of the dike. Depth averaged value was adopted following the approach introduced by Zaghloul and McCorquodale (1973), vorticity and turbulence effects were proportional to τ_s in their formula:

$$\tau = \tau_s (1 + k_1 \omega/\omega_{\max} + k_2) \quad (12)$$

where k_1 and k_2 are empirical coefficients related to the vortex strength and turbulence intensity. The suspended sediment transport was used to represent the 'pick up' and 'thrown up' kind of transport characteristics. The concentration boundary condition at the reference level near bed was evaluated by van Rjin's formula

$$c_a = \alpha 0.03 \frac{d_{50} T^{1.5}}{a D_*^{0.3}} \quad (13)$$

α is an empirical coefficient calibrated in this study, a is the height of reference elevation very close to the bed surface, the mobility function T was obtained by the stochastic approach of van Rjin (1993). Three dimensional convection-diffusion equation for suspended sediment was also solved by EEM.

For comparison, a test case was run using the experiment data measured by Rajaratnem and Nwachukwa (1984), The physical model experiment was performed in a 0.91m wide straight flume having a thin plate dike of 0.152m long and 3mm thick projecting perpendicular to the vertical side wall. The d_{50} of the sediment on the bed of the flume was 1.4mm. The data provided include depth change with time, final scouring hole elevation contours and the bed form change of a section (RUN6). Flow conditions of RUN6 are: discharge $Q=0.0227\text{m}^3/\text{s}$, water depth $h=0.117\text{m}$, average velocity $V=0.207\text{m/s}$, and the Froude number $Fr=0.19$.

VERIFICATION OF THE HYDRODYNAMIC MODEL

Since the experimental data of the flow field is not given in the paper reporting the scouring measurement (Rajaratnem and Nwachukwa 1984), the velocity field measured in the same flume with the same dike and similar flow condition reported in their earlier paper (Rajaratnem and Nwachukwa 1983) was used to verify the hydrodynamic model. A 25x44x11 mesh system was used for both flow and local scour simulations (Figure 1). The mesh was designed to achieve cost-effectiveness so that the nodal density is high near the dike and close to the bed. For the computational convenience, the thickness of the dike used for the simulation (1.88cm) was

thicker than that used in the experiment (3mm). The velocities at two levels ($z/h=0.03$, and $z/h=0.85$) and in 4 sections ($x/D=2, 4, 6, 8$) were measured in the recirculation zone behind the dike. Line AB is the section where the topographic form of the scour hole was measured. Figure 2 shows the comparison of the simulated results and the measurements. It can be seen that the flow field predicted by the CCHE3D model is in agreement with the physical model data. The maximum width of the recirculation zone predicted agrees with that measured at the level $z/h=0.03$ and is slightly narrower than the measured at the level $z/h=0.85$. The length of the computed recirculation zone is almost the same as that measured, $L/D=12$. The computed velocities in the recirculation zone are somewhat less than the measured.

SCOURING SIMULATION RESULTS AND DISCUSSION

The simulation of the scour hole development started after the flow reached the steady state while the bed was assumed being temporarily frozen. The flow, sediment transport and bed elevation change models were de-coupled, because the time scale for morphologic change simulation was much larger than that for the flow simulation. The flow and sediment transport were assumed to be quasi-steady when bed elevation change was calculated. The flowfield and sediment transport were recalculated only when the maximum bed elevation change exceeded 1% of local water depth.

Results of the test run show that the bed load predicted by van Rijn's stochastic formula (1993) was too low to have any effect on bed elevation change. Because the simulation case is a clear water scour problem and τ_{cr} is quite large compared with the flow condition ($\tau_{cr}/\tau_0=4.3$). By examining the shape of the measured scour hole (Figure 4), it can be seen that the slope of the scour hole is approximately 25° - 30° , which is close to submerged sediment repose angle. The flow model could not predict such a high shear stress capable of moving the bed material against this steep slope. In the present study, the calculated suspend sediment transport seemed to have greater contribution to the scour process than the bed load.

Figure 3 shows the depth development of the scour hole with time. The simulation and the measurement agree very well in general. The measured scour depth increases monotonically although the rate of increase varies, while the predicted depth shows some fluctuations. The fluctuation of the scour hole depth during its development may be explained as due to the fact when the slope of the scour hole becomes too steep, sediment near the upper part of the slope could slide down into the deepest part of the hole. This process is controlled by numerical criteria in the simulation, and it is activated periodically according to the steepness of the slope. When the scouring hole becomes deeper and deeper, the finite element mesh in the hole region especially that near the bed will follow the surface of the hole and the shape of those elements have larger and larger distortion. To maintain the accuracy of the computation and limit the error caused by the mesh distortion, the simulation was terminated when the maximum depth exceed about 30% of the average water depth of the channel. A technique to refine the mesh in the scour hole and reduce the mesh distortion is under development.

The simulated bed elevation change along the cross-section AB (Fig. 1), near the dike and in the direction perpendicular to the main flow is shown in Figure 4. In this cross section, the trend of the simulated scour hole development and the measurement agree well in general. The simulated scour hole depth matches the measurement except at the beginning (40min. line) of the scouring process. The deepest point of the measured hole is right at the tip of the dike (0.15m), and its

shape is symmetric; the deepest point (0.13m) of the simulated scour hole does not occur exactly at the tip, and the shape of the scour hole at this section is slightly skewed. The bed elevation contours of the simulated scour hole are shown in Figure 5, which shows the simulated scour hole around the dike and the deposition downstream of the dike at four different times corresponding to those illustrated in Fig. 4. At the beginning, the center of the scour hole was at the tip of the dike (a), and it gradually moved towards point A, a little away from the tip. The area of the scour hole increases with time as its depth grows, and so does the deposition behind the dike. The scoured sediment was deposited behind the dike. The deposition mound was close to the dike at the beginning (Fig. 5a), during the development of the scour hole it was moved downstream gradually, and finally stabilized. Its distance to the dike was about 2–3 times of the length of the dike (Fig. 5d), which is again consistent with the data.

The difference between the physical model data and computational simulation may reflect the combined influence of many variables included in the sediment transport model, hydrodynamic model, and turbulence model. The accuracy of the standard $k-\epsilon$ model may not be enough to capture all the features of the turbulent flow near the tip of the dike. The isotropic eddy viscosity calculated by the $k-\epsilon$ model may not be accurate enough to predict the actual stresses, especially near the tip of the dike where the flow is fully turbulent and complex (Mendoza-Cabral 1993). This may be the reason why the primary vortex is not predicted very well, which appeared in the front of the dike but not extended long enough around the dike. The other reason could be that the empirical sediment transport functions need further improvement before more accurate prediction of scour hole development can be achieved.

CONCLUSIONS

The present research project has demonstrated that the CCHE-3D model is capable of predicting three dimensional, time dependent, turbulent flow around a spur dike. With the inclusion of the effects of shear stress and vertical vortex component, local scour around the dike can be simulated reasonably. In the case studied, the suspended sediment transport played a more important role in morphologic changes around the tip of the spur dike than that of the bed load. Because the former was found to be more effective to produce an accurate bed form in the vicinity of the dike. The simulated scour depth development in time and the shape of the scour hole agree well with the measurements. More research are needed to explore the applicability of more sophisticated turbulence model and the insight of the sediment transport process in the scouring hole region.

ACKNOWLEDGEMENTS

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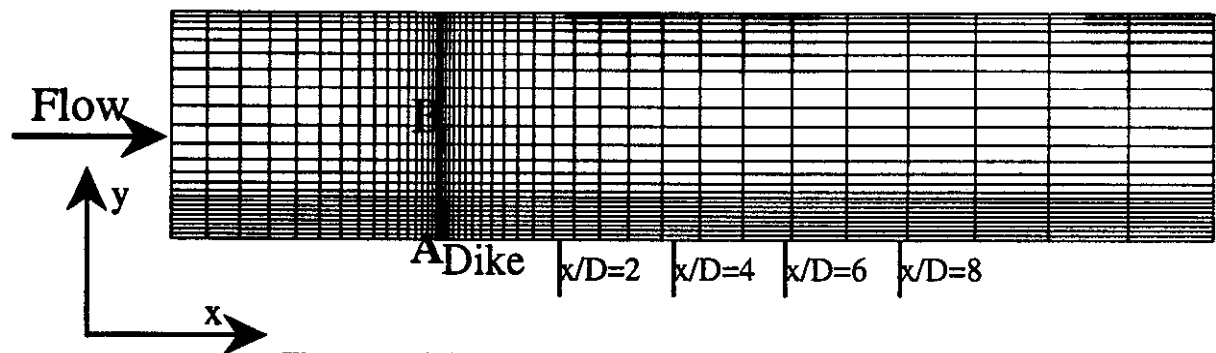


Figure 1. Finite Element Mesh and Measurement Locations

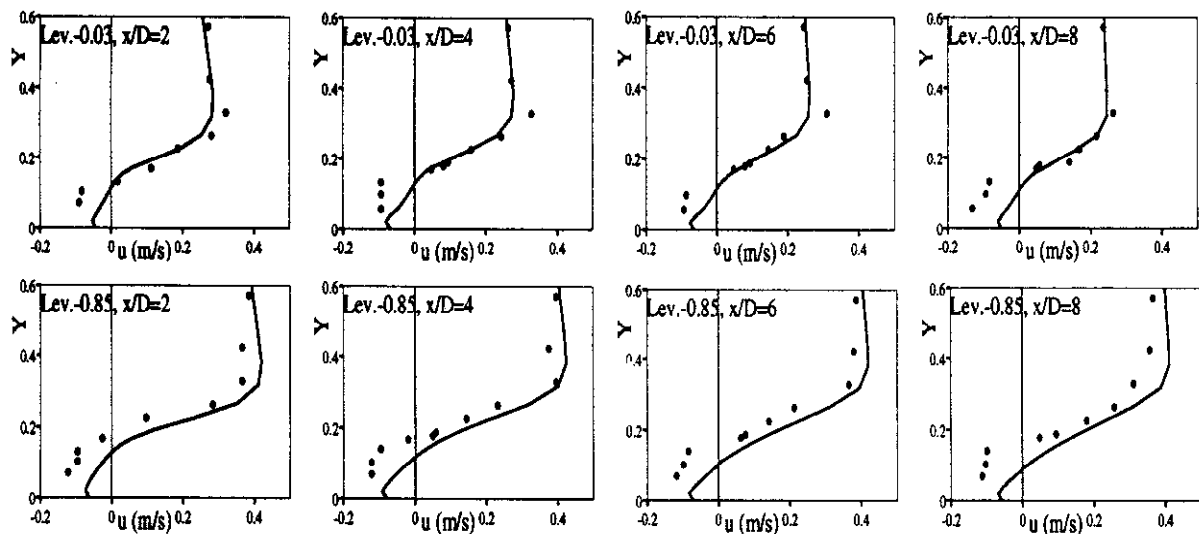


Figure 2. Comparison of the Simulation and Measured Flow Data,

(—: Simulation, *: Measurement)

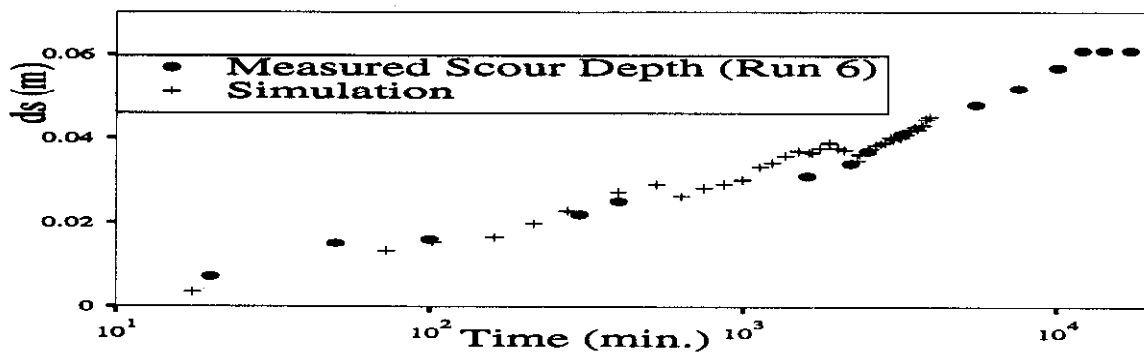


Figure 3. Simulated Scour Depth (d_s) Development

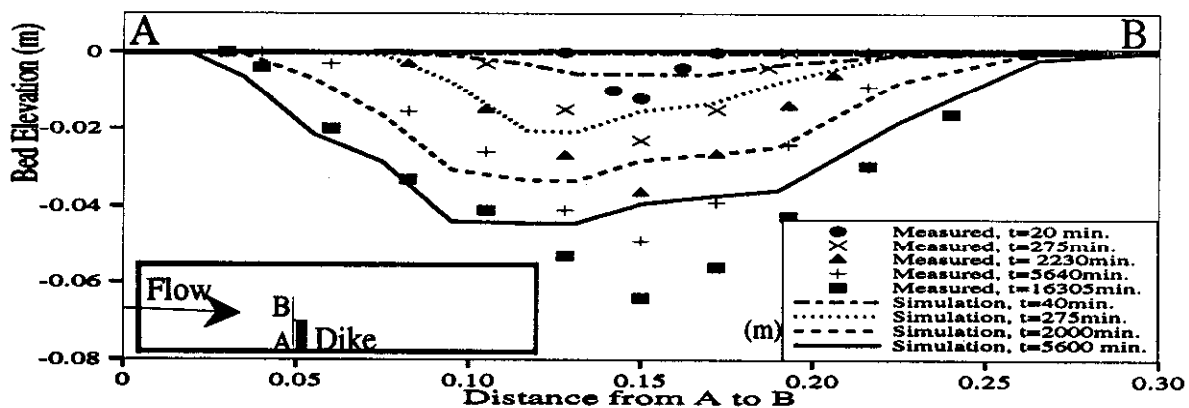


Figure 4 Scour Hole Development in a Cross Section

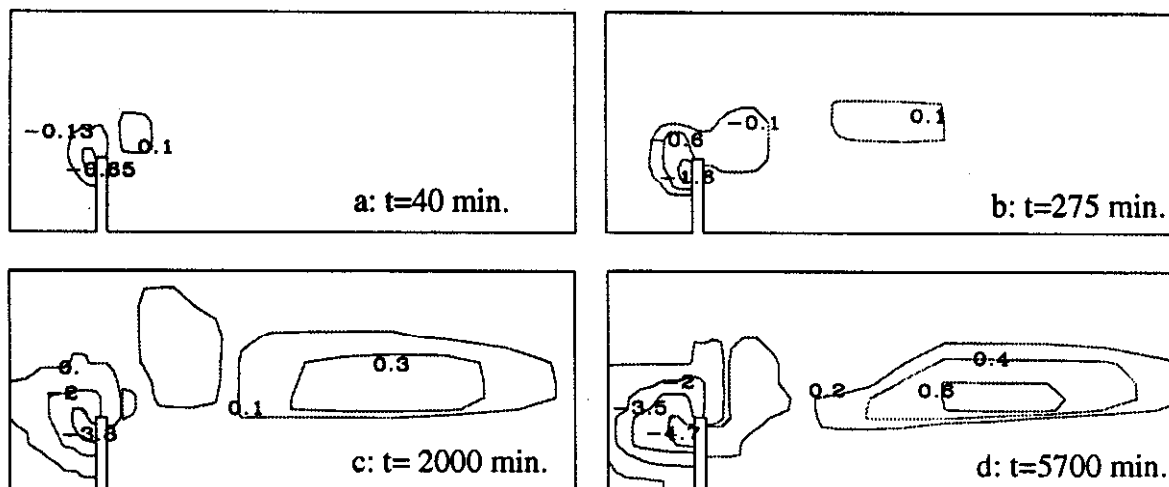


Figure 5 Simulated Scour Hole and Deposition at Different Times (unit: cm)

MODELING SEDIMENT TRANSPORT AND CHANNEL EVOLUTION

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Abstract: A numerical model to predict one-dimensional sediment transport and channel processes for large watersheds is presented. This model incorporates simplified basic channel bed and bank processes needed for numerical simulation of stream degradation, aggradation and bank erosion. Long-term evaluations can be performed with this model to determine the effects of channel and watershed erosion control measures on the channel. The model has been satisfactorily applied to degradation and aggradation laboratory experimental cases and to a case involving field data.

INTRODUCTION

Channel incision is a pervasive problem in northern Mississippi. Streams have responded by rapid incision to channelization and flood control projects implemented since 1930 resulting in both steeper grades and base lowering of tributaries. Monitoring projects conducted by Schumm et al. (1984) and by Murphey and Grissinger (1985) during the last two decades suggest that channels initially respond to base level lowering by deepening as knick points or zones migrate upstream. After bank heights exceed a critical threshold, mass failure occurs and channel widening ensues.

To simulate the dynamic response of streams to natural and man-induced changes in the watershed environment, a one-dimensional numerical model for channel processes has been developed. In this paper, a Bed and bank Erosion Analysis Model for Streams, BEAMS, is presented. The model works together with an uncoupled diffusion wave flood routing model, DWAVNET, and a watershed model SWAT (see accompanying papers of Langendoen et al. and Bingner et al.) to perform long term simulations of the impact of erosion control measures implemented within watersheds. BEAMS has been validated using laboratory as well as field data. The objective of this paper is to present the basic premise of BEAMS, its incorporation within the total watershed system simulation concept, and results of model validation.

MODEL DESCRIPTION

The BEAMS model includes two major submodels: a sediment transport and a bank erosion submodel. The first submodel performs sediment routing and computes bed erosion or deposition; the second one accounts for bank erosion and width adjustment. Each uses flow

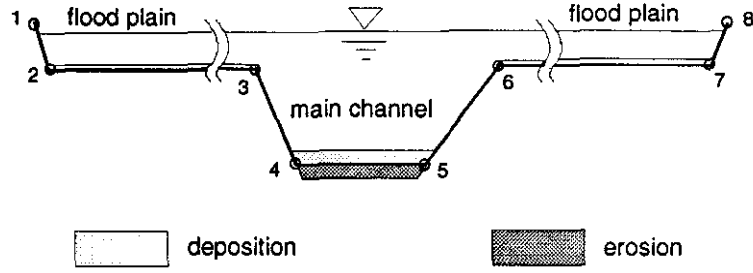


Figure 1: Diagram of erosion and deposition for the simplified cross section representation.

results computed by DWAVNET. An uncoupled solution of flow and sediment transport equations is used during each time step.

Because of the long term simulation objective, simplifications were made to the model to ensure that it is computationally efficient, e.g. an arbitrary channel cross section is represented by a polygonal geometry of eight points shown in Fig.1, in which flood plains and the main channel are of trapezoidal shape. The main channel bottom are erodible unless specified otherwise due to control points such as hydraulic structures, hard rock beds, etc.; on overbank areas only deposition or limited erosion within a previously deposited layer is allowed.

Heterogeneous bed materials are represented by nine size-classes with different fractional representations and densities. A sediment transport predictor, SEDTRA (Garbrecht et al., 1995) is used, which calculates for given flow conditions the total load transport capacities for each sediment size-class. The effect of the bed material size distribution on the initiation of motion of the individual sizes is considered in SEDTRA by adjusting the size used to calculate critical shear stress for each size-class.

The local bed elevation changes are determined from the sediment transport continuity equation which is solved for each size-class. The equation reads

$$\frac{\partial Q_s}{\partial x} + \frac{\partial}{\partial t}(AC_s) + (1 - \lambda) \frac{\partial A_b}{\partial t} = q_{sl} \quad (1)$$

in which Q_s = total volumetric rate of sediment discharge; C_s = volumetric concentration of total sediment load; q_{sl} = total volumetric rate of lateral sediment inflow per unit length; A = flow area; A_b = volume of sediment stored in the bed per unit length; λ = porosity of the bed material; x = streamwise distance; and t = time. A finite difference numerical solution is used to solve this partial differential equation.

In case of bed erosion, a hydraulic sorting may occur as finer particles are carried away and coarser ones remain. Consequently, an armor coat is formed in the top layer. This process is simulated by keeping track of the sediment fraction changes in a top active bed layer. In BEAMS, depth of the active layer is estimated according to height of the bedform which

is most likely to occur, and kept constant during simulation. The active layer shifts up or down as the bed aggrades or degrades. The fraction for each size-class is updated according to:

$$(P_{i,k})_t = \frac{D_{t-1}(P_{i,k})_{t-1} \mp (\delta_{i,k})_t \pm (\sigma_{i,k})_t}{\sum_{k=1}^m [D_{t-1}(P_{i,k})_{t-1} \mp (\delta_{i,k})_t \pm (\sigma_{i,k})_t]} \quad (2)$$

in which D = active layer depth; $P_{i,k}$ = fractional representation of size-class k at section i ; m = total number of size-classes; δ = fraction of sediment eroded from or deposited into the active layer; and σ = fraction of sediment which enters or leaves the active layer from beneath. The $\mp$ sign depends on whether erosion or deposition takes place.

Equation (2) implies that the current fractions stored in the active layer depend on particles entering or leaving the layer from both upper and lower levels. In a long-term simulation, it is possible that the original composition will be interrupted in the layer which is below the active layer, hence a mixing layer is introduced to keep track of the size fraction changes in this layer. In total, there are three bed layers defined for each section and the lowest one, or inactive layer, is used for storing the undisturbed substrate material. Compositions of the top active and mixing layers are updated in each time step for all sections.

Both vertical bed movement and channel widening processes occur as natural streams adjust to new boundary conditions. Therefore, a comprehensive stream model must take both processes into account. BEAMS assumes that channel widening is caused by both bed deepening and lateral hydraulic erosion (defined hereafter as basal erosion). These two processes undermine bank stability by increasing the bank height and by steepening the bank slope, respectively, eventually resulting in mass bank failure.

Alluvial stream banks are almost exclusively composed of cohesive material. Yet, cohesive sediment erosion remains a difficult problem to solve using the current state of knowledge. Therefore, an empirical formula by Alonso and Combs (1986) is used, which states that as long as shear stress at the bank toe (τ) exceeds the critical shear stress, $\tau = \rho g h S_f > \tau_c$, the basal erosion rate (E) can be estimated by $E = C (\tau - \tau_c)^\alpha$, where ρ = density of water; g = acceleration due to gravity; h = water depth; S_f = friction slope; C and α = empirical coefficients; and τ_c = critical shear stress for bank material movement, which is a complex function of the properties of bank material, vegetation density, soil pore pressure, etc., and it is thus treated as a calibrating input parameter.

Unlike basal erosion, mass bank failure occurs as a catastrophic, isolated event. This phenomenon has been studied extensively by Little et al. (1982) who evaluated bank stability by using critical bank height as a criterion. However, since cross sections of natural streams are highly irregular, the critical bank height criterion is difficult to apply. In BEAMS, plane slip failure surfaces are assumed, and

$$\sum_{j=1}^n (\rho_j \Omega_j) \sin \theta = \sum_{j=1}^n (\chi_j \ell_j) / g + \sum_{j=1}^n (\rho_{b,j} \Omega_j \tan \phi_j) \cos \theta \quad (3)$$

in which, Ω_j = volume per unit length of the j -th segment above the failure plane; n = total number of segments involved; χ = cohesive resistance force per unit bank area; ℓ = segment length along the failure plane; ϕ = repose angle; θ = angle between the failure plane and horizontal surface; and ρ_b = density of bank soil.

Equation (3) describes the critical condition when the bank is just about to fail. The left hand side of the equation is associated with the sliding force due to gravity and the right hand side with the frictional and cohesion forces. The failure angle (θ) is determined by solving Eq.(3) iteratively.

During the computation, debris from the failed bank is stored in a conceptual storage and released at pre-defined rates in subsequent time steps. The released bank material is treated as a special source of lateral sediment input in Eq.(1). The total volume and contents of the storage are updated in each time step.

Model Validation

The model has been tested against two well documented laboratory flume experiments on channel degradation and aggradation, respectively. Field data of Goodwin Creek, located in northern Mississippi, has also been used to test the channel widening simulations.

In one experiment by Ashida and Michiue(1971, Run 6), a nonuniform sand-gravel bed was placed in a flume measuring 20 m long, 0.8 m wide with an initial bed slope equal to 0.01. Clear water was allowed to flow in at a constant rate of 0.0314 m³/s. For numerical modeling purposes, the flume was divided into 13 unequal intervals from 1 m to 4 m , with a largest interval at the most upstream reach to compensate for the non-equilibrium transport effect. A Manning coefficient of 0.015 and active layer depth of 9 mm were used. Fig.3 shows the simulated and measured erosion depth plotted against time, at selected intervals in the flume. While the trend and the final depths of the erosion agree closely with the measured ones for all three sections, significant differences occurred during the earlier phase of bed degradation. The underprediction of the model can be caused by several reasons, e.g. the active layer depth, insufficient size-class representation, formation of bedforms and the actual condition at the entrance of the flume. The final result of size distribution in the armor layer is plotted in Fig.4 together with the measured ones. The discrepancy corresponding to the coarse particles could be caused by insufficient size-class representations in the model.

Seal et al. (1995) conducted experiments in a flume 45 m long and 0.3 m wide, with an initial bed slope of 0.002. In test Run-1, non-uniform sediment was added to the flow at the upstream flume end at a quasi constant rate of 0.19 kg/s and the water inflow rate was 0.049 m³/s . Due to sediment overloading a wedge shaped deposit gradually developed with a wave front moving downstream. Meanwhile, the bed elevation upstream continued to rise. Predicted aggradation agrees well with that measured, see Fig.5. The deviations are only minor.

Data collected from Goodwin Creek show that the measured left bank of the reach near the outlet of the watershed has retreated about 1 m in the period from October 1982 to October

1983 (Fig.6), while the channel bottom essentially did not change. Using the measured hydrographs during this period and simplified cross sections, the reach containing this cross-section was simulated. Fig.6 shows the comparison of the computed cross section at the end of October of 1983 with that measured. The simulated cross section shows a retreat of the left bank, that is more than measured retreat. The right bank shows a slumping of the material in the middle of the measured bank. BEAMS can not simulate exactly when a bank fails, but just the relative trends of bank failure.

SUMMARY AND CONCLUSIONS

A one-dimensional numerical model for simulation of stream bed erosion, deposition and channel width adjustment has been developed. The model is applicable to heterogeneous bed and cohesive banks. The hydraulic sorting and bed armoring process can be simulated. Model validation tests have been performed using both laboratory and field data. The predicted results of channel degradation, aggradation and the size distributions of the armored layer show that the model simulates the relative trends of the actual data. In addition to its capacity of simulating the vertical bed elevation changes, the model also predicts the relative channel widening processes.

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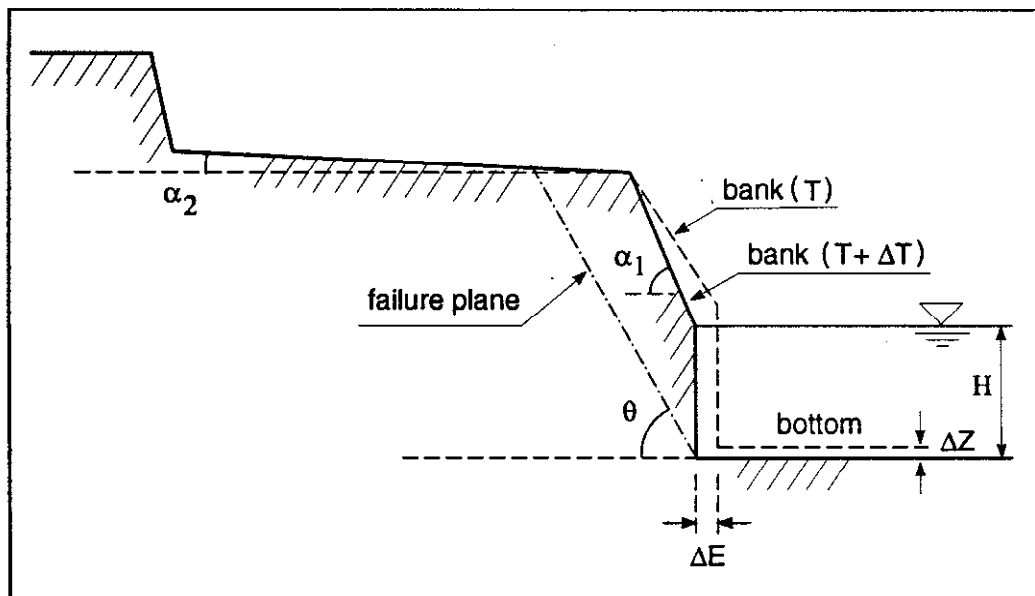


Figure 2: Definition sketch for bank stability analysis. The symbols are: θ = angle of failure plane; α_j = bank slope angle of segment j ; ΔZ = distance of bed erosion; ΔE = distance of basal erosion; T and ΔT = time and time step, respectively.

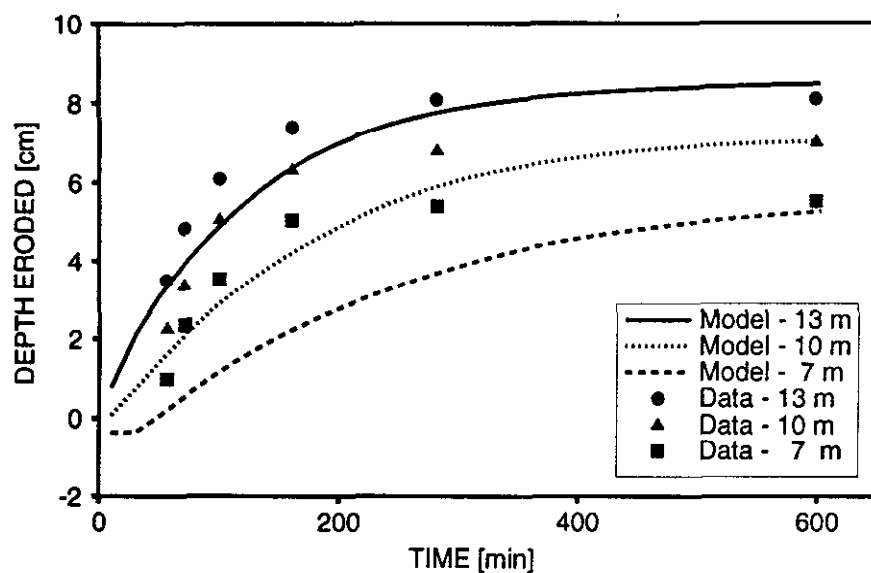


Figure 3: Comparison of computed and measured depth of erosion at three locations upstream from outlet weir (Experiment by Ashida and Michiue, Run 6, 1971)

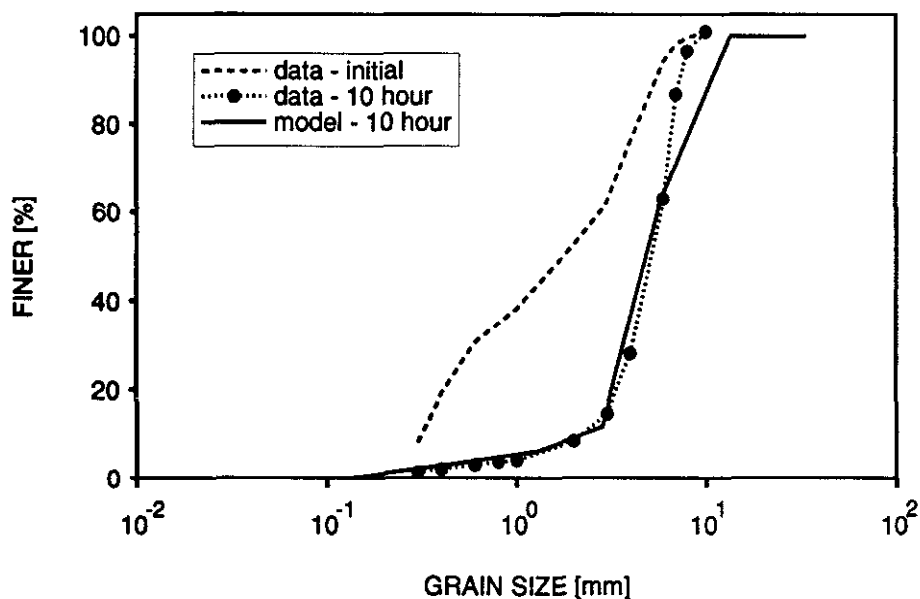


Figure 4: Comparison of computed and measured grain size distributions at 10 m upstream from outlet weir (Experiment by Ashida and Michiue, Run 6, 1971)

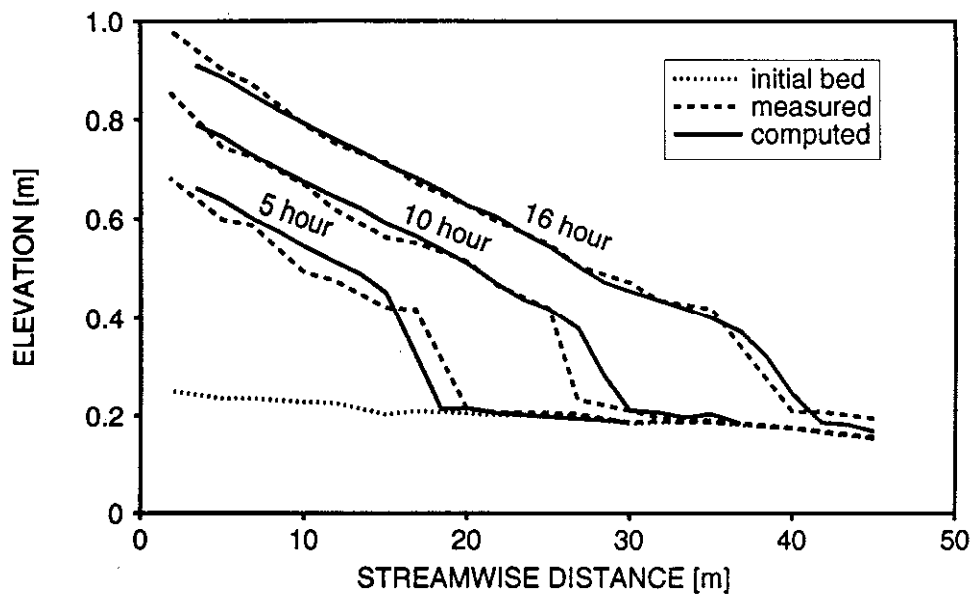


Figure 5: Comparison of computed and measured bed profiles at different time (Experiment by Seal et al., Run 1, 1995)

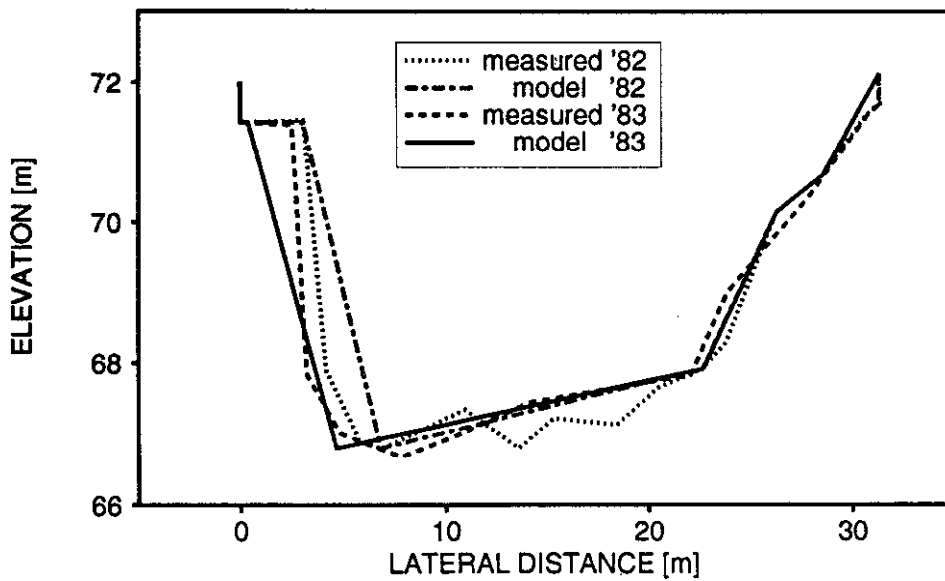


Figure 6: Channel cross-sectional change at a selected location of Goodwin Creek.