FINAL REPORT

PB99-123580

Stream Channel Migration Effects on Bridge Approaches and Conveyance

Project IA-H5, 1994

Report No. ITRC FR 94-4

Prepared by

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September 1998

Illinois Transportation Research Center
Illinois Department of Transportation

ILLINOIS TRANSPORTATION RESEARCH CENTER

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16. Abstract

The main objective of this project is to investigate channel protection techniques, especially those with low-cost and low-maintenance, for control of channel migration near bridge approaches. Such low cost techniques are mostly applicable to small streams than to large rivers. A literature review was conducted. Channel migration is mitigated either through protection of the bank and/or bed erosion and considerable deposition, or through instream structures to redirect the flow, or both. Useful control techniques include: instream structures such as bendway weirs, riprap and masonry revetment, bioengineering techniques such as willow posts, and geosynthetic membranes. Advantages and disadvantages of these techniques are summarized. Based on the information obtained from a survey of Illinois bridge sites and under the guidance of the Technical Review Panel, four sites were selected for further study of applicability of the protection techniques. For every site it has been identified that the meander pattern of the stream is a major factor for the scour and flow action endangering bridge safety. Therefore, the general goals of the suggested alternative actions are to stabilize and/or improve the stream meander alignment and to control and protect against local bank scour. The alternatives for a site range from positive "active" control structures to "passive" protection and sediment deposition enhancement. Generally, the more "active" alternatives are more costly but have less failure risk when compared to the more "passive" alternatives.

17. Key Words
Bank protection*; Bridge scour*; Channel migration*; Erosion; Hydraulics; Hydrology; Low-cost techniques; Meander; Monitoring; Risks; Stream bank

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We thank those officials who assisted in case study site selection and hosted site visits, especially, Mr. Wayne Kinney from the National Resources Conservation Service.

We also gratefully acknowledge Research Assistants Juan Fedele, Dahlia El-Kaddah, and Juan A. González for their role in this effort.

1. Background and Project Statement

Stream channel migration and associated sedimentation problems are the consequence of the dynamic adjustment of a stream's flow in an attempt to approach an equilibrium condition. Given an erodible land and a specified input of water, the flow will gradually form a channel through erosion and deposition while carrying some sediment downstream. In this land form evolution process, even for a constant discharge, the channel will gradually grow and migrate trying to establish a dynamic equilibrium condition. This equilibrium condition corresponds to a minimum energy expenditure for the given flow. For most natural streams, the normal equilibrium channel pattern is meandering in form. Straight channel is a rare equilibrium condition which exists only for exceptional cases. This phenomenon is similar to the pattern of a rope held between two hands. The rope is straight only when it is subject to considerable tension (Langbein and Leopold, 1966). The channel of a dynamic equilibrium stream is not stationary with time. Sediment eroded and transported downstream cannot return upstream by the flow. For a dynamic equilibrium stream, the channel's meandering pattern repeats itself in cycles over a long period of time. Under the ideal equilibrium conditions, for a constant discharge, the meandering loop will migrate downstream with continually time-changing loop geometry. Given a long period in a time scale of decades or centuries, the meandering pattern will repeat itself in a periodic manner (Figure 1-1). Although a natural condition is more heterogeneous and complicated than ideal equilibrium, the repetitive migration of meandering loops has been observed in the field (Brice, 1974).

Any natural or human induced changes in the flow or land conditions (including farming practice and channel straightening) will cause the channel to try to readjust itself to approach a new equilibrium condition. Theoretically, it will take an infinitely long time for the flow to achieve the new equilibrium; although, in fact, a majority of the changes will be accomplished in a shorter time span of decades, years, or even months. Thus, it is clear that any change in land use or in channel geometry will change the sediment transport conditions in the channel, and the channel morphology downstream will be altered.

Moreover, seasonal variation of runoff due to individual time-varying rainfall events causes the channel to continually adjust itself in an attempt to approach a new equilibrium that can never be achieved. In other words, the natural fluvial process is a dynamic process resulting in an ever changing geomorphology. Any built structure in the channel, including bridges and culverts, acts as an obstacle to the natural migration. Conversely, migration of the channel could seriously affect the bridge. For such cases, the protection measures should be upstream of the bridge to stabilize the approaching channel, not locally at the bridge.

The conventional concept of stream erosion protection, including that for bridges, is to protect and stabilize the channel preventing it from any migration. This channelization approach, in fact, is to restrict a naturally migrating dynamic channel into an artificially stationary static channel. With today's technology this is an achievable but expensive approach. This approach is unlikely to have effective low cost, long-term alternatives if one looks beyond a few years of time span.

An alternative concept is to stabilize and protect only the portion of the channel near, particularly just upstream of the bridge, while allowing the stream to exercise its wishes further upstream and downstream. Essentially, this approach makes the bridge a control point of the stream pattern migration. This approach may offer relatively less costly mitigation and protection but it definitely requires more careful design and execution, which in turn, are highly site specific.

Through the years, many channel mitigation and bank protection techniques have been developed in the United States, ranging from spur dikes, revetments, to bio growth on banks. Descriptions of most of these techniques can be found in the literature. A summary by Lagasse et al. (1997) on most techniques compiled in a project for FHWA is reproduced here as Table 1-1 which contains a good list of references. As mentioned by Brice and Blodgett (1978) and many others, applicability of these techniques depends on the stream characteristics. They show a good classification figure of stream properties affecting channel stabilization, and it is reproduced here as Figure 1-2. Table 1-2 lists major stream characteristics relevant to channel migration and bank erosion summarized by Brice and Blodgett (1978). Most of the techniques listed in Table 1-1 are relatively high cost and unsuitable for some small streams.

Illinois, like elsewhere, has many bridges under the threat of channel migration. Low cost and low maintenance counter measures are sought to protect the bridges over small streams. These low-cost techniques can be classified into five groups according to their common physical characteristics. However, application of these techniques is site-specific because each stream has it own geomorphologic and hydrologic characteristics. Without proper consideration of these characteristics from a holistic viewpoint, many attempts to address these problems have fallen short of success; some techniques, such as channelization, are now known to greatly upset stream equilibrium, worsening the situation.

The objectives of this study are: (1) to review the various channel migration control techniques in the literature, (2) to select representative sites for which the applicability of the techniques can be studied, (3) recommend application of appropriate techniques for the selected site which could provide low-cost channel stabilization, and (4) recommend monitoring procedures for IDOT to evaluate performance.

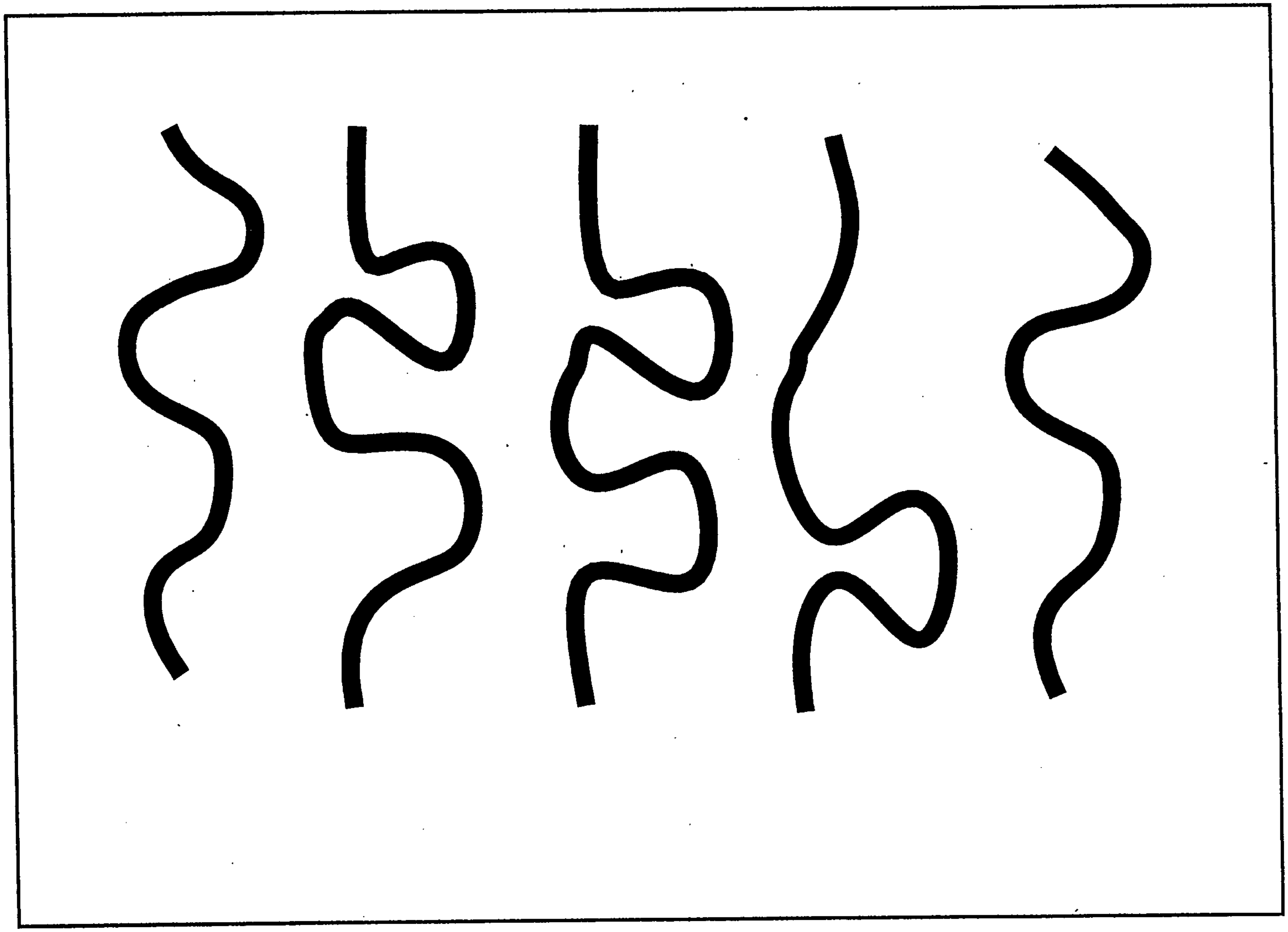

Figure 1-1. Meander migration with time-changing loop geometry.

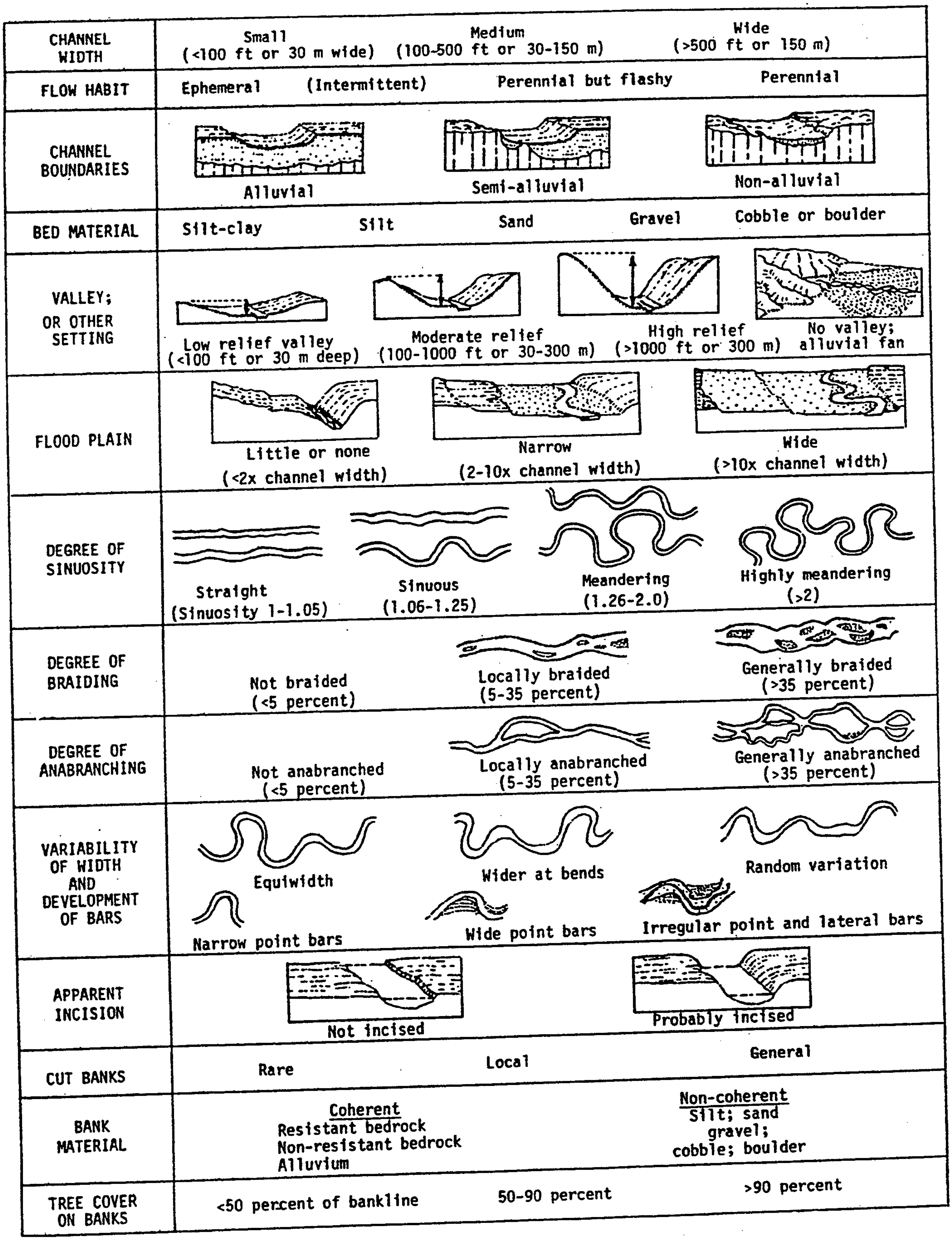

Figure 1-2. Stream properties for classification and stability assessment (Brice et al. 1978)

Table 1-1. Bridge scour and stream instability countermeasures (Lagasse et al. 1997)

Countermeasure Group	Functional Applications: Local Scour – Abutments	Local Scour – Piers	Contraction Scour – Floodplain and Channel	Stream Instability – Vertical	Stream Instability – Lateral	Suitable River Environment: River Type (B = braided, M = meandering, S = straight)	Stream Size (W = wide, M = moderate, S = small)	Bend Radius (L = long, M = moderate, S = short)	Velocity (F = fast, M = moderate, S = slow)	Bed Material (C = coarse bed, S = sand bed, F = fine bed)	Ice/Debris Load (H = high, M = moderate, L = low)	Bank Condition (V = vertical, S = steep, F = flat)	Floodplain (W = wide, M = moderate, N = narrow/none)	Maintenance: Estimated Allocation of Resources (H = high, M = moderate, L = low)	Installation Experience by State	Design Guideline Reference
GROUP 1. HYDRAULIC COUNTERMEASURES																
GROUP 1.A. RIVER TRAINING STRUCTURES																
TRANSVERSE STRUCTURES																
Impermeable spurs (jetties, groins, wing dams)	◗	◗	O	O	●	B, M	W, M	L, M	✓	✓	✓	✓	✓	M - L	Widely Used	1, 4, 5
Permeable spurs (fences, netting)	◗	◗	O	O	●	B, M	W, M	L, M	M, S	S, F	L	✓	✓	H - M	AZ, CA, IA, MS, NE, OK, SD, TX	1, 2, 4
Transverse dikes	O	O	O	O	●	B, M	W, M	✓	✓	✓	✓	✓	✓	M - L	NE	
Bendway weirs/Stream barbs[1]	◗	◗	O	O	●	M	✓	M, S	✓	✓	✓	✓	✓	L	CO, ID, IL, MO, MT, OR, WA	DG1
Hardpoints	O	O	O	O	●	✓	✓	✓	✓	✓	✓	✓	✓	L	CA, ND, NE, OR, SD	1,4, 5, 7
Drop structures (check dams, grade control)	◗	◗	◗	●	O	✓	✓	✓	✓	✓	✓	✓	✓	M	Widely Used	4, 5
Embankment Spurs	◗	O	◗	O	O	✓	✓	✓	✓	✓	✓	✓	W	L	AK, OK	14
LONGITUDINAL STRUCTURES																
Longitudinal dikes (crib/rock toe/embankments)	◗	O	O	O	●	✓	✓	L, M	✓	✓	M, L	✓	✓	M - L	AK, AZ, CA, OK, OR, MS	2, 4, 5
Retards	◗	O	O	O	●	✓	✓	L, M	✓	S, F	L	✓	✓	H - M	Widely Used	2, 4, 5
Bulkheads	●	O	O	O	●	✓	✓	✓	✓	✓	✓	V, S	✓	M	Widely Used	2, 4, 5
Guide banks	●	◗	◗	O	◗	✓	W, M	✓	✓	✓	✓	✓	W, M	M - L	Widely Used	6, 4, 5
AREAL STRUCTURES/TREATMENTS																
Jacks/tetrahedron jetty fields	O	O	O	O	●	B, M	W, M	L	M, S	S, F	M, L	✓	W, M	M	IL, KS, MS, NM, OK, SD, TX	1, 2, 4, 5
Vanes	O	◗	O	O	●	B, M	W, M	L, M	M, S	S, F	L	✓	✓	H - M	IA	7, 16
Channelization	◗	◗	O	O	●	B, M	✓	✓	✓	✓	✓	✓	✓	M	MS, MO, MT, TX	
Flow relief (overflow, relief bridge)	◗	◗	●	O	O	✓	✓	✓	✓	✓	✓	✓	W	M	AZ, MD	
Sediment detention basin	O	O	O	●	O	✓	✓	✓	✓	C, S	✓	✓	✓	H - M	Widely Used	
GROUP 1.B. ARMORING COUNTERMEASURES																
REVETMENTS AND BED ARMOR																
Rigid																
Soil cement	●	◗	◗	◗	●	✓	✓	✓	✓	S, F	✓	S, F	✓	L	AZ, CO, NM, TX	DG2, 4, 5
Concrete pavement	●	◗	●	◗	●	✓	✓	✓	✓	✓	✓	S, F	✓	M	Widely Used	2, 3
Rigid grout filled mattress/concrete fabric mat	●	◗	◗	◗	●	✓	✓	✓	✓	✓	✓	S, F	✓	M	GA, MA, MO, ME, SD, WA	17
Grouted riprap	◗	O	O	O	◗	✓	✓	✓	✓	✓	✓	S, F	✓	M	AZ, CA, CT, ME, MI, TN	2, 3
Flexible/articulating																
Riprap	●	◗	◗	◗	●	✓	✓	✓	✓	✓	✓	S, F	✓	M	Widely Used	2, 3, 5
Self launching riprap (windrow)	O	O	O	O	◗	✓	✓	✓	✓	C,S		✓	✓	H - M	GA, CA, IL, PA	2, 4, 5, 7
Riprap fill-trench	◗	O	O	O	●	✓	✓	✓	✓	✓	✓	✓	✓	M	Widely Used	2, 3, 4, 5
Gabions/gabion mattress[2]	●	◗	◗	◗	●	✓	✓	✓	✓	S, F	M, L	✓	✓	M	Widely Used	2, 3, 5, 7, 20
Wire enclosed riprap mattress (rail bank/sausage)	●	O	O	O	●	✓	✓	✓	M, S	S, F	M, L	S, F	✓	M	AZ, CO, NM	DG3, 8
Articulated concrete blocks (interlocking/cable tied)	●	◗	◗	◗	●	✓	✓	✓	✓	✓	✓	S, F	✓	M - L	AZ, CA, FL, MI, MN, OH, ME, TX	DG4, 2, 3, 10, 11, 12, 19
Articulating grout filled mattress	●	◗	◗	◗	●	✓	✓	✓	M, S	✓	✓	S, F	✓	M - L	AZ, CA, IA, IL, OR	DG5, 17
LOCAL SCOUR ARMORING																
Riprap (fill/apron)	●	◗	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	S, F	✓	H - M	Widely Used	DG8, 5, 9
Grouted riprap	◗	O	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	S, F	✓	H - M	Widely Used	3
Concrete armor units (Toskanes, tetrapods, etc.)[3]	◗	◗	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	S, F	✓	M - L	AZ, PA, NY, VA	DG6, 21, 23
Grout filled bags/sand cement bags	●	◗	N/A	N/A	N/A	✓	✓	✓	✓	✓	M, L	S, F	✓	H - M	Widely Used	DG7, 2, 4, 5, 8, 23
Gabions	●	◗	N/A	N/A	N/A	✓	✓	✓	✓	S, F	M, L	✓	✓	M	FL, OR, TN, WA	
Articulated concrete blocks (cable tied)	●	◗	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	S, F	✓	M - L	AZ, CA, FL, MI, MN, OH, ME, TX	DG4, 13, 23
Sheet pile/cofferdam	◗	◗	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	✓	✓	M - L	CA, CT, FL, NH, WA	2, 3, 5, 7

● well suited/primary use
◗ possible application/secondary use
O unsuitable/rarely used
N/A not applicable

✓ = suitable for the full range of the characteristic

DG# design guideline attached

Table 1-1. (Continued) Bridge scour and stream instability countermeasures (Lagasse et al. 1997)

Countermeasure Group	Countermeasure Characteristics															
	FUNCTIONAL APPLICATIONS					SUITABLE RIVER ENVIRONMENT								MAINTENANCE	INSTALLATION EXPERIENCE BY STATE	DESIGN GUIDELINE REFERENCE
	Local Scour		Contraction Scour	Stream Instability		River Type	Stream Size	Bend Radius	Velocity	Bed Material	Ice/Debris Load	Bank Condition	Floodplain	Estimated Allocation of Resources		
	Abutments	Piers	Floodplain and Channel	Vertical	Lateral	B = braided M = meandering S = straight	W = wide M = moderate S = small	L = long M = moderate S = short	F = fast M = moderate S = slow	C = coarse bed S = sand bed F = fine bed	H = high M = moderate L = low	V = vertical S = steep F = flat	W = wide M = moderate N = narrow/none	H = high M = moderate L = low		
GROUP 2. STRUCTURAL COUNTERMEASURES																
FOUNDATION STRENGTHENING																
Crutch bents/Underpinning	○	●	●	●	◗	✓	✓	✓	✓	✓	✓	✓	✓	L	FL, NC, OR, TX	
Cross bracing	○	●	●	●	○	✓	✓	✓	✓	✓	✓	✓	✓	L	NC, FL, LA	
Continuous spans	○	●	●	●	○	✓	✓	✓	✓	✓	✓	✓	✓	L	NC	
Pumped concrete/grout under footing	●	●	◗	◗	◗	✓	✓	✓	✓	✓	✓	✓	✓	M	GA, MO, ME, MN, NC, NY, OR	
Lower foundation	●	●	●	●	●	✓	✓	✓	✓	✓	✓	✓	✓	L	CA, OR, TX	
PIER GEOMETRY MODIFICATION																
Extended footings	N/A	●	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	✓	✓	L	AK, AZ, NY, PA, TN, TX, WA	23
Pier shape modifications	N/A	●	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	✓	✓	M	FL	
Debris deflectors	N/A	●	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	✓	✓	H - M	CA, FL, NM, OR	
Sacrificial piles/dolphins	N/A	●	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	✓	✓	H - M		22, 24
GROUP 3. MONITORING																
FIXED INSTRUMENTATION																
Sonar scour monitor	◗	●	●	●	◗	✓	✓	✓	✓	✓	L	✓	✓	M	CO, FL, IN, NY, VA, TX	15, 18
Magnetic sliding collar	●	●	●	●	◗	✓	✓	✓	✓	S, F	✓	✓	✓	M	CO, FL, IN, MI, MN, NM, NY, TX	15, 18
Sounding rods	◗	●	●	●	◗	✓	✓	✓	M, S	C	M, L	✓	✓	H	AR, IA, NY	15, 18
PORTABLE INSTRUMENTATION																
Physical probes	●	●	●	●	●	✓	✓	✓	M, S	✓	M, L	✓	✓	L	Widely Used	15, 18
Sonar probes	●	●	●	●	●	✓	✓	✓	M, S	✓	L	✓	✓	L	Widely Used	15, 18
VISUAL MONITORING																
Periodic Inspection	●	●	●	●	●	✓	✓	✓	✓	✓	M, L	✓	✓	H	Widely Used	15, 18
Flood watch	●	●	●	●	●	✓	✓	✓	✓	✓	M, L	✓	✓	H	Widely Used	15, 18

● well suited/primary use
◗ possible application/secondary use
○ unsuitable/rarely used
N/A not applicable

✓ = suitable for the full range of the characteristic

DG# design guideline attached

NOTES:
1. There is limited but successful field experience using bendway weirs/stream barbs as stream instability countermeasures.
2. Performance of welded versus twisted wire and PVC coated versus uncoated wire gabions is not distinguished in the matrix.
3. There is limited field experience using concrete armor units as protection for bridge piers.

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Table 1-2. Flow and geomorphic factors in bridge and countermeasure design (Brice et al. 1978)

Flow factor	Purpose of data to define flow factor	Relation of flow factor to design (with site numbers of selected case history examples)
1. Water level or stage	Document water levels for existing channel conditions. Relate water level to discharge. Determine backwater (Bradley, 1973; Blodgett and Stiehr, 1974).	Clearance requirements of bridge (8,9,25,69,71,99,250). Height of bridge, approach embankment and countermeasures (9,18,41,49,99,201,267). Size of bridge to prevent excessive backwater caused by bridge constriction (99). Type of bridge structure and approach embankment if bridge submergence or approach overtopping is expected (11,18,19, 22,34,132,201).
2. Discharge	Determine magnitude of flood events (Blodgett and Stiehr, 1974). Evaluate distribution of flows in channel and flood plain (Bradley, 1973; Blodgett and Stiehr, 1974). Determine relation of discharge to water level.	Design flood discharge (8,9). Design flood can be compared to historical events on the basis of flow quantity (8,9,11,46,48,54,101,250). Bridge size and waterway requirements (4,7,8,9,12,16,20,28,29, 30,31,35,45,46,48,49,61). Need for overflow bridges or road overflow (5,22,41,46,100). Bridge height (by use of discharge-water level relation)(69).
3. Recurrence interval (flood frequency)	Determine recurrence interval of design flood (Water Resources Council, 1976; Waananen and Crippen, 1977). Determine recurrence interval of other flood events (Blodgett and Stiehr, 1974).	Flood discharge and related stage for subsequent use in bridge design (8,9,99,260). Enable relation of design flood to historical event on basis of interval of time (237,272). Enable relation of design and historic floods to meet legislative requirements.
4. Duration of flooding	Estimate length of time water surface is above a base level (Blodgett and Stiehr, 1974).	Requirements to prevent excess pore-water pressure (134, 167, 217). Height of bridge, approach embankment, countermeasures. (Inundation for short periods of time acceptable; long periods unacceptable)(18,19,22,100,223).
5. Velocity	Determine Froude number, possibility of scour.	Bridge size by comparing existing flow velocities to anticipated values at bridge constriction (53). Bridge location and countermeasures for unstable channel boundary conditions (8,28,51). Bridge size and pier type to prevent scour caused by bridge constriction (4,11,23,26,28,45,53,61,154,155).
6. Floating debris	Estimate size of debris (Blodgett and Stiehr, 1974). Estimate source (anchor ice, logs, trees) (Neill, 1973).	Clearance requirements of bridge (6,8,9,14,165). Pier shape and footing type (1,8,60,115,132,168,258,268). Bridge size and spans to prevent debris blocking bridge waterway (27,58,70,88,89,155,157,168,262,266).

Table 1-2. (Continued) Flow and geomorphic factors in bridge and countermeasure design (Brice et al. 1978)

Flow factor	Purpose of data to define flow factor	Relation of flow factor to design (with site numbers of selected case history examples)
7. Source of flow	Evaluate changes in distribution of flow at bridge site (Blodgett and Stiehr, 1974). Evaluate changes in flow patterns associated with artificial inflow, such as storm drain outfall, dam failure, or new dams. Determine changes in sediment discharge rates (Porterfield, Busch, and Waananen, 1978).	Height of bridge, approach embankment, and countermeasures (41). Size and location of main channel and overflow bridges (57, 138,204). Countermeasures to prevent scour, degradation, and lateral erosion (27,39,160,167,173,184,238,267). Type of pier and abutment footing.
8. Channel geometry	Determine channel area, wetted perimeter, conveyance, distribution and depth of flow, velocity, Froude number, boundary shear stress (Chow, 1959). Document existing channel size and shape.	Location and size of main channel and overflow bridges (57, 68,218). Part of channel conveying flow (46,48,151,153). Length of piers and location (1,8,11). Size, shape, and alinement of proposed channel based on existing channel size (7,9,13,20,45,53,91,92,99,101,220, 240,243).
9. Channel slope	Determine channel capacity. Determine boundary shear stress (Nece, 1974, and Volume II of this report). Determine existing channel gradient (Blodgett and Stiehr, 1974).	Size of bridge waterway to maintain channel capacity for design discharge (8,13,57,99,262). Countermeasures to prevent degradation, scour, and lateral erosion (1,8,13,39,45,48,85,89,124,125,134,148,174,277,282).
10. Presence of existing features	Evaluate effect on channel capacity. Evaluate effect on flow alinement and distribution. Estimate effect on channel sediment transport rates.	Length and location of bridge (features upstream or downstream from site may affect capacity of channel)(37, 39,40,41,42,44,50,52,55,56,85,146,254). Need for countermeasures to maintain desired flow alinement or capacity (42,44,50,55,99,125,194,196,261). Need for countermeasures to prevent scour (39,52,56,177,197, 198,201).
11. Suspended sediment and bed material	Estimate availability of bed material for transport (Normann, 1975). Estimate roughness coefficient in computation of channel capacity (Mannings n)(Barnes, 1967).	Size of bridge required to convey design discharge or prevent damage by aggradation (57,71). Type of bridge pier and abutment footings (8,11,31,32,44,51,73). Type and location of piers to prevent abrasion and impact damage (1,3,8,24,60,74,127,132,140). Countermeasures to stabilize channel boundaries (1,7,8,17, 21,23,39,126,164).
12. Channel alinement	Document existing channel alinement (Brice, 1977).	Bridge and approach location relative to channel and flood plain (7,14,101,157,163,177,182,216,234). Countermeasures to stabilize channel location (7,13,21,33,36, 43,45,63,85,92,95,99,116,120,126,130,131,133,169,170,180, 192,211,225,226,233). Pier shape and location (7,38,99,126,228,263). Type of pier and abutment footings (11,51).

2. Literature Review

Introduction

Existing stream bank protection literature is extensive, encompassing soil mechanics, hydraulics, hydrology, agriculture, and geomorphology. The literature is in the form of government agency reports, university studies, pamphlets, design sketches, journals, papers, and books; the scope of the works ranges from general "how to" stream bank protection manuals to detailed construction plans for stream bank protection projects.

As part of the Stream Bank Erosion Control Evaluation and Demonstration Act of 1974, the U.S. Army Corps of Engineers (1981) published the results of a study which examined the extent of stream bank erosion on navigable rivers, new stream bank protection techniques, and the causes of stream bank erosion. The study also included bank protection demonstration projects on larger rivers where bank erosion was problematic.

The resulting literature survey (U.S. Army Corps of Engineers, published 1981, conducted 1974-76) provided a preliminary investigation of the mechanics of stream bank erosion and a comprehensive evaluation of existing stream bank protection methods in use prior to its publication. The stream bank protection methods examined were traditional approaches such as stone riprap revetment, concrete mattresses, kellner jack fields, gabions, and vegetation; such methods are usually not cost-effective on smaller rivers and streams. The report detailed the advantages and disadvantages of these and other stream bank protection methods, providing photographs and diagrams of stream bank protection designs. Most of the bank protection methods examined in that study (tire revetments, jack fields, and paved banks, for example) have since become obsolete as modern stream management practices seek to become more environmentally friendly and aesthetic. However, where relevant, information from this literature search has been included in the related sections of this report.

A U.S. Army Corps of Engineers booklet (Keown, 1983), resulting from the same study and intended for landowners and government agencies, provided a useful summary of stream bank migration mechanics and possible solutions to these problems. The bank protection methods described in this pamphlet are primarily for larger rivers, but the context and causes of stream bank erosion problems are presented effectively.

Under the support of the Federal Highway Administrationm a large amount of works has been done on channel migration and bank erosion for the purpose of bridge protection. These studies have been reported in the references quoted in Table 1-1, most noticeably FHNA HEC-11, 18, 20 and 23.

Hemphill and Bramley (1989) provide a more modern approach to stream bank stabilization. The book outlines a recommended design procedure of bank protection works and discusses the relative merits of various bank protection methods. The stream bank protection methods discussed in this book, ranging from riprap revetments to vegetation, reflect the state of modern stream bank protection work in Europe. A comparable authoritative reference describing modern, more "environmental" stream bank protection methods in the United States was not found.

The following literature review concentrates on the five groups of techniques that can be regarded as realatively low cost and to some degree applicable to relatively small streams for mitigation of channel and bank protection.

Riprap Revetment

Riprap cover, the most traditional method of stream bank protection, has been extensively documented. Standard use of riprap in the United States is effectively described and summarized in the United States Army Corps of Engineers report (1981). Some advantages and disadvantages of riprap are highlighted in Table 2-1. A traditional riprap blanket design requires bank shaping to a uniform , gradually sloping bank. Then, a permeable filter (usually gravel) is placed in order to permit seepage but prevent erosion of bank soil. Finally, a riprap blanket supplies the top layer of protection. The report cites the following points as being essential to a successful riprap blanket design: (a) the shape, size, and weight of stone which suit the hydraulic conditions; (b) a bed filter which prevents the erosion of bank material through the riprap; (c) the correct riprap blanket and gravel filter thickness; and (d) stabilization of the bank toe with trench fill or peaked stone fill.

Guidelines by Gray and Leiser (1982) summarize important points in riprap stone selection, giving specifications for stone size based on the average stream velocity and bank slope; Jansen et al. (1979) gives a similar design relation based on the Shields diagram for sediment transport. The U.S. Army Corps of Engineers (1983) gives more general design criteria for stone size, recommending a well-graded mix of stones weighing from 20 to 200 lbs for streams with maximum velocities less than 10 ft/sec. The (1981) U.S. Army Corps of Engineers literature summary recommends blocked stones and discourages the use of stones with length-to-width ratios greater than 3. Grading specifications for riprap can be found in Hemphill and Bramley (1989), and all sources agree that a well-graded mixture is desirable. In Illinois, the most common riprap grading used in stream bank protection is RR-5 gradation (See Appendix I).

The U.S. Army Corps of Engineers (1981) recommends a maximum uniform bank slope of 1V:2H for dumped stone and 1V:1.5H for hand-placed stone; Hemphill and Bradley (1989) give the angle of repose of riprap as being between 35º and 42º. The method of toe stabilization used traditionally is a key trench or riprap apron (U.S. Army Corps of Engineers, 1983). The method of "longitudinal peaked stone toe protection" has been successful in providing toe protection to streams with degrading beds. The riprap blanket should be keyed in to the stream bank at its upstream and downstream ends to prevent the riprap blanket from being unraveled at its ends. The recommended thickness of the riprap blanket is 1-1.5 times the diameter of the largest-sized stone in the riprap (usual thickness is 12-18"). The blanket is usually designed to extend to the high-water elevation. Figure 2-1 illustrates traditional riprap stream bank protection designs.

Design of a riprap bed filter is not documented in the U.S. Army Corps of Engineers literature. Gray and Leiser (1982) suggest the following criteria for the selection of the filter material: D_{15}(of gravel)<$5D_{85}$(of bank) and the ratio of D_{15}(of gravel) to D_{15}(of bank) ratio should be between 5 and 40. Hemphill and Bradley (1989) relate the recommended filter material to the bed material and discuss the use of more expensive geotextile fabrics as filter material in place of gravel or sand. The thickness of the granular filter is generally under six inches. However, the use of a filter is only warranted for fine bank material likely to be lost through the pores in the riprap blanket; if vegetation becomes established among the riprap, then this may be sufficient to halt erosion of the bank material.

Table 2-1. Advantages and disadvantage of riprap revetment.

Advantages	*Disadvantages*
• easily designed and constructed; requires no specialized labor, equipment, or design experience	• aesthetics: unsightly and not natural-looking
• effective and durable; resistant to ice flows.	• only cost-effective if rock is nearby and can be transported cheaply
• low maintenance requirements and easily repaired	• does not enhance wildlife habitat or promote bank vegetation
• can permit vegetation in time	

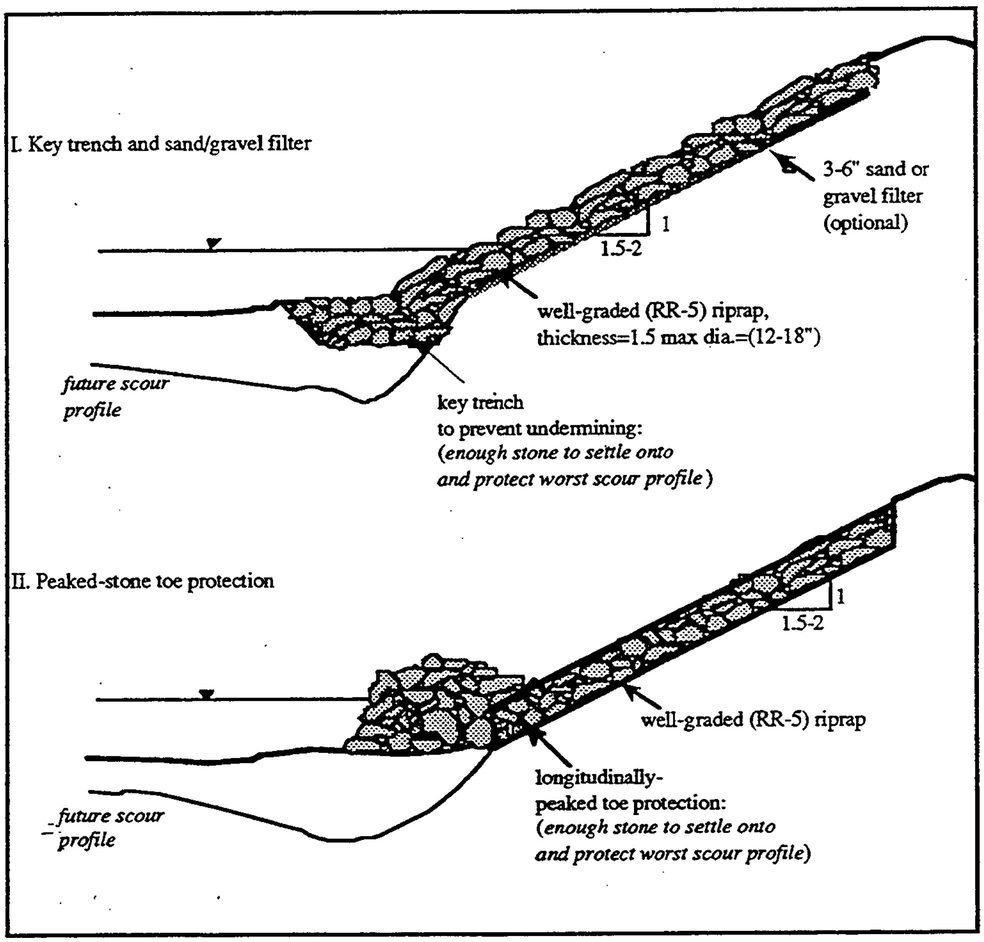

Figure 2-1. Traditional riprap stream bank protection design. (After: U.S. Army Corp of Engineers, 1983)

Bendway Weirs and Other In-stream Structures

In-stream structures have been used widely in stream bank protection and rehabilitation. Weirs, vanes, and dikes have been used in stream bends to realign flow and encourage sediment deposition behind the structures. These structures, made of riprap, timber piles, or trees, can counteract the scouring secondary currents in stream bends, removing the eroding pressure of the flow from the bank. Although many of these structures have been used on larger streams and rivers, recently several stream bank protection projects in Illinois have used rock weirs ("bendway" weirs) on smaller streams. Design of these smaller structures has been thus far based on experience. Table 2-2 summarizes some advantages and disadvantages of bedway wiers.

Early stream bank protection trials by the U.S. Soil Conservation Service (SCS) (1949) on the Winooski River in Vermont used stone-filled log-crib jetties ("wing cribs") and permeable pile jetties, as flow deflectors. The structures were angled at 45° downstream and were submerged during high flow. Both types of structures were not successful when submerged, encouraging local scour behind the structures. Recommendations were made for the construction of the cribs and jetties, and it was suggested that the structures would be insufficient protection on sharp stream bends.

The development of the "Iowa Vane" system by the University of Iowa produced some good theoretical and experimental work (Odgaard and Kennedy, 1982; 1984) on the dynamics of in-stream vanes placed to counteract secondary currents in stream and river bends. Earlier physical analysis and theory of the flow deflectors, developed by Ascanio and Kennedy (1983), and Zimmerman and Kennedy (1978), and Odgaard and Kennedy (1982, 1984) give analytical expressions for vane design (width, spacing, number, angle) based on the stream bend characteristics (velocity profile, transverse slope, roughness). Laboratory and prototype (Sacramento River) experiments showed that the vanes were effective in reducing near-bank velocities and bank scour while promoting bank development through deposition.

Further submerged vane work by Odgaard and Lee (1984, 1987) and Odgaard and Wang (1991a) refined the design process after more laboratory and theoretical work. They designed a vane system to halt secondary scour on the East Nishnabotna River near a highway bridge in Iowa. The performance of the installed vane system (vertical sheet piles) is described by Odgaard and Mosconi (1987), Odgaard and Lee (1987), and Odgaard and Wang (1991b). The installed vane system was shown to be effective in redistributing sediment across the stream to the outer portion of the bend and moving the highest velocities back to the middle of the channel, without changing the river slope or roughness. However, after installation, an approaching meander just upstream of the installed vanes had changed the alignment of the entering flow, changing the attack angles of the installed vanes. The effects of this change have not been documented. Odgaard and Wang (1991b) describe another successful use of Iowa vanes on the West Fork Cedar River.

Izumi et al. (1991) provided results of numerical and laboratory experiments on the effects of distributed drag techniques on straight channel bank erosion. The experiments used permeable fences to promote sediment deposition and to lower near-bank velocities. The results indicated that permeable dikes can be effective in preventing bank erosion, and that this effectiveness was proportional to the density of the permeable structures.

The use of bendway weirs in smaller rivers and streams has been described in several project reports. Derrick and Kinney (1995) give the project specifics of bank stabilization work on the Wood River Creek in Madison County, Illinois. The project used five stone weirs to protect about 500 ft of stream bank along two bends near a highway bridge. Riprap toe protection was installed between the weirs, and the project is being monitored. Figure 2-2 depicts the Wood River Creek design.

Derrick (1995b) describes the "theory" and application of bendway weirs in both navigable rivers and in smaller rivers and streams. The informal report also outlines work done on Harland Creek in Mississippi to protect fourteen stream bends with bendway weirs. Several design points are stressed: the need to correctly design the upstream weir to align the flow toward the remaining weirs, the need to design all of the weirs for both low flow and high flow conditions, and the frequent need for toe protection in conjunction with the bendway weirs. Another stream bank stabilization project used tree trunks and hand-placed stone as low bendway weirs on the East Fork of Wood River Creek. Five trees were placed as weirs on one bend by anchoring the trees to the stream bed with various materials. Stone was placed manually for weirs on another bend, and both projects are being monitored. Figure 2-3 provides a schematic representation of the Harland Creek Demonstration project.

The Harland Creek Bank Stabilization Project has been further described in Derrick (1995c), which includes design plans of the project. The project spanned a stream reach of 11,700 ft, and weirs were spaced at intervals ranging from 75-100 ft. Longitudinal peaked stone toe protection was also used in many of the bends. Problems encountered with the project included scour between the weirs. The bends, however, appear to be mostly stable and native vegetation is becoming established.

Bendway weir use on larger rivers has also been described by Derrick (1995a). Eleven weirs were used on the Big Blue River in Kansas to stabilize an active meander upstream of a highway bridge. Willow posts were also planted, and this project is being monitored.

Table 2-2. Advantages and disadvantages of bendway weir protection.

Advantages	*Disadvantages*
• effective in halting secondary scour when properly designed	• Incorrect design can cause adverse realignment of stream
• promotes fish habitat and sedimentation behind weirs	• As stream alignment changes with time, weirs must be adjusted
• local materials can be used as weir materials (timber, trees, riprap)	• riprap stone protection may also be required (cost)
• more aesthetic than traditional riprap	• local scour can occur between weirs
• weirs function well at high-flow conditions	
• allows native vegetation to establish	

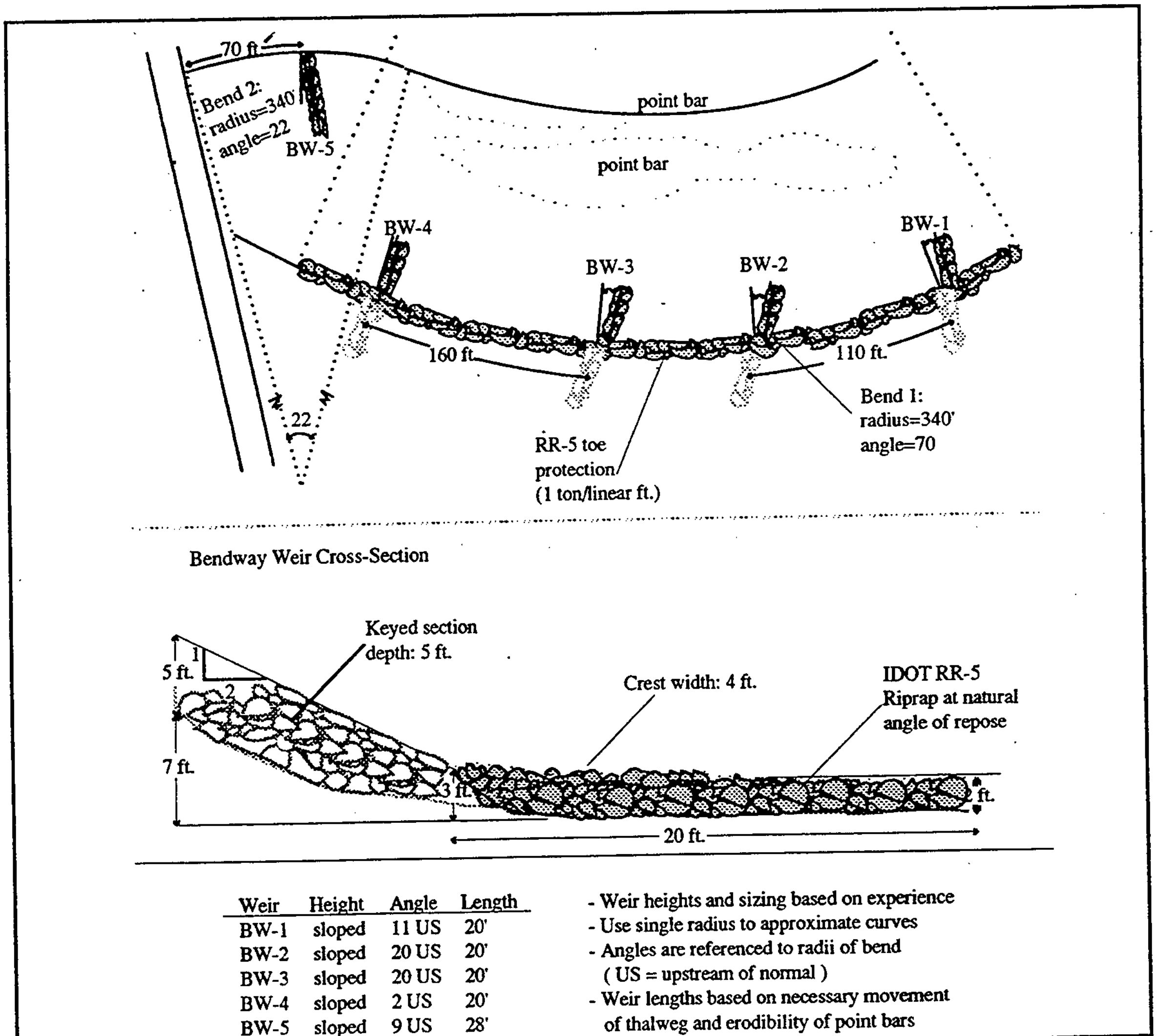

Figure 2-2. Bendway Weir design example - Wood River Creek design. (After: Derrick and Kinney, 1995)

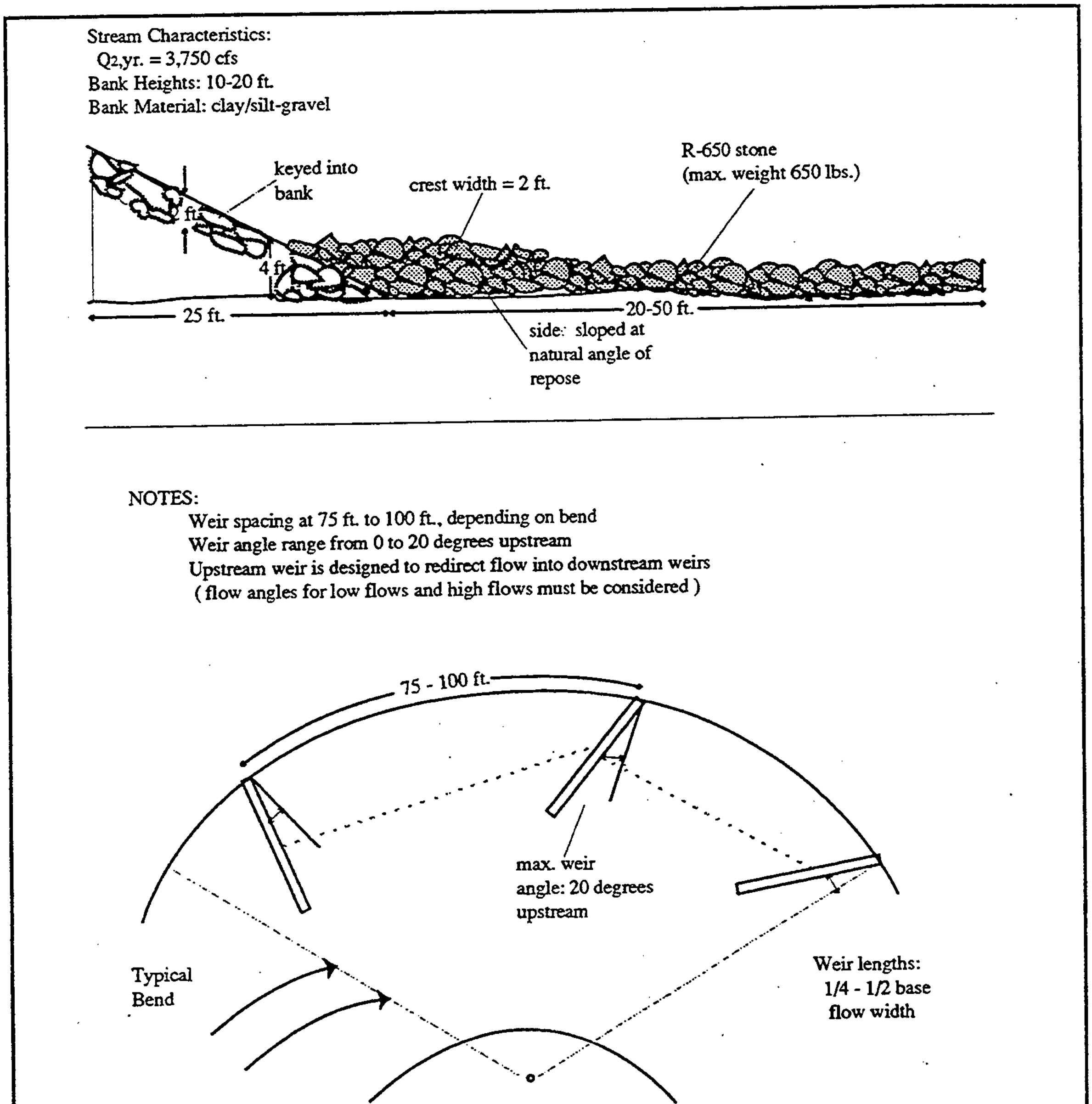

Figure 2-3. Bendway weir design example- Harland Creek design. (After: Derrick, 1995)

Willow Posts

Planting dormant willow posts along eroding stream banks has become a popular technique of stream bank stabilization in Illinois. The benefits of using vegetation as stream bank protection measures are well cited; however, practical design information on vegetative stream bank stabilization has not been well documented in North America until the last decade. Table 2-3 presents some advantages and disadvantages of willow post protection.

The U.S. Soil Conservation Service (SCS) (1949) conducted some of the first experiments using vegetative stream bank protection on the Winooski River in Vermont; many of the same bank protection methods tried then are being used today in Illinois. The SCS worked on undercutting stream banks by treating them with combinations of bank shaping, riprap toe protection, and vegetative bank cover.

The experiments determined that purple-osier willows were successful plants for bank erosion protection, but that without toe protection, vegetation alone was rarely successful in stabilizing a stream bank. The purple-osier willow was noted for its flexibility and resiliency in absorbing the impacts of high flows and ice flows. The SCS noted also that willow cuttings produced a better bank cover than willow poles, and recommended planting the cuttings with a mulch to protect the bank soil while the willows were establishing. Red-osier dogwood also proved successful in surviving adverse conditions, and worked well at the bottom of the bank.

Porter and Silverberger (1960) described a decade of experiments on Buffalo Creek in northwestern New York that tested various grasses, shrubs, and trees for effectiveness as stream bank stabilizers. Porter concluded that, when established, willows and certain grasses could be used effectively as erosion protection measures. The experiments showed that dormant willow cuttings survived better when planted in the spring. Porter et al. discouraged the use of vegetation as stream bank toe protection, indicating that the erosive forces in this region are too strong to allow for plant establishment.

An early work by Seibert (1968) in Germany noted the benefits of using vegetation and provided some simple design criteria for planting both woody and herbaceous species on stream banks, including willows. Schiechtl's (1980) book seems to be the first practical design document encouraging engineers to use vegetation in stream bank protection. He suggested the use of dormant cuttings on stream banks and provides some general planting guidelines.

Bowie's (1982) experiments in northern Mississippi seem to have formed the basis for much of what now is termed the "Willow Post Method". Bowie investigated the comparative effectiveness of various combinations of vegetation, bank shaping, and structure on actively eroding alluvial stream banks. The study concluded that native plant species, specifically willows, were more hardy (hence, effective) than introduced species. Control of bed degradation was necessary to prevent bank failure due to bank toe undercutting.

Henderson's (1986) general work on environmental stream bank protection designs suggested using structure in conjunction with vegetation without giving any design criteria. Hemphill and Bramley(1989) provided useful, mostly conceptual design information on different ways of using vegetation, including willows, as stream bank protection. The designs are based on stream bank protection experience in the United Kingdom.

An interesting project in Virginia described by Kohnkeand Boller(1989) used "live soft gabions" and "live cribwalls" placed in the lower stream bank (toe) as a technique to establish dormant willows. "Live soft gabions" are described as basket-like structures in which successive layers of soil and plants are held in place by a geogrid material or filter fabric. A "live cribwall" is a box-like timber crib filled with rock, earth, and plantings. The advantage of using such structures to establish vegetation appears to be the additional strength provided by the basket or crib in which the plants are rooted.

Slowikowski et al. (1992) conducted trials with willow posts and other woody vegetation on Richland Creek in Illinois, comparing the relative effectiveness of different sized woody plants in bank stabilization. The most successful site utilized dormant willow and cottonwood posts, 3-6 inches in diameter, and tree revetments as toe protection. Short (2-3ft) willow stakes installed at another site died due to burying by sediment and drought; a third site planted with thin (dia<1in) woody species fared poorly due to a drought. A final site treated with bank shaping and grass was also ruined during a severe drought.

Shields (1991) investigated the effects of naturally established vegetation on the stability of riprap bank protection along the Sacramento River in northern California. Revetments with natural woody plants proved more effective in bank erosion control than unvegetated revetments. Shields concluded that the effects of natural vegetation on bank stability and channel conveyance is site specific, depending on the type and density of vegetation, bank geometry, and hydraulic conditions.

Shields et al. (1995) conducted a study similar to Bowie's (1982) investigations, comparing the effectiveness of vegetation, toe protection and bank shaping as stream bank protection measures. Heavily incised alluvial streams in northwest Mississippi were treated and monitored for 10-13 years. Eleven sites were treated with three different vegetative configurations: vegetation alone, vegetation with toe protection, and vegetation with toe protection and bank shaping. The sites were monitored by conducting repeated cross-section surveys and collecting hydrologic data.

The five vegetation-alone sites were planted with various species of willows placed both manually and mechanically. Poor short-term survival was attributed to bank erosion, covering by sediment, poor soils, and competition from other vegetation. Willows planted after leaf buds began to swell fared poorly. Long-term bank stability was achieved in reaches where the channel bed had stabilized and ceased to degrade. After ten years, native woody species had overtaken the stable banks, regardless of whether the banks had originally been planted or unplanted.

Three other sites were treated with woody plants (willow and water elm) and structural toe protection in the form of longitudinal stone windrows. Although these three sites were subjected to severe droughts and floods, the vegetation and toe protection achieved bank stability and supported native vegetation over the 10-13 year study period. Shields, Bowie, and Cooper (1995) hypothesized that, following bank failure, the wasted soil had been detained up-bank of the toe protection, eventually supporting vegetation. Again, banks became naturally vegetated following stabilization.

The reaches treated with vegetation (willow and grass), toe protection (stone windrows), and bank shaping saw varied success, with no single factor contributing to failure or success. One site failed as cellular blocks were undercut during a season of major (1m) bed degradation. Two grasses, Alamo switchgrass and sericea lespedeza, proved hardy enough to withstand the northern Mississippi stream flows and climate. Although the degree of success varied at the sites, all banks stabilized over the study time period. However, the cost of using vegetation, bank shaping, and toe protection together seemed unjustified in light of commensurate success achieved with a partial treatment.

Shields et al. (1995) concluded that bed stabilization was essential for bank stabilization in incising channels. Planted vegetation proved successful in protecting banks against erosion, and native plant species (particularly willows) seem to fare well in stream bank environments.

Sotir (1995) documented the success of stream bank stabilization and stream restoration with vegetation and bank shaping in several North American streams (including Crow Creek in Illinois), but provided no practical details on the planted species or procedures.

White detailed the Illinois Department of Conservation use of willow posts as stream bank stabilizers on Court Creek, Illinois, in a series of informal reports. Dormant willow posts were planted in holes bored by metal rams or power augers. A row of willows was placed directly instream as an attempt to protect other growing willows. White cited the need to place the willow posts deep enough to ensure that the willow roots would remain below the water table during the first year of growth.

The United States Department of Agriculture/ Soil Conservation Service's (1990) technical note provides explicit design guidelines for the use of willow cuttings and posts on streams with eroding stream banks but stable beds. The document summarizes plant handling procedures, necessary site evaluation, plant selection, planting procedures, and provides a design worksheet. This document is provided in Appendix II.

A 1995 project description by Derrick and Kinney (1995) gives a useful summary of some guidelines in dormant willow post planting. The report describes a stream bank stabilization project on Wood River Creek, located in Madison County, Illinois, and recommends willow post planting in addition to the use of bendway weirs at the site. A schematic representation of the Wood River Creek willow post design can be found in Figure 2-4. Derrick's (1995) report on the bendway weir and willow post stabilization on the Big Blue River in Kansas gives more guidelines on willow post planting and selection. The willow post selection and planting guidelines presented in these reports have been summarized in the Appendix II.

Another Madison County, Illinois stream bank stabilization project on Cahokia Creek has been summarized by Kinney (1995). Willow posts were used with stone toe protection and bank shaping to stabilize a steep bank experiencing toe undercutting. On-site riprap was placed around the willows following planting, and this seems to have increased willow survival. The Cahokia Creek design is presented in Figure 2-5.

Preliminary results from a large demonstration project in Mississippi on Harland Creek are given by Derrick (1995c). The willow post planting guidelines established by Roseboom (Illinois State Water Survey) were used to plant nearly 10,000 willows along an 11,000 ft reach of Harland Creek, a deeply incised stream. Initially, the willow survival rate at the site was less than expected; Derrick speculates that improper backfilling of the auger holes and flooding may have been the cause. Some willows thought to be dead later had live shoots growing and appeared well.

Table 2-3. Advantages and disadvantages of willow post protection.

Advantages	*Disadvantages*
• improved aesthetics over traditional structural designs	• need for specialized labor or very explicit planting instructions
• increases wildlife habitat; encourages vegetation of banks by native species	• difficulty in establishing vegetation; willow survival not guaranteed
• method is generally less costly than structural stabilization techniques	• woody vegetation alone will not control toe undercutting (a major cause of stream bank instability)
• plant roots impart additional strength to soil	• inability of certain species to tolerate extended inundation and/or drought
• woody vegetation filter non-point source pollutants from groundwater inflow	• reduction of channel conveyance
• method has been well documented; design has been refined through field trials	

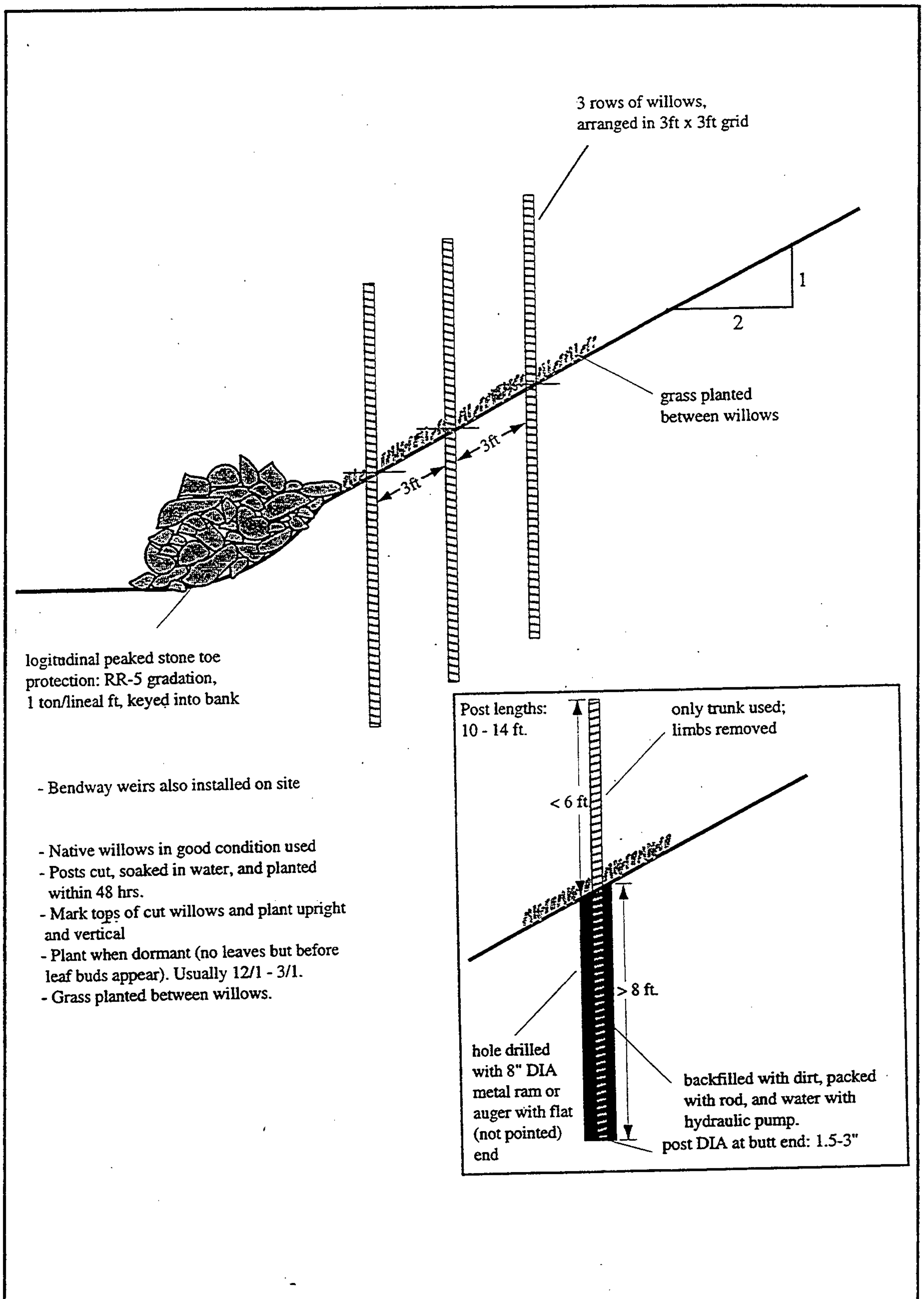

Figure 2-4. Willow post design example - Wood River Creek, Madison Co., Illinois. (After: Derrick and Kinney, 1995)

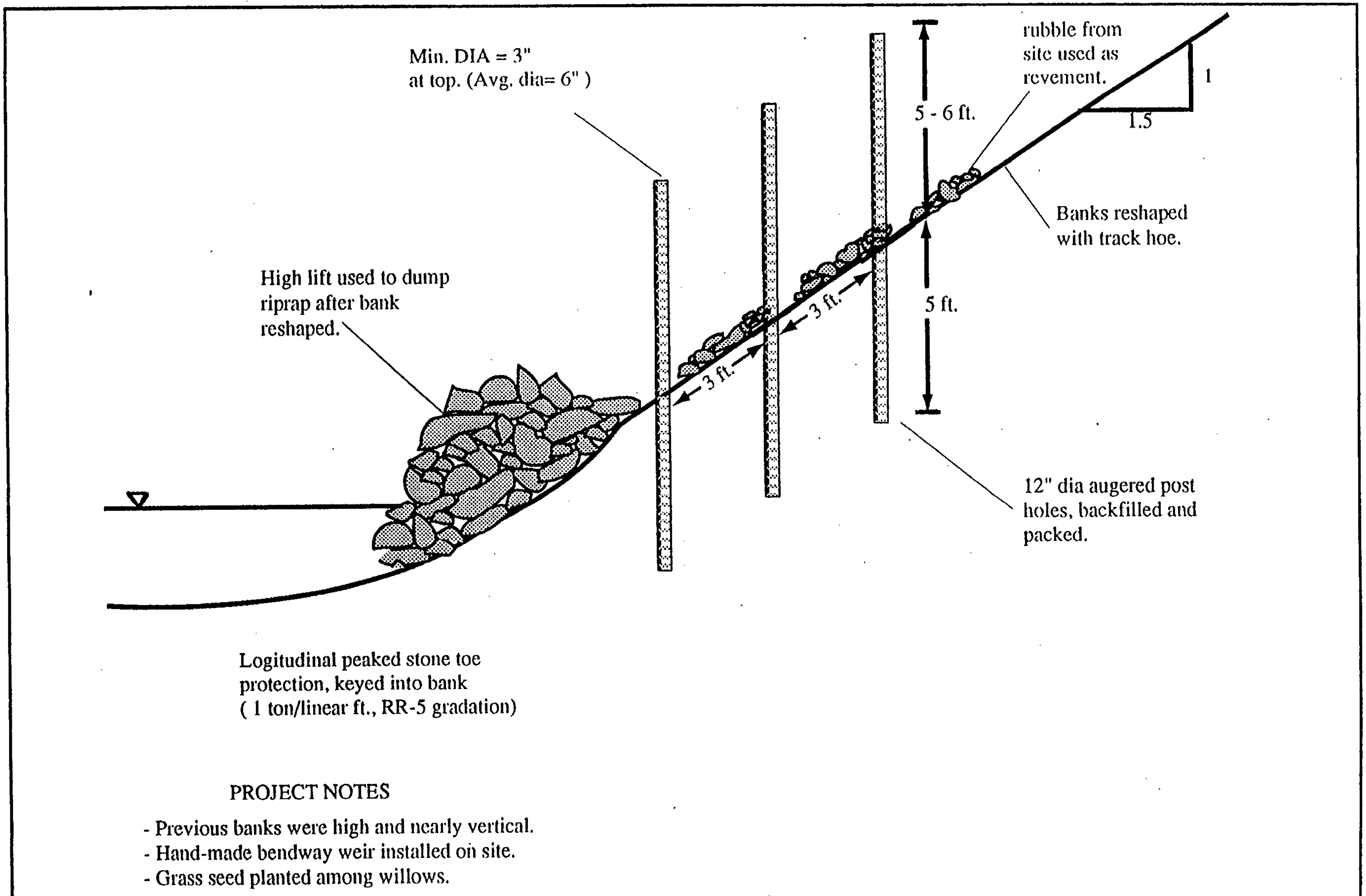

Figure 2-5. Willow post design example - Cahokia Creek, Madison Co., Illinois. (After: Kinney, 1995)

Bioengineering Methods

The term "soil bioengineering" has come to define the use of biological and engineering concepts in the design and management of soil erosion control works. The concept has been widely applied in Europe toward lake shore and stream bank stabilization projects and is now becoming popular in the United States with the growing interest in channel restoration. Examples of soil bioengineering projects include the use of willow posts, tree revetments, live gabions, fascines (bundles of branch cuttings), and a wide range of construction materials manufactured from plant matter, including coconut fiber mattresses, "bio-log" revetments, and geotextiles. Designs are usually site-specific and incorporate a combination of bioengineering concepts regarding plant selection and use. An exception to this lack of repetition in bioengineering design in Illinois has been the use of willow posts in stream bank protection. This specific design has been extensively documented and refined, making the method a viable alternative for larger (government) agencies. However, the site-specific nature of bioengineering has thus far mostly reserved bioengineering to the consulting industry and specialized pockets of government agencies. The lack of ability to produce repeatable bioengineering stream bank protection "recipes" has made this method of bank protection difficult to adapt to large agencies which use repeatable, standardized designs. Table 2-4 presents some of the advantages and disadvantages of bioengineering methods.

Schiechtl's (1980) work, considered an authority in bioengineering at that time, gives many examples of bioengineering methods applied to slope protection, and more specifically, to stream bank protection. Other more recent sources containing information on plant selection, handling, and application are Bache and MacAskill (1984), Gray and Leiser (1982), and Hartmann and Kester (1983). Gray and Leiser (1982) effectively describe the necessary role of structural enhancement (toe protection, "cribs," geo-reinforcement, etc.) in addition to vegetation in bioengineering. Project reports also contain important information on planting successes and failures; these reports are often most informative due to the regional nature of bioengineering techniques. Bioengineering techniques in stream bank protection can be categorized in the following manner:

Live Stakes: Planting live stakes in stream bank protection works can be used as stakes to secure other "live" mattresses and wattles, to add aesthetic value to a project, or to secure revetments such as trees and "biologs". "Biolog" is a general term used to describe any rolled, biodegradable, vegetative material. Often, biologs are composed of coconut fibers encased in a coconut fiber netting. The use of willow stakes is an example of live stakes in stream bank protection projects; these stakes are usually planted when dormant. In addition to willow species, dogwood, baccharis, alder, and elm cuttings have all been used in stream bank stabilization projects. Native species found thriving on site are the best candidates for use as cuttings. Hartmann and Kester (1983) and Gray and Leiser (1982) provided specifics on cutting selection and handling. Much information regarding willow selection and planting can be found in the references cited in the willow post method description in this report. The cuttings are always cut and planted when dormant (after leaves have fallen but before buds have formed), which usually occurs in late winter and spring. Once cut, the stakes should be kept moist (preferably out of the sun), and the top ends should be marked to ensure correct vertical orientation when planted.

The placement and use of cuttings/stakes as stream bank protection varies. Stakes can be planted in the bank in a grid pattern as in the willow post method; alternate uses of live stakes is in securing wattles and similar bundles to the bank. Hemphill and Bramley (1989) provide other uses of live stakes, including close-driven willow poles as toe protection and stakes for securing fascines. Schiechtl (1980) gives ideas on stake use, as do Gray and Leiser (1982).

Brush Layering: The placing of brush layers, ends protruding from the bank, in terraces excavated in the stream bank, can be effective in promoting vegetation and increasing the geotechnical stability of a stream bank. These brush layers eventually develop into woody plants whose roots lend additional strength to the stream bank. This method is best suited also for stream banks experiencing major overbank erosion and gully erosion. Gray and Leiser (1982) describe applications of brush layering in gully and bank protection. The brush layers are planted during the growing season along slope contours. The soil is compacted about the layers; spacing and size of plantings depends on the species used. Willow cuttings are often used in this form of slope protection. Figure 2-6 illustrates the brush layering technique.

Brush Mattresses: Layers of brush placed over a stream bank as revetments are often termed brush mattresses, brush matting, or brush revetments. Brush mattresses placed over a stream bank and anchored with stakes or wire can halt bank erosion; large quantities of brush are often required for this type of protection. Live or dead brush can be used in the mattress; if live brush is used then the dormant brush can be planted in a trench at the toe of the bank and laid over the bank. The brush thickness ranges from 2 to 8 inches and extend from the toe of the bank to a suitable distance up-bank. The mattresses are usually anchored at the toe using either rock, fascines, or geotextile rolls. Figure 2-7 provides a schematic of the brush matressing technique. Alternately, bundles of brush (fascines) can be tied together to form a grid which can be then tied to the bank with stakes and wire (Hemphill and Bramley, 1989). Brush mattresses have also been used in combination with live stakes to halt erosion while the stake plantings become established (Edminster, 1949; Gray and Leiser, 1982). This method could, for example, be used around willow stakes instead of riprap to protect the soil around the dormant willow plantings. The method of brush revetments should almost always be used in combination with toe protection, as the brush revetment protects the bank against erosion but not undercutting.

Wattles or live fascines: Cigar-shaped bundles of plant cuttings placed into trenches and secured with wire or stakes are known as either wattles or live facsines. Wattles are most often used in combination with other bank protection measures to promote other vegetation and halt down-slope erosion of soil. Gray and Leiser (1982) describe in detail the process of contour wattling, in which wattles are staked into a slope or stream bank along a contour. Figure 2-8 depicts the contour wattling technique. Wattles can be made of live (dormant) or dead brush; the handling of live cuttings for use in wattles is similar to stake handling. This technique is best suited to stream banks in which overbank runoff and gully erosion is severe. Structure can be added along the bank toe in addition to the wattling if bank undercutting is problematic.

Live Cribwalls: Log or timber cribs encasing plant cuttings, rock, and soil placed at the foot of stream banks can provide the additional strength of structure while masking the artificial appearance through vegetation. By enhancing toe stability, the cribwalls may indirectly revegetate the rest of the bank, as the new stable bank will be more conducive to native plants. Cribwalls have more often been used on dry slopes, but modified cribwalls have lately been used by the Wisconsin Department of Natural Resources (Vetrano, 1988) and the United States Forest Service (US Department of Agriculture, 1995). These modified cribwalls (also termed *Lunkers*) provide good toe protection and enhance fish habitat but require considerable design and construction details.

Trees as Revetments or Weirs: Using trees as stream bank revetments (sometimes termed the "Palmiter method") can provide necessary protection to a stream bank until vegetation can become established. Fallen trees recovered from the stream channel and/or trees cut from on-site can be used in this method. Whether placed as toe protection or along the bank as revetment, the trees need to be anchored to the bank with wire, rebar, or any other means. The advantage of this method is that the trees used in the revetments can usually be found on site; a disadvantage is the decay of the trees over time and the need to securely anchor the trees. The trees are anchored either along the bank or along the toe of the bank through any means possible. Trees have also been used experimentally as bendway weirs on an Illinois stream (Derrick, 1995b); the use of trees as bendway weirs may be advantageous because of their shorter lifetime. Because bendway weirs change the alignment of the flow, the angle of flow relative to the weirs changes also, and the bendway weirs can redirect the flow adversely. Good use of the bendway weir method might be to change the alignment of the weirs over time to compensate for the changes in stream alignment ; the degradation of tree weirs can be timely with the need to realign weirs.

Other Bio-materials: The use of plant materials in manufactured components of erosion control has led to the existence of a variety of mattresses, grids, fabrics, and "bio-logs". Coconut fiber biologs have been effective as wave barriers and as toe protection in stream banks. These biologs can be placed as toe protection and are often planted with plant stems or even woody plants. The biologs are not as durable as riprap for toe protection, but provide a "softer" alternative. The diameter of these coconut fiber rolls ranges from 30 to 70 cm and are biodegradable. However, these manufactured materials are often much more expensive than their constitutive materials, and the strength of these materials is usually not sufficient to justify their cost.

Table 2-4. Advantages and disadvantages of bioengineering techniques.

Advantages	*Disadvantages*
• improved aesthetics over traditional structural designs	• planting and arranging is labor intensive; specialized labor may be required
• increased wildlife habitat (fish, birds), encourages vegetation of banks by native species	• experience in design and planting is usually required
• method can be less costly than structural stabilization techniques	• vegetation alone cannot control toe undercutting (a major cause of stream bank instability)
• plant roots impart additional strength to soil	• success of project is dependent on plant survival (not guaranteed). Time for plant establishment is required.
• woody vegetation filters non-point source pollutants from groundwater inflow	• designs vary with location and are not easily repeated between sites
	• "dead" bio-revetments will eventually decay

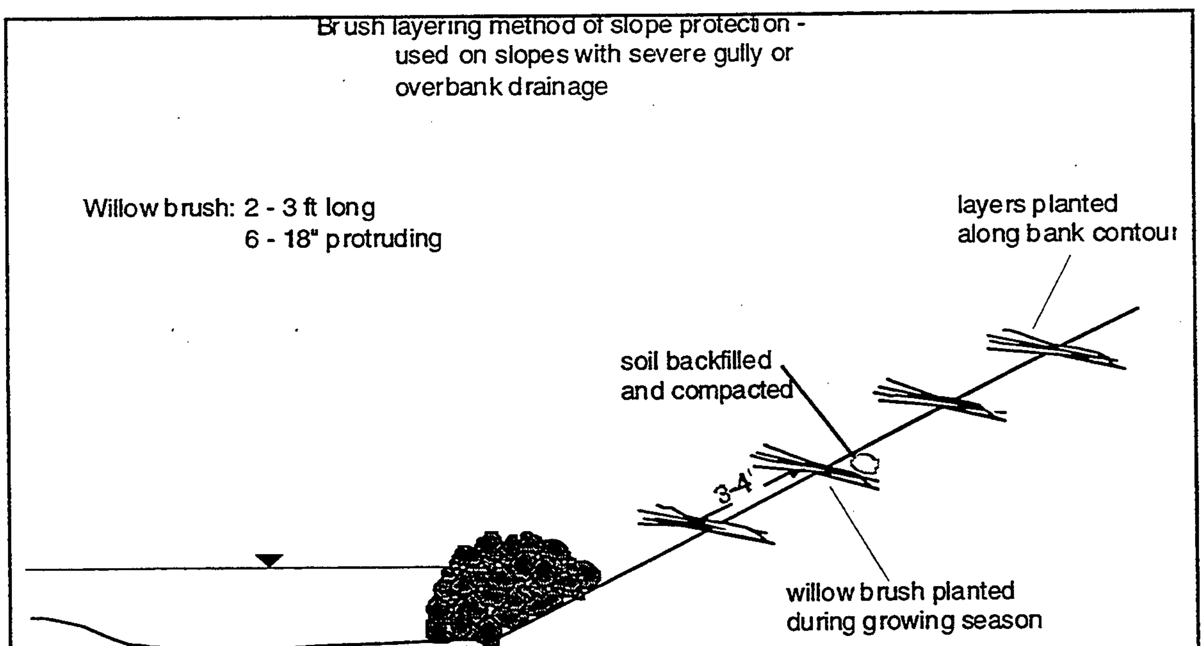

Figure 2-6. Brush layering method. (After: Gray and Leiser, 1982)

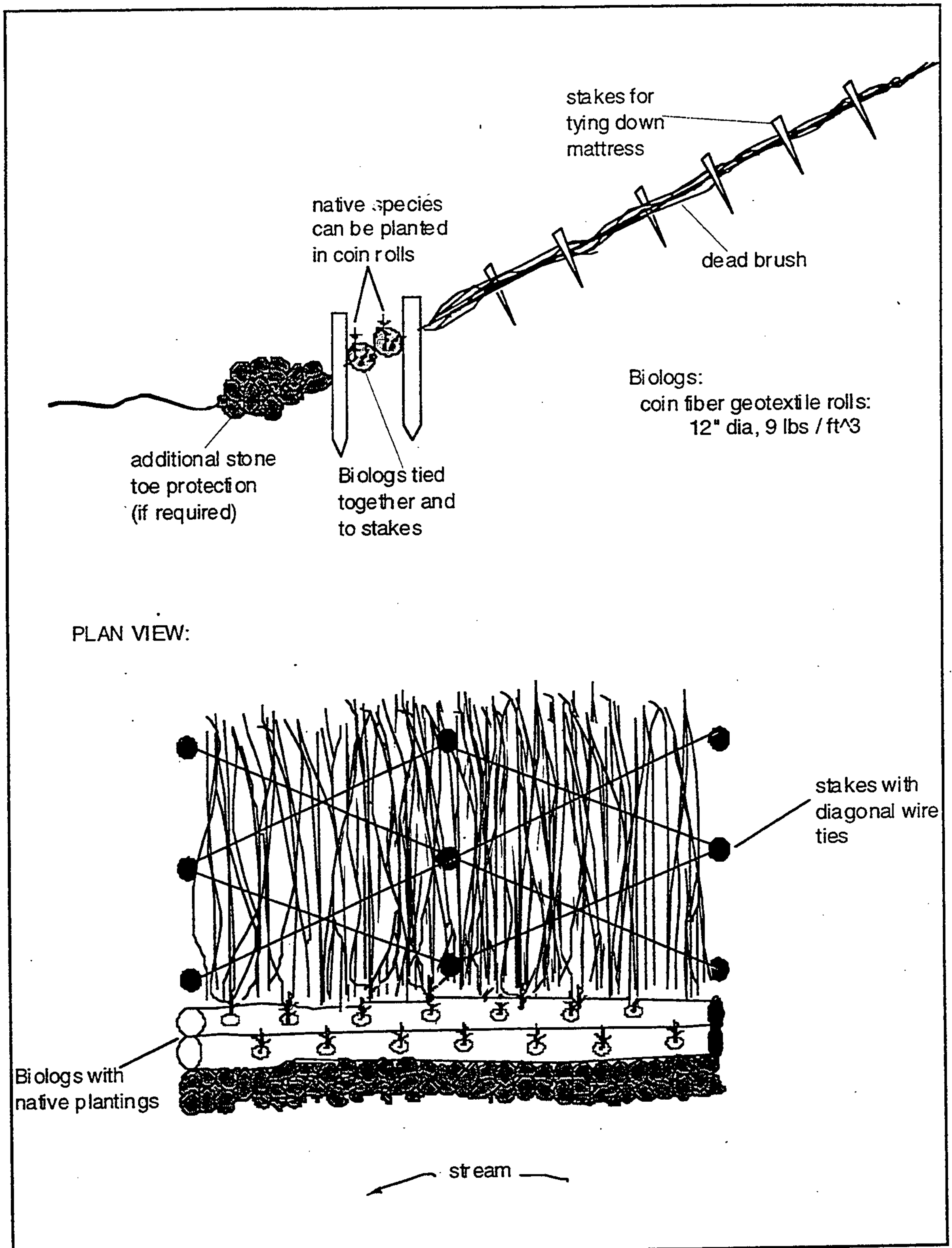

Figure 2-7. Brush mattressing with biolog toe protection. (After: Gray and Leiser, 1982)

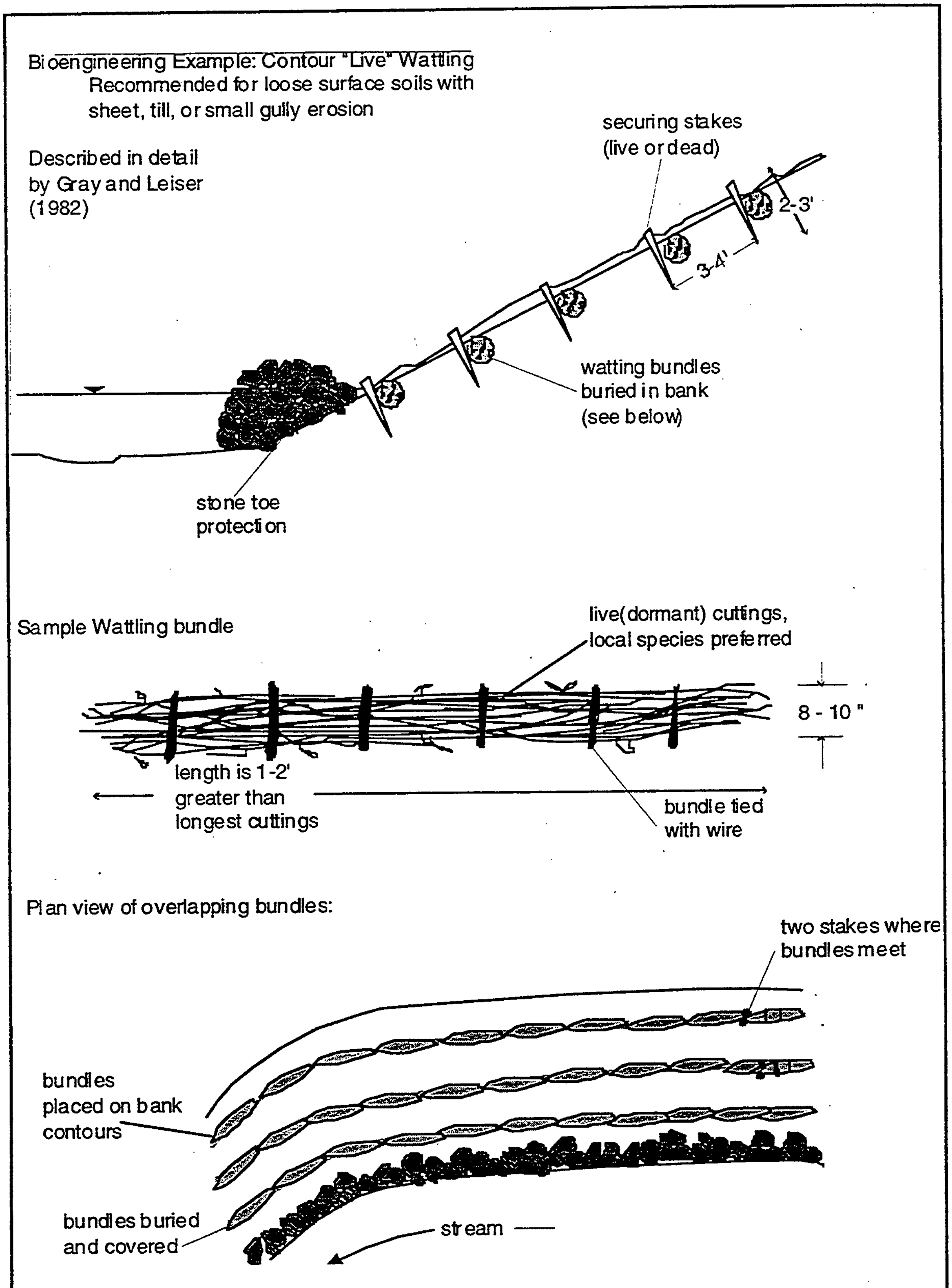

Figure 2-8. Contour "live" wattling. (After: Gray and Leiser, 1982)

Geo-Reinforcement Materials

The term geo-reinforcement material is adopted here to refer to a large number of commercial products which, when applied on or into the soil of a stream bank, provide geological support and protection against stream erosion. In general, geo-reinforcement materials can be divided into two categories: those composed of biological materials and those of the synthetic type. Table 2-5 discusses advantages and disadvantages of geo-reinforcement materials.

Biological geo-reinforcements products were discussed briefly in conjunction with bioengineering methods of erosion control. Commercial products composed of plant material are available in a variety of forms including mattresses, grids, fabrics, mats, blankets, and bio-logs. While these products may provide environmentally and aesthetically sound protection for small streams and channels, they cannot often be applied as a sole protection measure on medium to large size streams. Typically, they do not possess the strength and durability to withstand higher flows. Also, cost for these biological geo-reinforcements materials limits their use on large projects. Biological reinforcement materials also have the disadvantage of decaying over time, this may require higher maintenance costs. In some bioengineering protection schemes, the biodegradable nature of these material can be exploited to supply temporary support until natural vegetation can be established.

Synthetic geo-reinforcement materials also come in an extensive variety of forms including grids, mats, blankets, fabrics, honeycombs, and bricks. Depending on the material chosen, varying degrees of protection can be attained. Brick revetment, for example, is capable of supplying high levels of protection. Of course, purchasing and installation costs reflect the level of protection attained. Each product has its own particular installation requirements and applications; therefore, the use of geo-reinforcement materials is very site dependent. In general, a large portion of the products on the market do not possess extensive historical track records, making evaluation of field success difficult. Also, many of these products can be quite costly, rendering them impractical for large bank protection projects. Geo-reinforcement materials do play a successful role in bank protection when they are utilized as a supplement to the measures and design methods previously discussed in this report.

Table 2-5. Advantages and disadvantages of geo-reinforcement materials.

Advantages	*Disadvantages*
• Varying degrees of available protection levels allow for versatile application	• Biological geo-reinforcement has questionable strength for medium to large streams
• Biological geo-reinforcement may provide temporary support for the development of natural vegetation	• Biological geo-reinforcement may decay over time, increasing maintenance costs
• Can be highly successful when implemented as a supplement to other protection measures	• Limited historical track record makes evaluating potential for success difficult
	• Cost

Monitoring Methods

A comprehensive literature review by Lawler (1993) examines existing methods of measuring lateral stream and river channel migration. The method of measurement chosen is dependent on the time scale over which the channel migration is being considered. For longer time scales (100-1000 yrs), the quantification of channel migration involves the use of sedimentary evidence, botanical evidence, or historical sources. Intermediate time scales of measurement (1-30 yrs) usually involve planimetric survey and stream cross-profiling; the smallest measurement time scales require high-resolution measurement techniques such as erosion pins, photogrammetry, and even photo-electronic sensors. All of these methods are described in Lawler (1993), which includes schematics of typical surveys. The selection of the measurement technique is based on the site characteristics, the time scales over which measurements will be taken, the finances and logistics, and the technology available.

For short time scales (months-years), the measurement technique must be capable of detecting migration on the order of centimeters. Several such methods are currently in use. Table 2-6 summarizes some advantages and disadvantages of the various monitoring methods.

1. *Erosion pins*. Erosion pins are a traditional method of erosion measurement in which rods are inserted vertically into the ground. The portion of the pins protruding from the ground is measured at initial insertion; subsequent observations measure the portion exposed at later times, quantifying the rate of erosion at each pin location.

The placement and spacing of the pins depends on the site. This measurement is point-specific, and the degree of spatial resolution depends on the spacing of the pins, which in turn depends on the financial, time, and logistic constraints of the project. The time between pin readings will depend on the degree of temporal resolution desired and the site accessibility.

An erosion pin network can be easy to establish and maintain. Erosion pins are capable of detecting small amounts of bank migration, without requiring special technology or skills to obtain the measurements. Furthermore, the pins are relatively inexpensive and easily replaced.

Erosion pin measurements have the disadvantage of measuring relative motion between the soil and the pin, and thus it cannot be known whether erosion has actually taken place, or if the soil has merely swelled or contracted relative to a stable pin. Usually, however, this type of soil movement is small. Also, like most measurement techniques, the measurements are point measurements and must be extrapolated to yield a continuous profile.

2. *Photogrammetric measurement.* Using a specially built terrestrial photogrammetric camera, in conjunction with a theodolite, stereoscopic photographs can yield a three-dimensional resolution of the site. Subsequent photographs can give the terrestrial change over time and can be plotted as contour maps.

Although accurate and efficient, this technology is expensive and often not an alternative for individual projects with small budgets. However, the expense might be warranted if the survey was to be of many different sites over longer time scales.

3. *Repeated photography*. Another less expensive photographic survey technique that is available is repeated photography. Repeated photography is most often used to give a general idea of where bank erosion is most prevalent; detailed measurements (quantification) are not obtained, nor can slight erosion be detected. This technique is best suited for dynamic erosion where the terrestrial change can be observed between successive photographs. Techniques have been developed to obtain measurements from normal photographs, but the accuracy of these techniques is not suitable for environments which are not very dynamic. This photographic method is more suitable for obtaining qualitative information about bank migration and for identifying dynamic areas.

4. *Painted sections*. Another visual technique for following bank migration is painted sections. Sections of the bank can be painted with a grid of paint markings; observations over time can reveal areas which are being eroded more than others. This technique is often used in conjunction with repeated photography to give a qualitative idea of areas undergoing severe erosion.

5. *Repeated cross-profiling or surveying*. Traditional survey techniques are popular in measuring bank and channel erosion over longer time scales (1-30 yrs). These techniques either involve profiling the stream at regular cross sections or making a planimetric survey of the stream channel using theodolite and tape or EDM equipment. Although this method is more suited for longer time scales, it can be used to quantify dynamic channel migration.

Table 2-6. Advantages and disadvantages of monitoring methods.

Method	*Advantages*	*Disadvantages*
• cross-profiling	easy	markers may be lost
• planimetric survey	result is plan view of lateral migration	definition of bank sometimes unclear
• erosion pins	accurate	requires repeated readings of pin network
• remote sensing	Allows for dynamic data acquisition	Costly. Current technology-insufficient resolution.
• photography	easy, cheap if simple pictures are used	without special equipment, method only gives qualitative results

Introduction

A study by Herricks and Mattingly (1991) documented the status of Illinois streams. The report outlines the effects and locations of the most heavily channelized drainage basins in the state, while providing case studies of streams undergoing channelization. Stream channelization is often indicative of bridge-stream or bridge-road alignment problems; furthermore, bridge alignment problems and stream migration problems in general are often caused by stream channelization. For this reason, channelized streams may be good indicators of where bridge-stream alignment problems exist in Illinois.

A main mechanism characteristic of stream channelization and/or basin development is channel incision. Channel incision is a stream adjustment process, involving channel bed degradation and aggradation, that often results in stream channel migration through toe undercutting. This mechanism was prevalent in many of the sites observed during this study.

Site Selection

In order to find sites with bridge-stream alignment problems, personnel in various government agencies were questioned, including IDOT, the Department of Natural Resources, and the Illinois State Water Survey. All nine IDOT, district offices were contacted. Many district offices did not respond or reported no known problems. The survey generated approximately 20 sites. Though it was not possible to visit all potential sites due to time and budgetary constraints, 14 sites were visited. The following sites were investigated but not chosen:

1. Kickapoo Creek near Lawndale (County road, Logan Co.)

The Kickapoo Creek just east of Lawndale is eroding an outer bank severely; this meander is now (3/96) approximately thirty feet from the road. The stream is about 40 ft wide at the troubled bend. A sandy point bar has developed on the inner portion of the bend, and several large trees have fallen into the creek where the banks are nearly vertical. It appears that someone has attempted to halt the bank erosion by dumping a large amount of concrete rubble onto the outside bank. The trees and this concrete have made the bank erosion problem at this site very complicated, and it seems unlikely that any low-cost measures would stabilize this site. The fallen trees might be anchored against the bank as an immediate effort. However, even if the erosion is halted at this outside bend, it appears that erosion will continue just upstream of this part of the 180° bend. Long term stabilization of the site might be achieved with a long field of bendway weirs and bank shaping.

2. *Prairie Creek near Springfield (IL Rte. 125, Sangamon Co.)*

The Prairie Creek near Springfield is scouring just upstream of the IL Route 125 bridge. This bridge, however, does not seem to be in immediate danger, and the bank erosion at this site could not be considered severe. A small fence has been placed along the right bank upstream of the bridge, apparently to halt some of the bank erosion. Although the creek is making a sharp left turn into the bridge, it appears that the flow magnitudes are not sufficient to cause excessive scour.

3. *Prairie Creek near Farmingdale (Farmingdale Road, Sangamon Co.)*

A second site on Prairie Creek near Springfield is a bridge on Farmingdale Road not far from the IL Rte. 125 site. This site is also in good condition, except for one bridge pier which has accumulated some woody debris. The bridge debris may be deflecting the flow, but no major bank erosion appears to be threatening the bridge. Several hundred feet upstream of the bridge, an active meander exists. The stream is well-aligned with the bridge, and no major problems exist at the site.

4. *Sangamon River north of Oakford (IL Rte. 27, Menard Co.)*

The Sangamon River's size at Oakford precludes this site from being a possible candidate for low-cost bank protection works; however, the Sangamon River at Oakdale is actively eroding the banks and depositing sediment in sand bars. This site might be a candidate for a bendway weir system or other flow modification structures. Figure 3-1 shows Sangamon River as observed on a March 3, 1996 site visit.

5. *Lick Creek Tributary near Pekin (IL Rte. 98, Tazewell Co.)*

The Lick Creek Tributary near North Pekin crosses IL Rte. 98 just east of IL Rte. 29. The left bank just downstream of the bridge is slightly eroding as the flow emerging from the bridge is forced to turn to the right. A riprap covering and a mesh have already been placed over the bank and along the bottom of the channel. The riprap appears to have halted the bank erosion and effectively armored the stream. The bridge does not appear to be threatened by the downstream erosion.

6. *Mackinaw River near Congerville (County road, Woodford Co.)*

The Mackinaw River north of Congerville is a large river. Although the banks are undergoing local erosion upstream and downstream of the bridge, this site has no pronounced meander threatening the bridge. Only local scour problems exist, and these problems might be solvable with vegetative means. It appears that in the future, an upstream bend could migrate toward the bridge, but at present the site appears to be stable.

7. Shoal Creek (IL Rte. 140, Bond Co.)

The Rte. 140 bridge crossing of Shoal Creek is seriously scouring downstream of the bridge. Riprap dumped under the bridge has caused a major elevation difference (estimate: 3 ft) between the upstream and downstream water elevations. This is causing a deep scour hole just downstream of the bridge; the right downstream bank is also eroding as the water has been deflected through the riprap bed toward this bank. An active meander also exists several hundred yards downstream of the bridge; this meander requires stabilization to prevent more adjacent farmland from being lost. The land owner, in conjunction with another government agency, is planning to implement bend protection measures, probably involving riprap and toe protection. Concrete rubble is already on site and may be used as bank fortification. This bend, although not threatening the Rte. 140 bridge, might still be useful in observing the effectiveness of protection measures. A site survey is planned by the land owner. Figure 3-2 shows Shoal Creek as observed on a February 26, 1996 site visit.

8. Small stream (IL Rte. 26, Putman Co.)

A Photograph (Figure 3-3) from this site shows a small stream, draining approximately 3.0 mi^2, which makes a sharp turn into a bridge. The banks show active scour problems, with exposed tree roots and irregularities. The site is small enough to be a candidate for low-cost stream bank protection. This site was not visited.

9. Mission Creek (US Rte. 52, LaSalle Co.)

A Photograph (Figure 3-4) and bridge inventory shows this site as a small stream meander compressing on an upstream-facing bridge abutment. This small stream has well- vegetated banks; however, the bridge abutments have been exposed by erosion. The small stream may continue to erode the supporting material from the bridge if no bank protection measures are taken.

10. Drainage ditch (IL Rte. 18, Putman Co.)

Bridge #078-0013 on IL Rte. 18 has problems with an abutment which has been undermined. No water was flowing at the time of the inspection, and the bridge disrepair cannot be definitely attributed to stream activity.

The following four sites have been chosen for case study. The description below provides a brief introduction to the site. Detailed site profiles are found in the next chapter.

1. Cahokia Creek (IL Rte. 140)

Cahokia is an actively meandering creek. Just upstream of the Rte 140 bridge, there is the potential for the creation of an oxbow lake. Such a cutoff would isolate several acres of farmland and potentially put the bridge at risk to a downstream progressing degradation wave. The downstream bend of the meander has been previously treated with a riprap toe and willow posts. The upstream bend is protected by a riprap toe and "minimal stone" bendway weirs. Still, there is concern that erosion of the unprotected upstream banks will continue the trend towards cut-off.

2. Piasa Creek (IL Rte. 3)

Erosion at the Piasa site is responsible for severe scour immediately upstream of the Rte. 3 bridge. Several large trees on the left bank have been undermined and are likely to fall. The meander is beginning to press toward the bridge. The near vertical banks on-site are being undermined at the toe, and irregular scarring indicates that mass failure is primarily responsible for bank movement. Considerable effort will be required to stabilize this site.

3. Senachwine Creek (IL Rte. 29)

On the Senachwine Creek, a sharp meander is pressing against the upstream side of the Rte. 29 bridge abutment and road. The toe of the roadway and bridge foundation is under considerable risk of being undermined. At the opposite bank, an electrical tower is also being threatened by encroaching banks. Any protective measures taken on this site must armor both banks. The channel is cluttered with fallen trees and brush. These materials will hamper initial remediation attempts; however, it is possible that this debris may be used in some low cost bank stabilization designs.

4. Spoon River (IL Rte. 17)

Upstream of the Rte. 17 bridge, the Spoon River presents a mild meander which currently threatens nearby farmland. In the long run, the meander may begin to have adverse effects on the bridge and roadway. At this stage, the site is a perfect candidate for low cost treatment; such an effort should effectively prevent the situation from worsening.

Figure 3-5 shows the general locations of the sites considered for case study. Those four sites chosen for case study have been highlighted with an inlay photograph.

Figure 3-1. Sangamon River - March 4, 1996 site visit.

Figure 3-2. Shoal Creek - February 26, 1996 site visit.

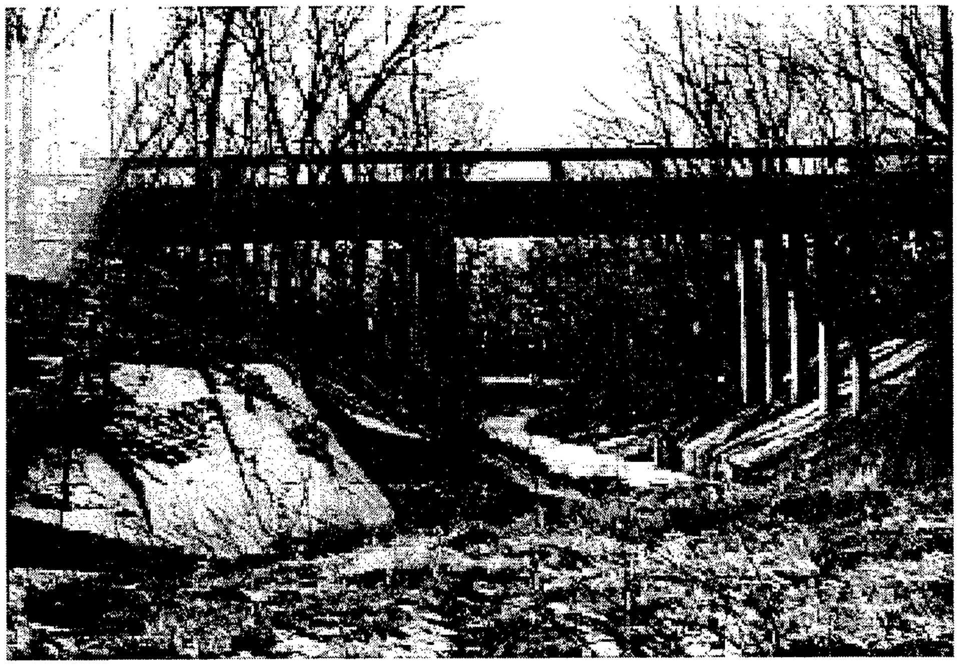

Figure 3-3. Small Stream at IL Rte 26, Putnam Co. - Photograph provided by IDOT.

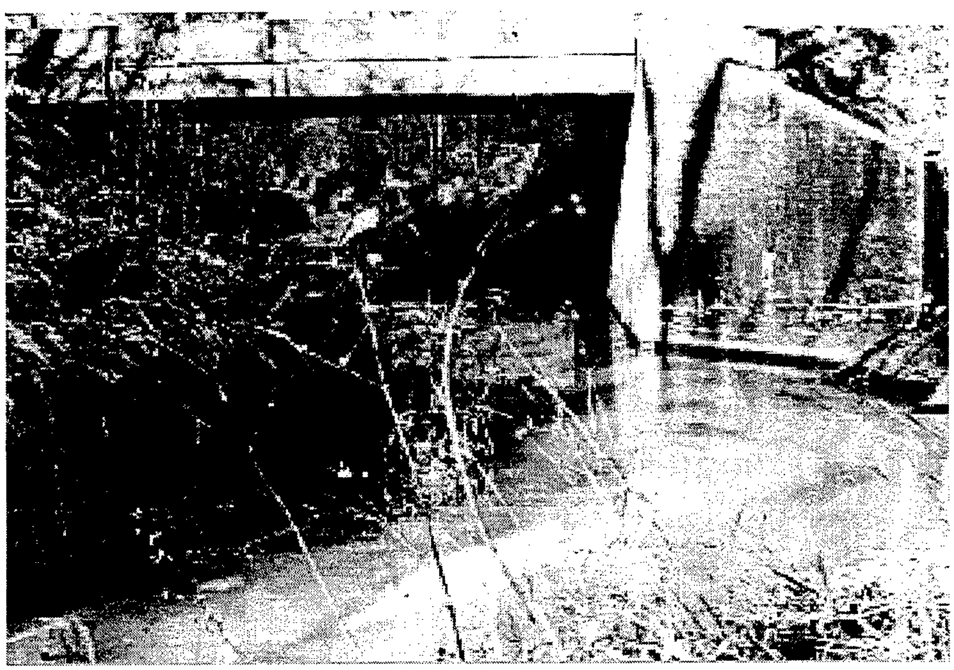

Figure 3-4. Mission Creek at US Rte 52, LaSalle Co. - Photograph provided by IDOT.

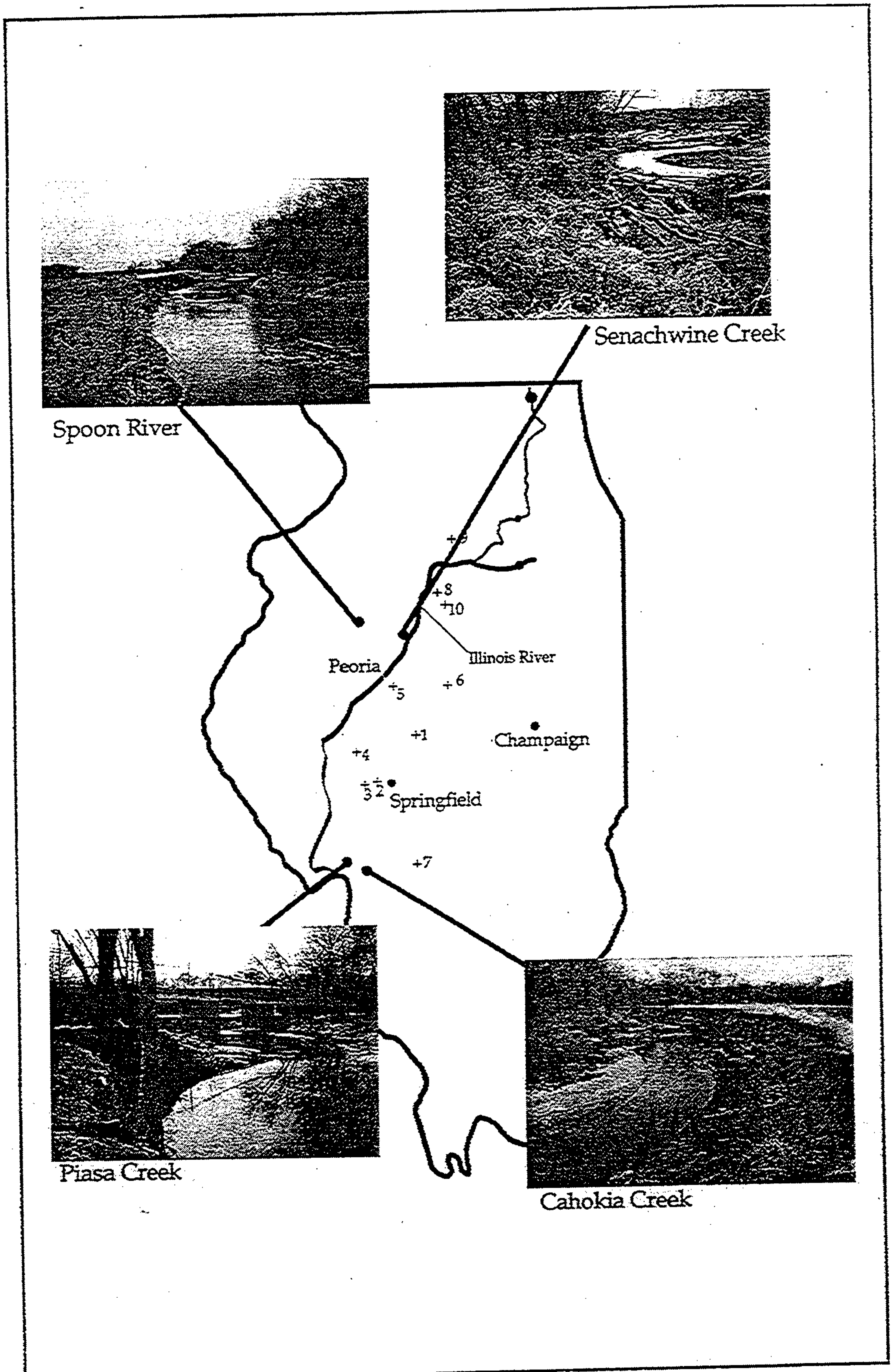

Figure 3-5. Stream sites investigated throughout Illinois.

Introduction

The final case study sites were selected on the basis of the applicability of low-cost stream bank protection methods to the site, the availability of site data, and the degree to which the sites represented stream bank erosion problems with bridges in Illinois. The relative location of the chosen sites have been marked on Figure 3-5. Below is a detailed description of the sites chosen for case study.

Cahokia Creek

Cahokia creek begins at Mount Olive Lake in the vicinity of Edgarville in Macoupin County. It travels south by southwest through Madison County into a manmade diversion channel which leads its flow into the Mississippi River east of St. Louis. The site is located 1/4 mile downstream of the creek's intersection with Illinois Route 140 in Madison County (Figure 4-1).

Hydrologically, the site drains an approximate 150 square miles over an average slope of 5.3 feet per mile. The basin has been outlined in Figure 4-2, and Figure 4-3 shows the time lapsed orientation of Cahokia Creek in the vicinity of the study area. Soils predominant to the basin are summarized below. The erosion factor, K, gives a qualitative feel for the tendency for erosion within the basin.

Table 4-1. Major soil associations within Cahokia Creek site drainage basin.

Soil Association	Brief Description	K[1]
Hickory-Marine-Hosmer	• Well to somewhat poorly drained • Moderately to very slowly permeable • Nearly level to very steep	0.39
Herric-Piasa-Virden	• Poorly drained • Moderately slowly permeable • Nearly level	0.31
Coffeen-Lawson-Wakeland	• Well to somewhat poorly drained • Moderately permeable • Nearly level to very steep	0.32

1. The erosion factor, K, indicates the susceptibility of a soil to sheet and rill erosion by water. Values of K range from 0.05 to 0.69; the higher the value, the more susceptible the soil is to erosion by water. For purposes of this table, K is the arithmetic mean of the erosion factors across the soil association.

Flood magnitudes and frequencies for the Cahokia Creek at Route 140 have been calculated through hydrologic anaysis. For more information concerning this effort, see Appendix IV. The resulting flood data are given in Table 4-2.

Table. 4-2 Estimate of flood frequency and magnitued at Cahokia Creek Site.

Flood Frequency (yr.)	Flood Magnitudes (cfs)
2	3,703
5	4,998
10	5,855
25	6,938
50	7,742
100	8,540

Inspection of the drainage basin aerial photographs (Figures 4-4 and 4-5) show the evolution of Cahokia Creek. It has been channelized in many sections, often cutting off large stretches of meandering stream. In addition, stands of trees have been replaced with farmland throughout the drainage basin. These two human activities, deforestation and stream channelization, are responsible for the current dynamic behavior of the water body.

The land surrounding the Route 140 site is used for farming, and at some locations farm plots extend to the banks. Other areas exhibit dense brush, grass, and trees vegetating the banks. In general, the banks are quite steep, almost vertical, specifically, the outer banks which abut against the farm land. Soil on-site is Tice Silt Loam. This soil is a dark, grayish brown friable silty loam. Loam soils exhibit characteristics similar to that of loess soils; however, they have a greater percentage clay constituent. This make loam soils silightly more capable of sustaining steep slopes. Surface runoff is low, and the soil is somewhat poorly drained. It is frequently flooded for brief periods from March through June. An erosion factor of 0.32 (K scale 0.05-0.69) indicates the soil has a moderately high susceptibility to erosion by water. Figures 4-6 and 4-7, photographs taken during the February 26,1996 site visit, characterize the site.

Upon choosing the site for further study, a detailed survey was completed. Figures 4-8 and 4-9 summarize the survey effort. The upstream cross-section at station 10+22.36 exhibits both outer and inner banks of relatively gradual slope. Each cross-section downstream thereafter have increasingly steep-sloped outer banks.

In the vicinity of Route 140, the Cahokia is actively meandering. Here, the trend toward the creation of an oxbow lake is unmistakable. The bend is very close to cutting through the remaining land bridge, isolating several acres of farmland. If the stream cuts off, bed erosion may ensue, leading to stability problems at the Route 140 bridge. The downstream section of the bend has already been stabilized with riprap and willow posts, but the upstream portion of the bend is untreated and continues to erode. The vertical banks appear to be migrating by mass failure. The creek currently appears stable next to the bridge. The site has been investigated by the Illinois Natural Resource Conservation Service, the Illinois Water Survey, and the

Corps of Engineers; these agencies cooperated with the land owner to implement existing protection measures.

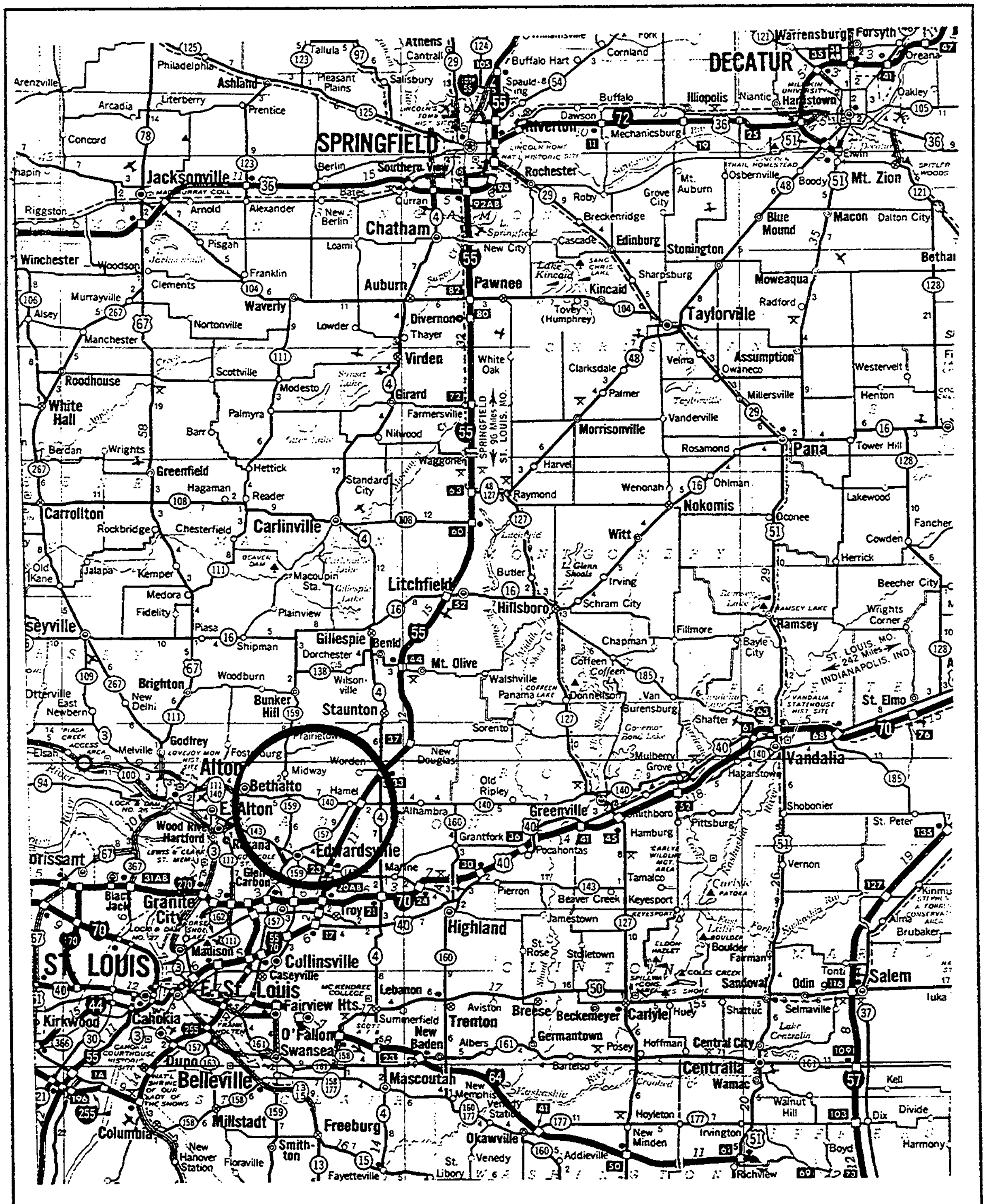

Figure 4-1. Location map of Cahokia Creek site.

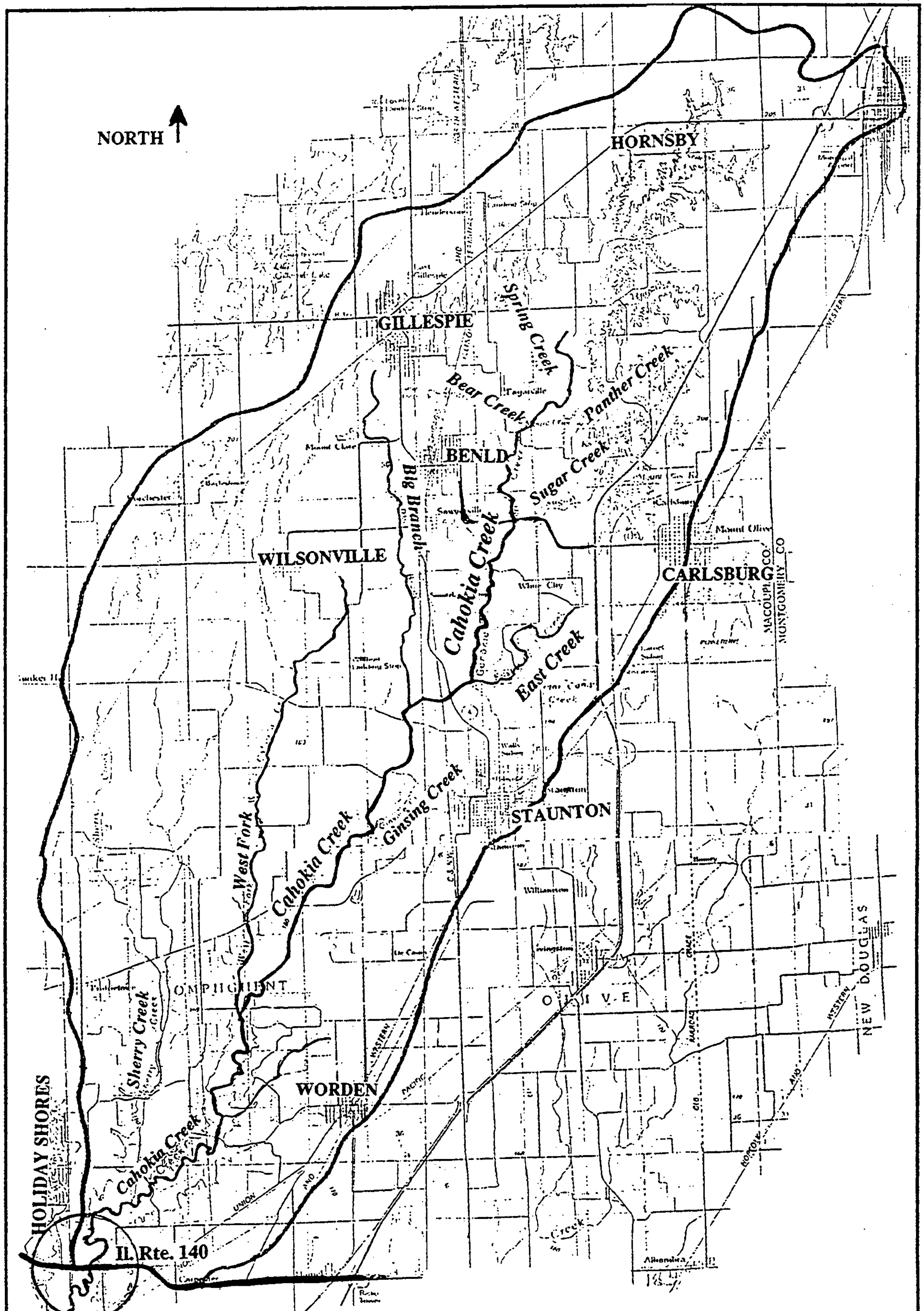

Figure 4-2. Outline of Cahokia Creek case study site drainage basin.

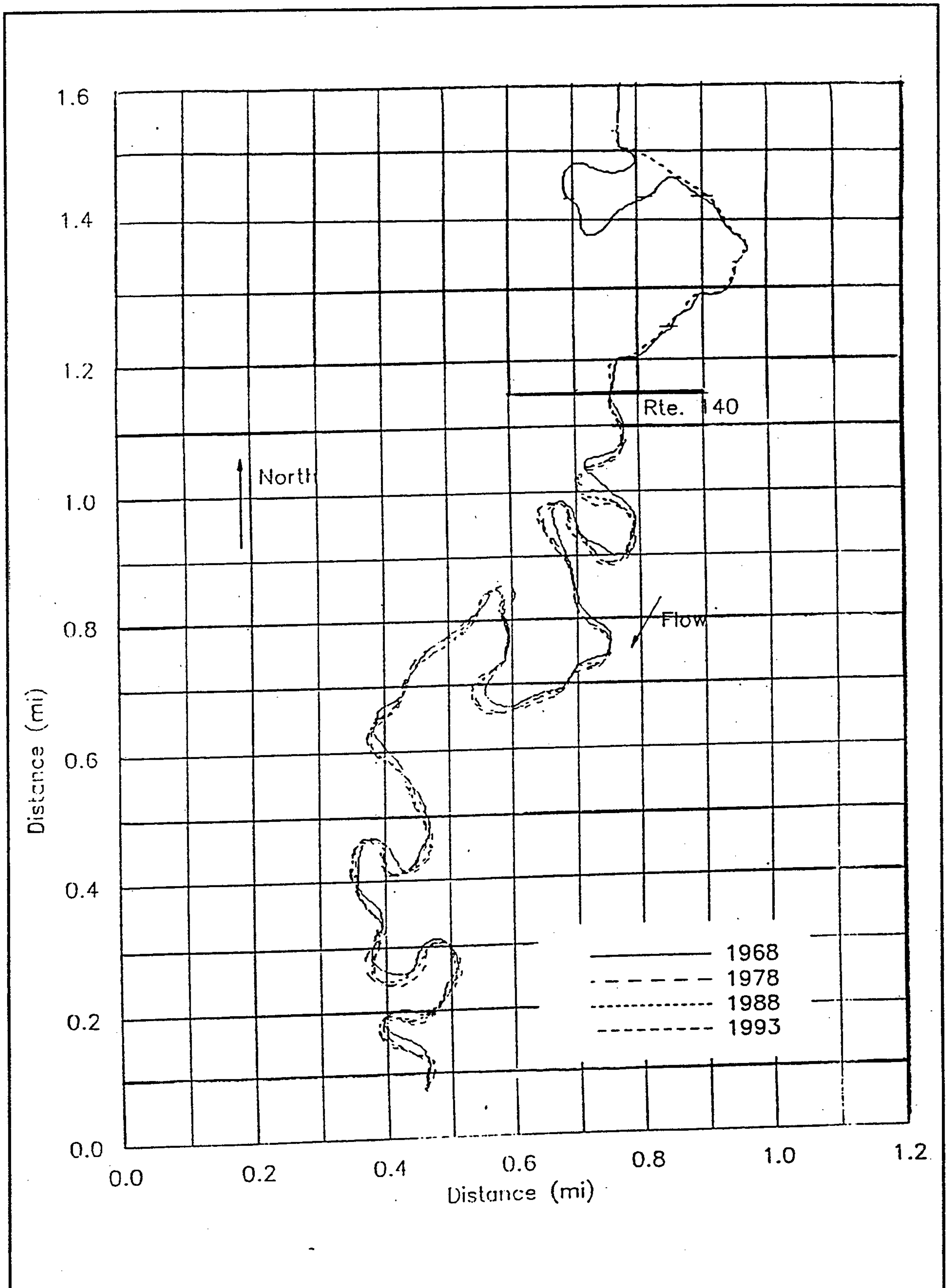

Figure 4-3. Orientation of Cahokia Creek in vicinity of study area.

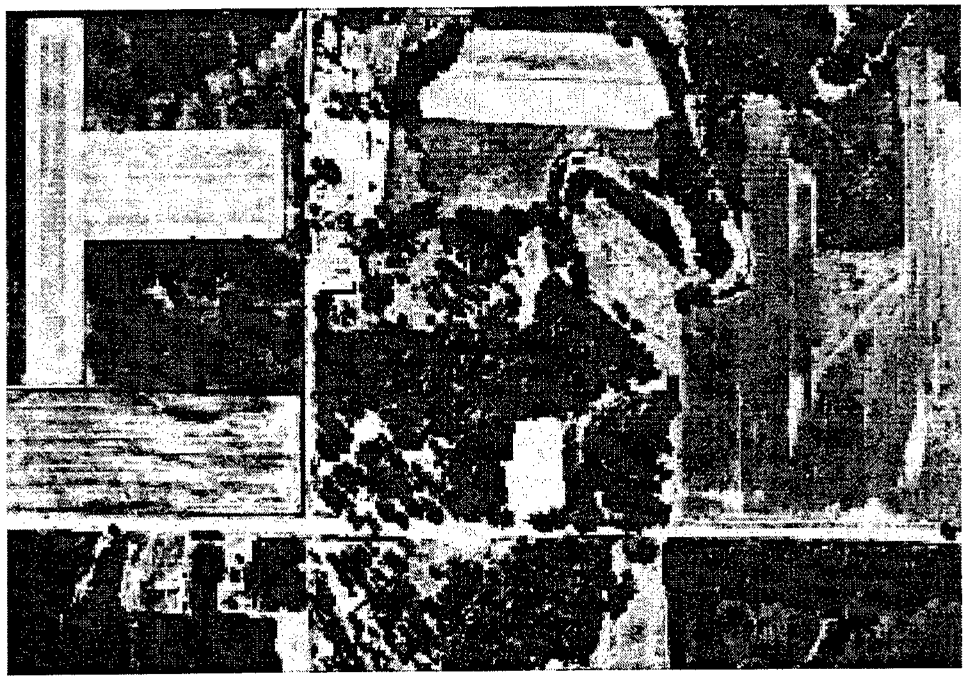

Figure 4-4. Aerial photograph of Cahokia Creek basin - 1968.

Figure 4-5. Aerial photograph of Cahokia Creek basin - 1994.

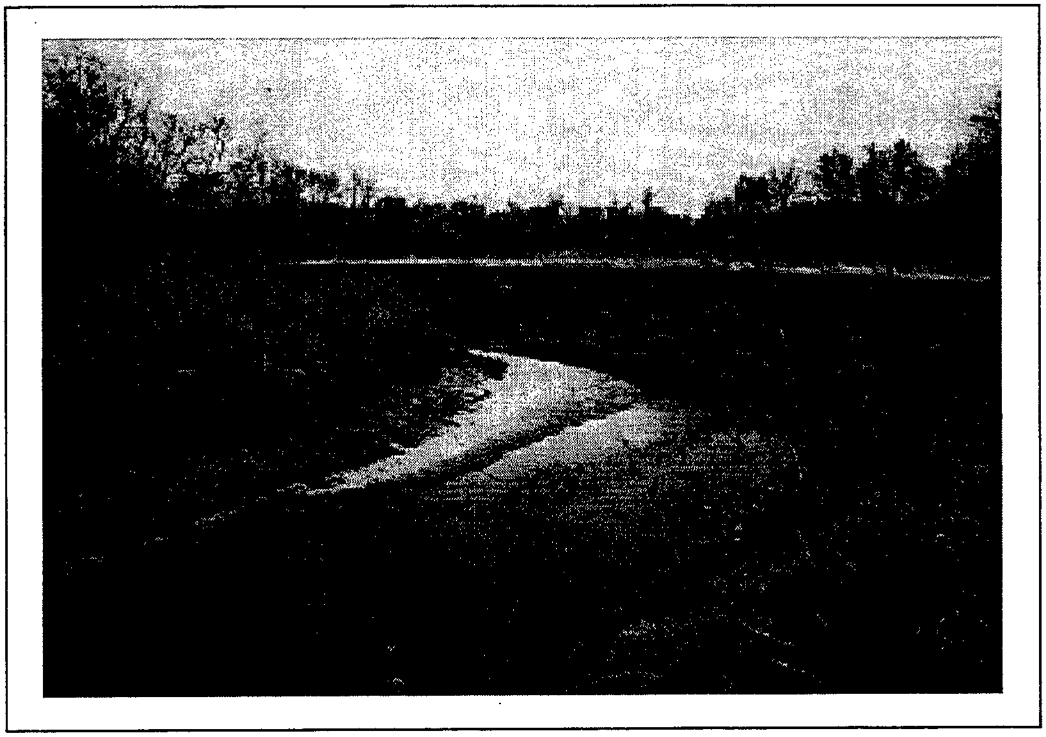

Figure 4-6. Cahokia Creek site - taken during February 26, 1996 site visit.

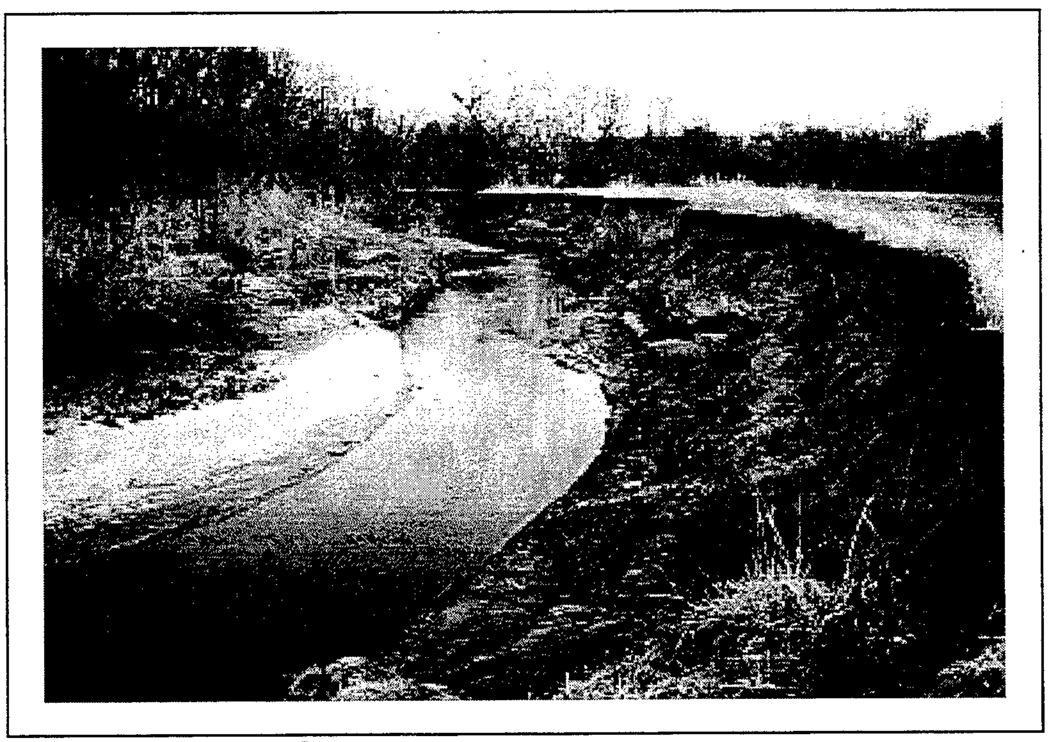

Figure 4-7. Cahokia Creek site - taken during February 26, 1996 site visit.

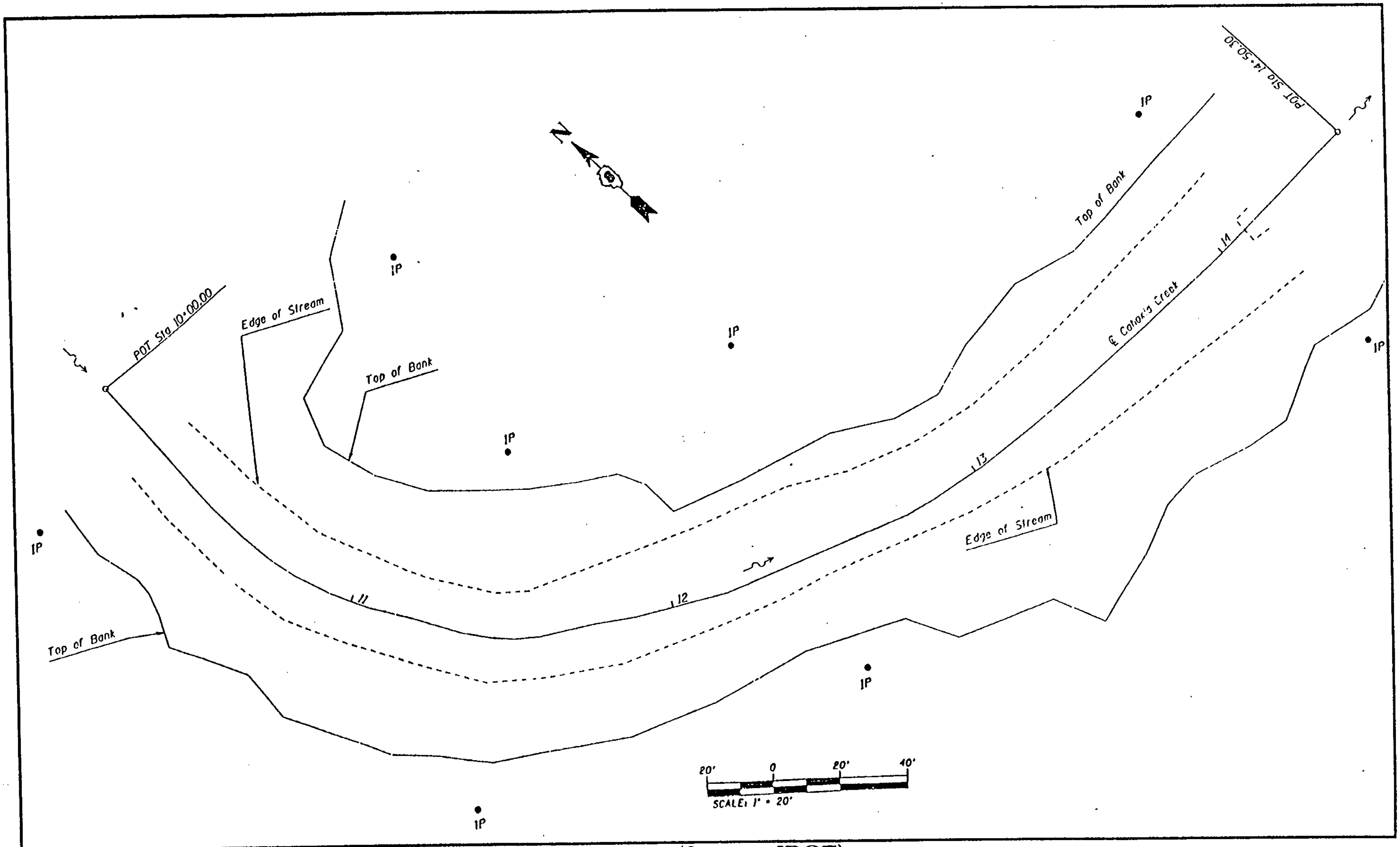

Figure 4-8. Plan survey of Cahokia Creek, July 1996. (Source: IDOT)

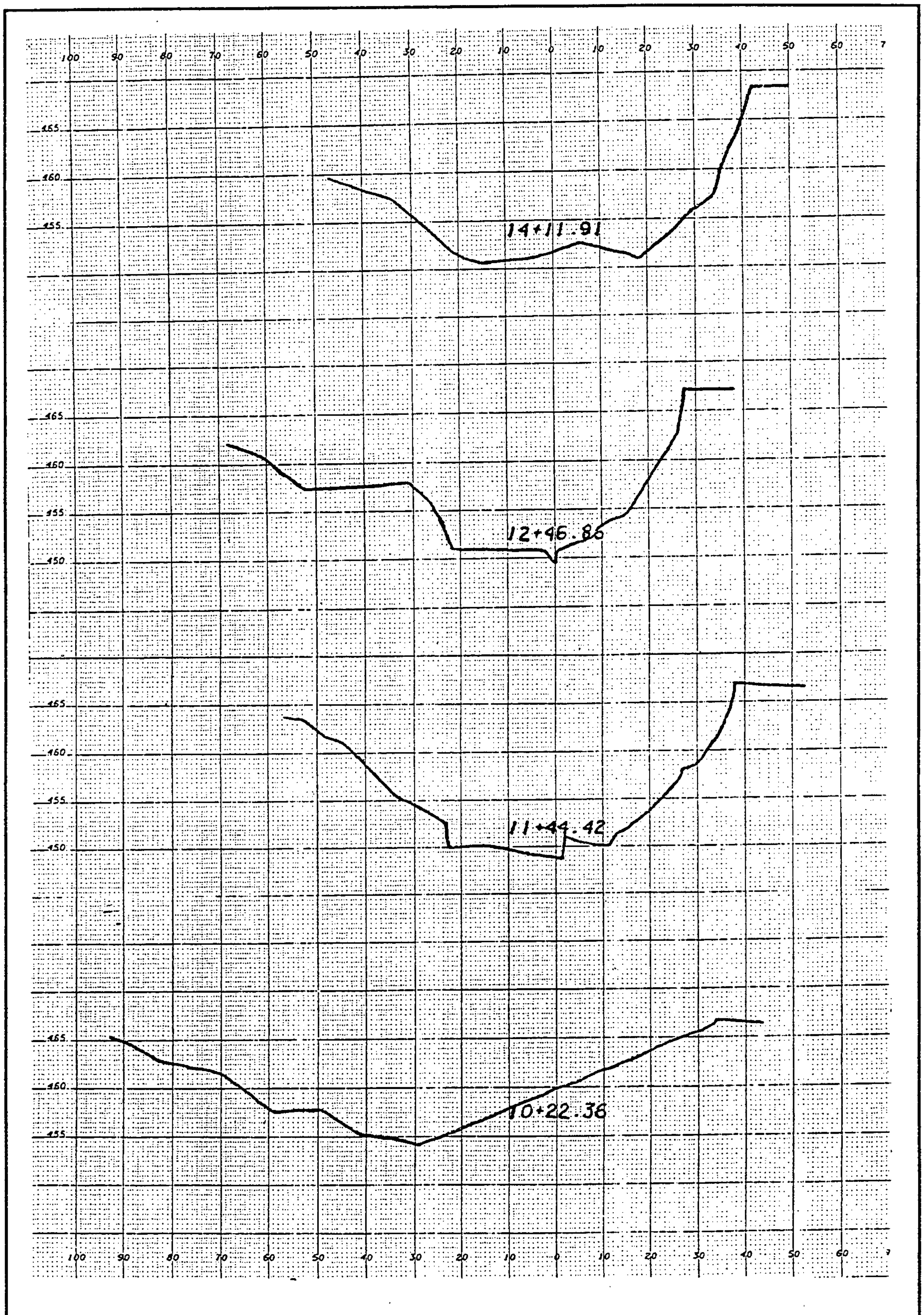

Figure 4-9. Cross-sectional survey of Cahokia Creek, July 1996. (Source: IDOT)

Piasa Creek

Piasa Creek begins west of Shipman in the County of Macoupin. It travels south by southwest through Jersey County where it empties into the Mississippi River just west of Godfrey, Illinois. The site is located at the Creek's intersection with Illinois Route 3 (Figure 4-10).

Hydrologically, the site drains approximately 98 square miles with an average slope of 5.8 feet per mile. The drainage basin has been outlined in Figure 4-11, and Figure 4-12 shows the time lapsed plan view of Piasa Creek in the vicinity of the study area. Soils predominant to the site have been summarized in Table 4-3.

Table 4-3. Major soil associations within Piasa Creek site drainage basin.

Soil Association	Brief Description	K[1]
Fayette-Rozetta	• Well drained • Moderately permeable • Gently sloping to Steep	0.37
Marine-Rozetta Stronghurst	• Poorly to moderately well drained • Slowly to moderately permeable • Nearly level to gently sloping	0.37
Wakeland-Orion Birds	• Somewhat poorly drained • Moderately permeable • Nearly level	0.37
Hickory-Rozetta	• Well drained to poorly drained • Moderately to very slowly permeable • Nearly level to very steep	0.37
Herrick-Virden	• Somewhat poorly to poorly drained • Moderately slowly permeable • Nearly level	0.37
Herrick-Piasa-Virden	• Poorly drained • Moderately slowly permeable • Nearly level	0.31

1. The erosion factor, K, indicates the susceptibility of a soil to sheet and rill erosion by water. Values of K range from 0.05 to 0.69; the higher the value, the more susceptible the soil is to erosion by water. For purposes of this table, K is the arithmetic mean of the erosion factors across the soil association.

Flood magnitude and frequency for the Piasa Creek at Route 3 have been calculated using the USGS estimating techniques for rural Illinois Streams (Curtis, 1987). For more information concerning this technique and its accuracy see Appendix III. Flood data are summarized in Table 4-4.

Table 4-4. Estimate of flood frequency and magnitude at Piasa Creek site.

Flood Frequency (yr.)	Flood Magnitudes (cfs)
2	3,260
5	5,550
10	7,130
25	9,190
50	10,720
100	12,220

Inspection of the drainage basin's aerial photographs (Figure 4-13 and 4-14) show that the Piasa Creek has not been as active as the Cahokia Creek in the past 35 years. The exception is a meander which has been approaching the Route 3 bridge from upstream; this meander is now beginning to migrate downstream of the bridge. However, the problems at the Illinois Route 3 bridge appear to be localized and not representative of an otherwise stable drainage basin.

The land surrounding Route 3 site is used primarily for farming. The banks are vegetated with small trees, brush, and grass. Some trees have been eroded from the banks and clutter the channel. In general, the banks at the Piasa site are not as steep as those on the Cahokia site. Downstream of the bridge and at several upstream scour holes, the slopes do degrade to near vertical. The soil on-site is Fayette Silt Loam. This soil is moderately permeable and well drained with average slopes of 7-12%. An average erosion factor of 0.37 (K scale 0.05-0.69) indicates a moderately high susceptibility to erosion. Figures 4-15 and 4-16, photographs taken during the February 26, 1996 site visit, characterize the site.

Upon choosing the site for case study, a detailed survey was completed. Figures 4-17 and 4-18 summarize the survey effort. The plan view shows the skewed bridge approach with the left bank scarred by numerous scour holes. Cross-sections at stations 102+54.48 and 103+20.93, immediately downstream of the bridge, exhibit steep sloped banks.

In the vicinity of the Route 3, the Piasa exhibits strong erosion trends. There is active scouring of the left bank upstream of the bridge. This mechanism appears to be forcing the meander towards the Route 3 bridge. Examining the time lapsed position of the Creek, Figure 4-12 confirms a strong push to the left just upstream of the bridge. No protective measures have been taken at the site, except for a small stone weir near the left bank. This has deflected the flow against the bank resulting in a large scour hole. Several other deep (approximately 6 feet) scour holes exist on site, and at these locations, the banks are nearly vertical. Irregular failure scars indicate that the banks are apparently undergoing mass failure. A telephone cable crossing under the creek near the bridge should be kept in mind when executing any erosion control plan.

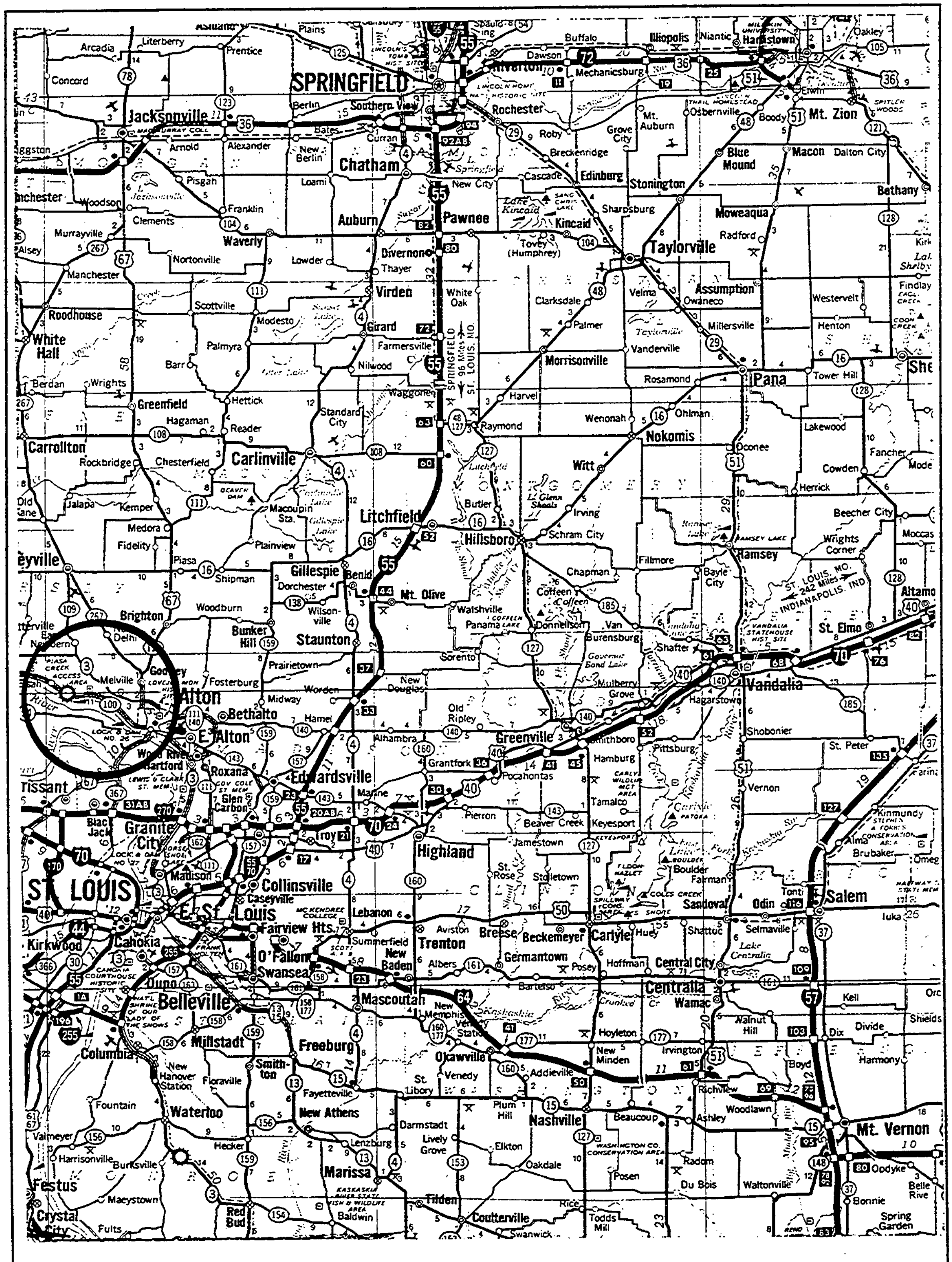

Figure 4-10. Location map of Piasa Creek site.

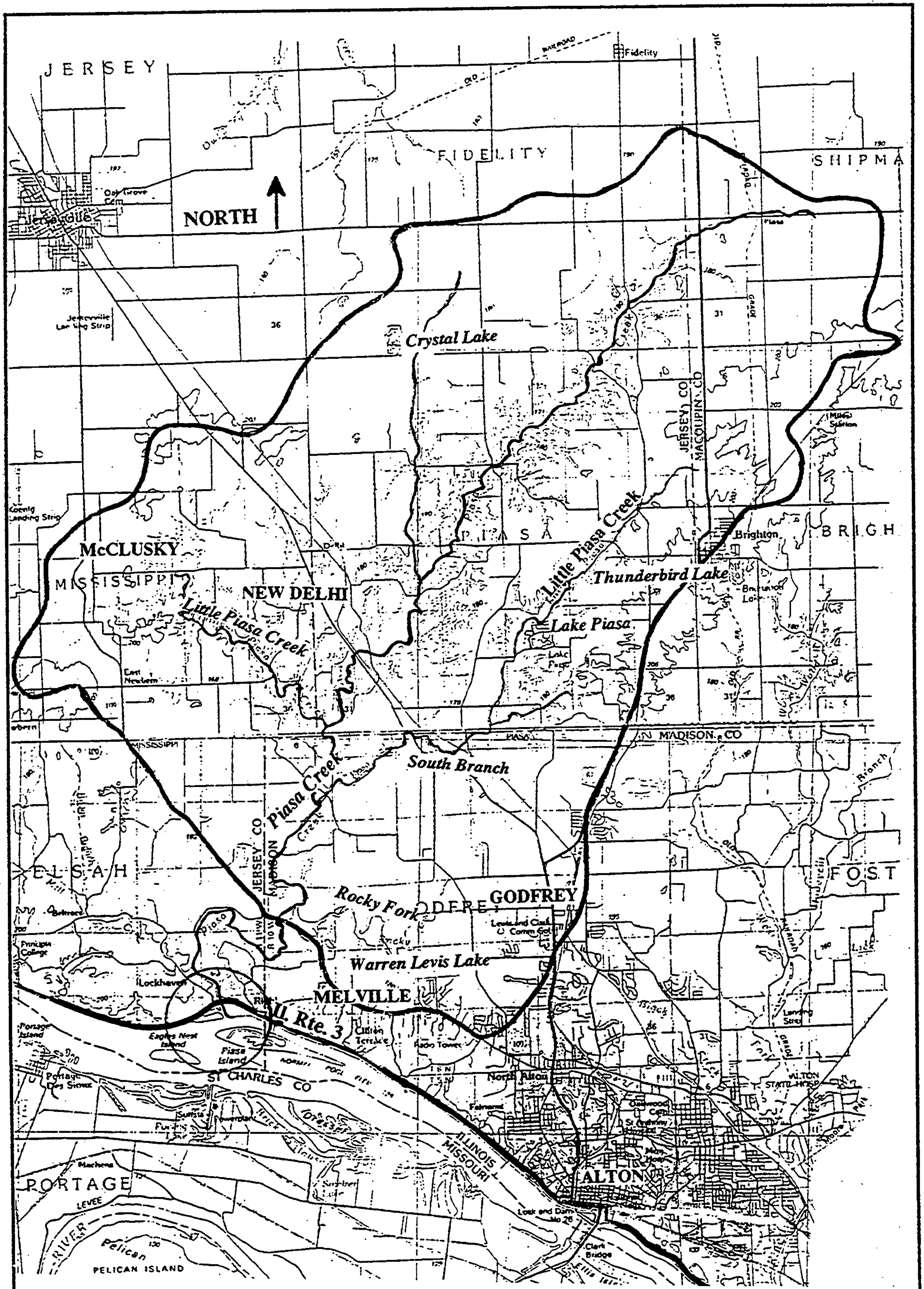

Figure 4-11. Outline of Piasa Creek case study site drainage basin.

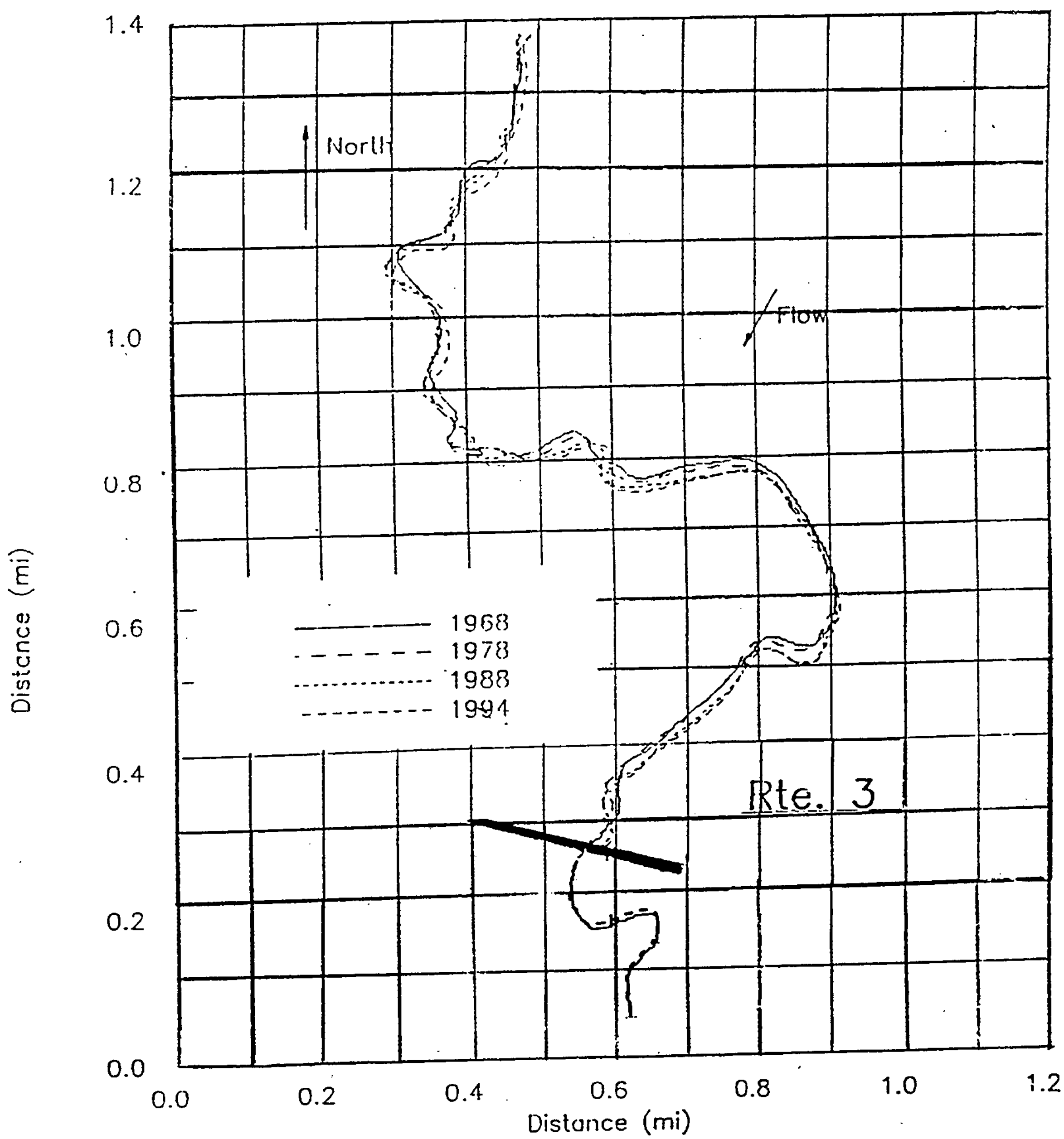

Figure 4-12. Orientation of Piasa Creek in vicinity of study area.

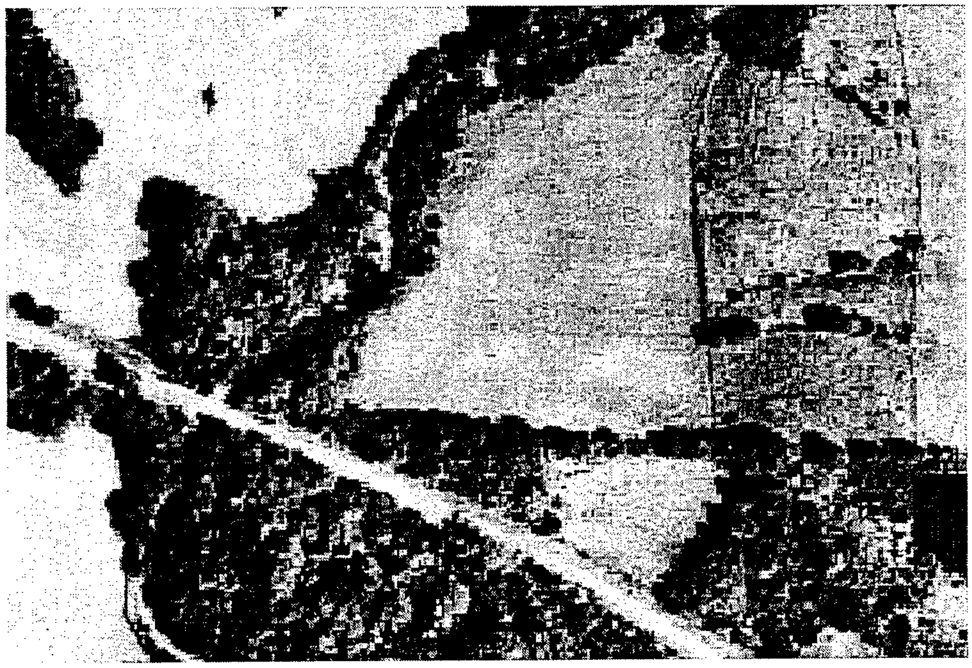

Figure 4-13. Aerial photograph of Piasa Creek basin - 1968.

Figure 4-14. Aerial photograph of Piasa Creek basin - 1994.

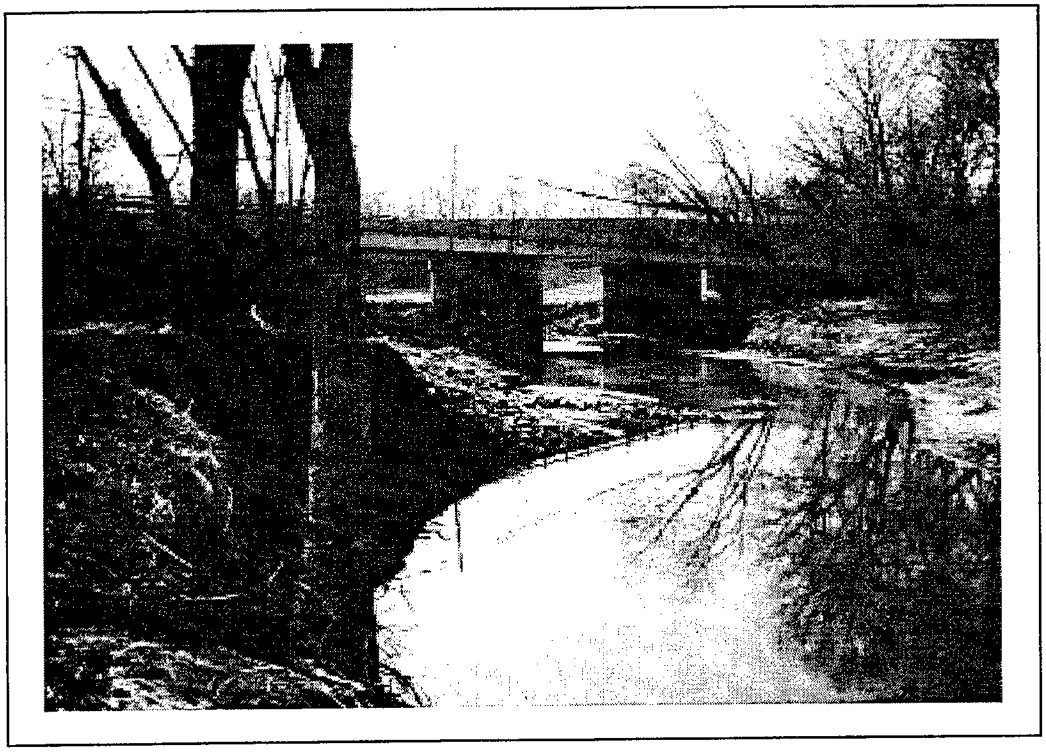

Figure 4-15. Piasa Creek site - taken during February 26, 1996 site visit.

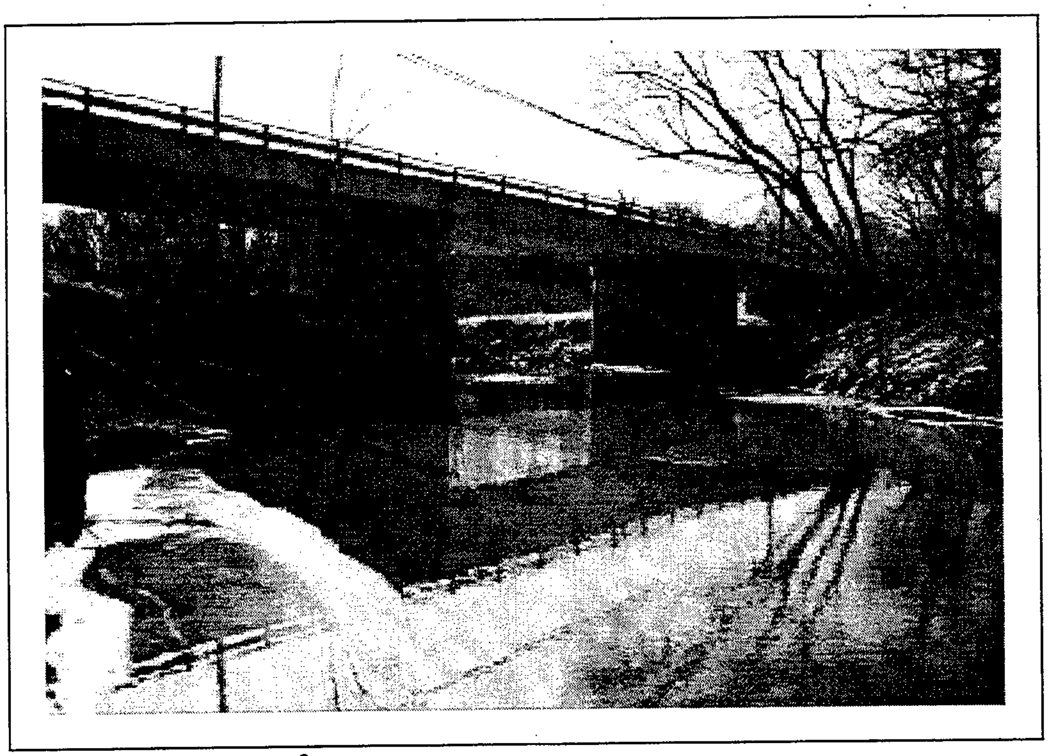

Figure 4-16. Piasa Creek site - taken during February 26, 1996 site visit.

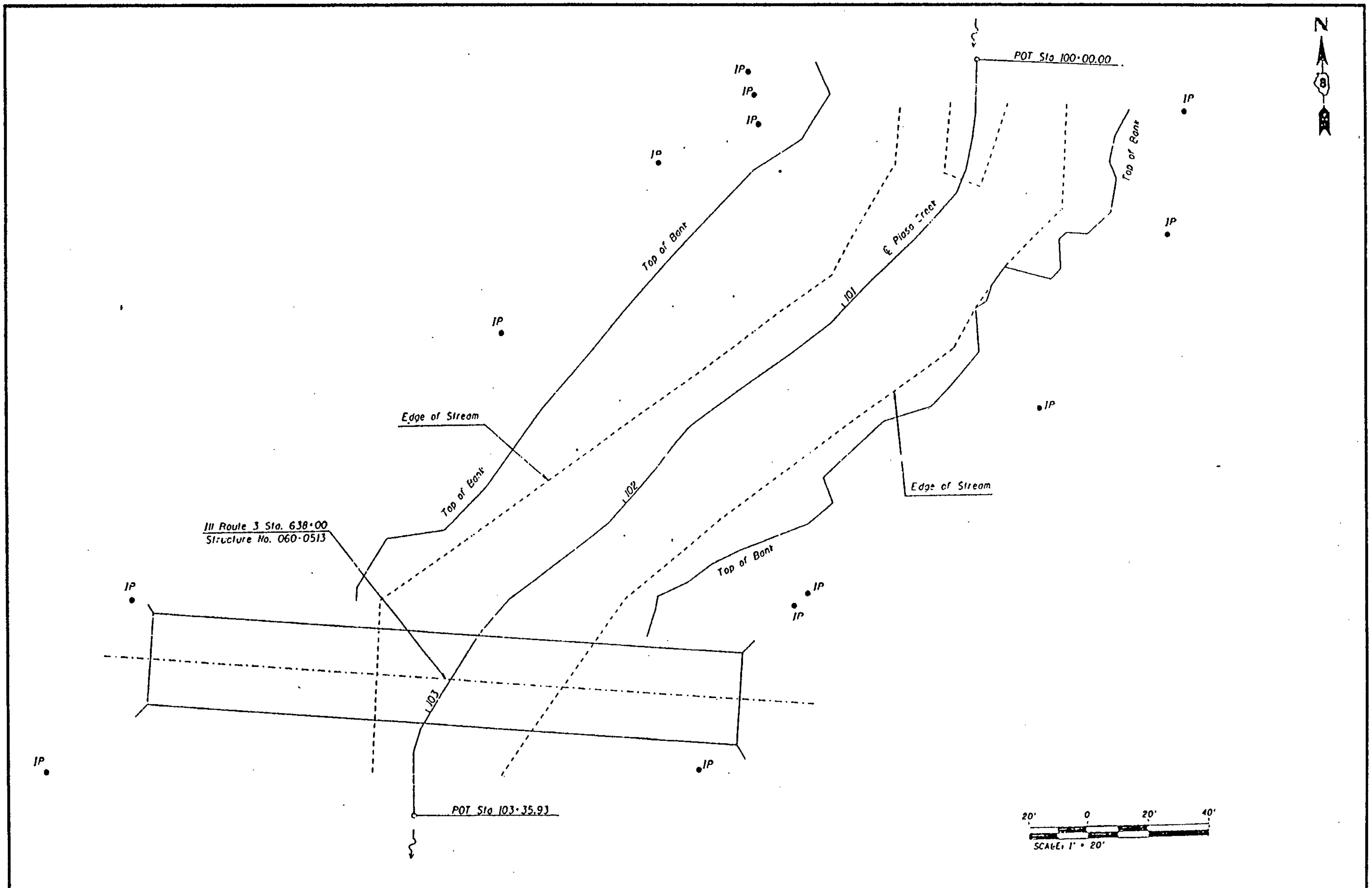

Figure 4-17. Plan survey of Piasa Creek, July 1996. (Source: IDOT)

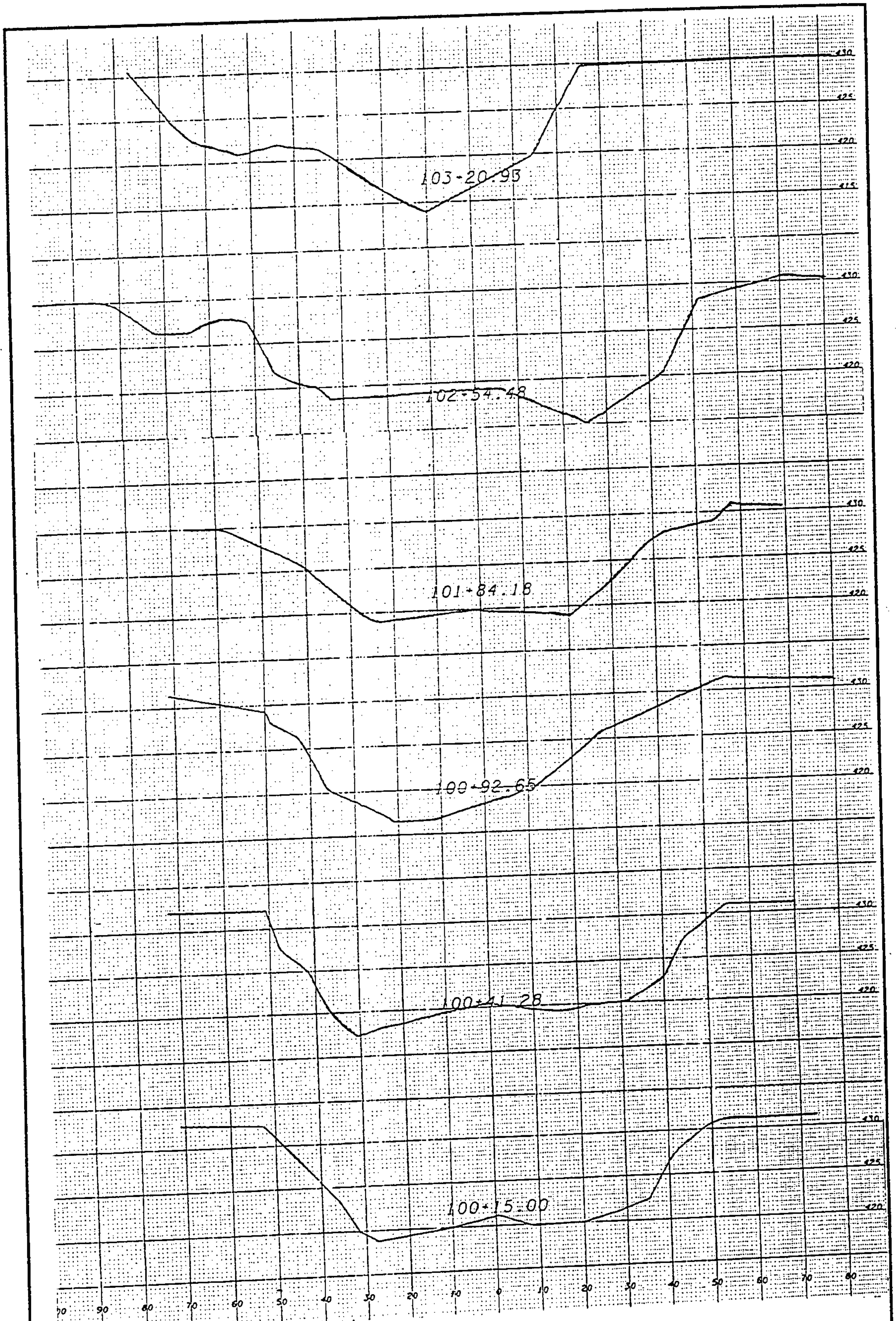

Figure 4-18. Cross-sectional survey of Piasa Creek, July 1996. (Source: IDOT)

Senachwine Creek

The Senachwine Creek is a many-branched water body with its furthest reaching tributaries extending northeast of Camp Grove in Marshall Co. The creek flows predominantly south until it reaches North Hampton where it makes a sharp turn to the east. The Senachwine enters the Illinois River upstream of Chillicothe, in Peoria County. The study site is north of Chillicothe where the creek passes under Illinois Route 29 bridge (Figure 4-19).

Hydrologically, the site drains approximately 89 square miles with a relatively steep average slope of 10.3 feet per mile. The basin has been outlined in Figure 4-20, and Figure 4-21 shows the time lapsed orientation of Senachwine Creek in the vicinity of the study area. Soils composing the basin are summarized in Table 4-5.

Table 4-5. Major soil associations within Senachwine site drainage basin.

Soil Association	Brief Description	K[1]
Catlin-Saybrook-Osco	• Moderately well drained • Moderately permeable • Nearly level	0.31
Harco-Sable Elkhart	• Moderately well drained • Moderately permeable • Nearly level to sloping	0.31
Rozetta-Keomah	• Moderately well drained • Moderately permeable • Nearly level to gently sloping	0.37
Muscatine-Osco-Sable	• Somewhat poorly drained • Moderately permeable • Nearly level	0.28
Hennepin-Birkbeck-Miami	• Well drained • Moderately slowly permeable • Steep to very steep	0.33

1. The erosion factor, K, indicates the susceptibility of a soil to sheet and rill erosion by water. Values of K range from 0.05 to 0.69; the higher the value, the more susceptible the soil is to erosion by water. For purposes of this table, K is the arithmetic mean of the erosion factors across the soil association.

Flood magnitudes and frequencies for the Senachwine Creek at Route 29 have been calculated using the USGS estimating techniques for rural Illinois Streams (Curtis, 1987). For more information concerning this technique and its accuracy, see Appendix III. The resulting flood data are presented in Table 4-6.

Table 4-6. Estimate of flood frequency and magnitude at Senachwine Creek site.

Flood Frequency (yr.)	Flood Magnitudes (cfs)
2	2,670
5	4,600
10	5,970
25	7,780
50	9,120
100	10,460

Inspection of the drainage basin aerial photographs (Figure 4-22, 4-23) show the evolution of Senachwine basin. Land use within the basin does not appear to have changed much over the last 35 years. Still, the meandering nature of the stream has been quite pronounced over the same period.

The land surrounding the Route 29 bridge site is primarily farm pasture. The immediate banks, however, are wooded and cluttered with dense brush. In some locations, bank slopes degrade to vertical. Soil on-site is Jules Silt Loam. This soil is dark to yellowish brown and is nearly level and well drained. Figures 4-24 and 4-25, photographs taken during the February 26,1996 site visit, depict the site.

Upon choosing the site for further study, a detailed survey was completed. Figures 4-26 and 4-27 represent the survey effort. The plan view shows a large electrical tower approximately 30 feet from the eroding banks. The cross-sections exhibit a relatively wide, flat channel. The primary channel is complimented by a secondary flat flood plain along the outer banks. This flood plain begins to break down at sections D-D and E-E as the creek turns into the bridge. The result is a wider channel eroding along the outer banks.

The creek meander just upstream of the Route 29 bridge is threatening both the bridge and power line tower. Figure 4-21 shows that this meander has become very pronounced during the last 35 years. Bank failure and scour is occurring where the river is forced to turn at a right angle toward the bridge. The bend meander is very active and littered with fallen trees. More trees are in immediate danger of being undercut and will undoubtedly fall soon, complicating any erosion control efforts. The creek has experienced some extreme flood events, evident by the woody debris deposited on the high parts of the banks. A large point bar exists immediately upstream of the site, and the outer parts of the channel have been scoured deeply. The difficulties presented by this site will test the limits and success of low-cost stream bank protection methods. At this site, any low-cost effort should be viewed as temporary until it is proven successful, and more extensive measures may be necessary.

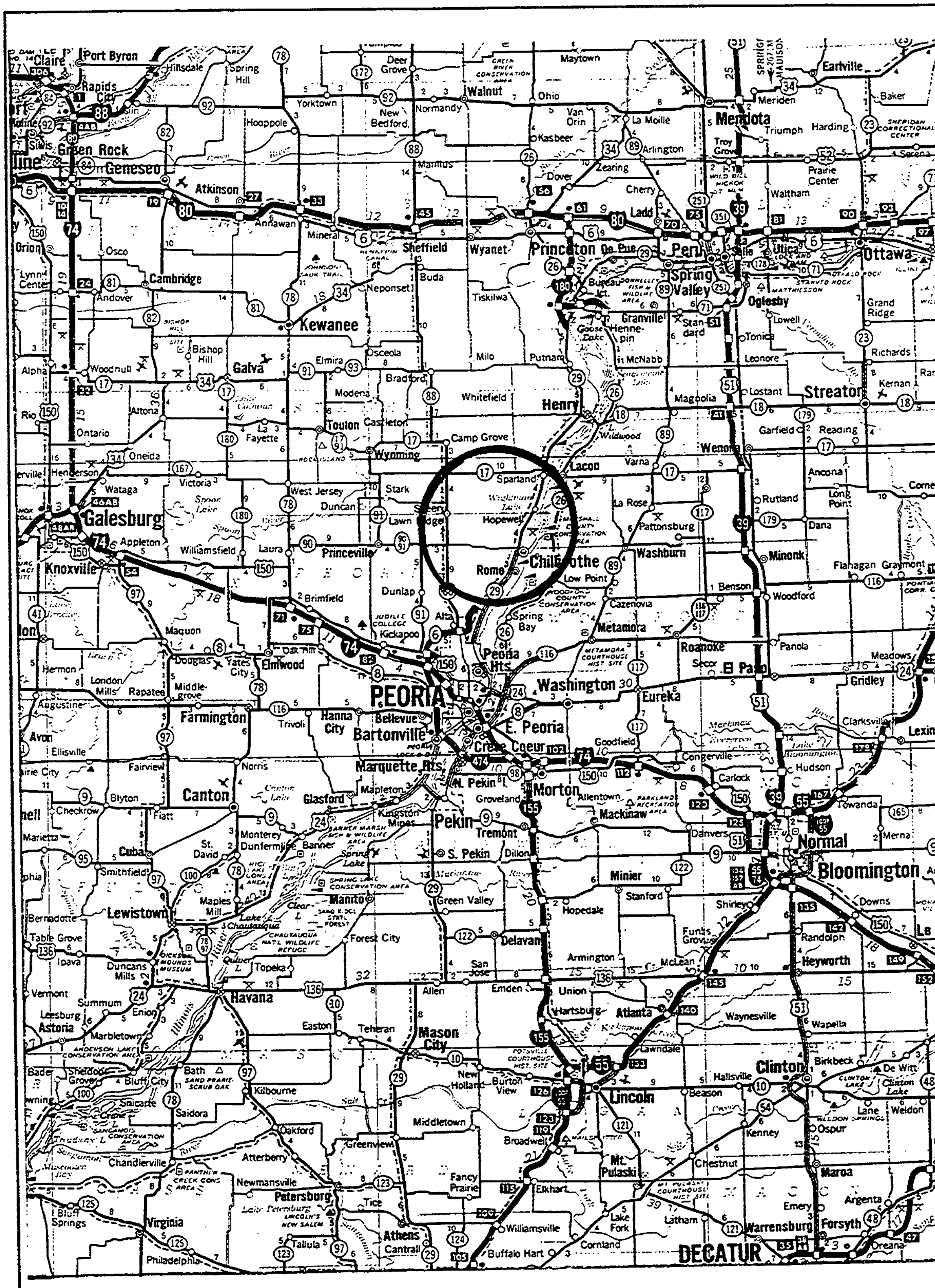

Figure 4-19. Location map of Senachwine Creek site.

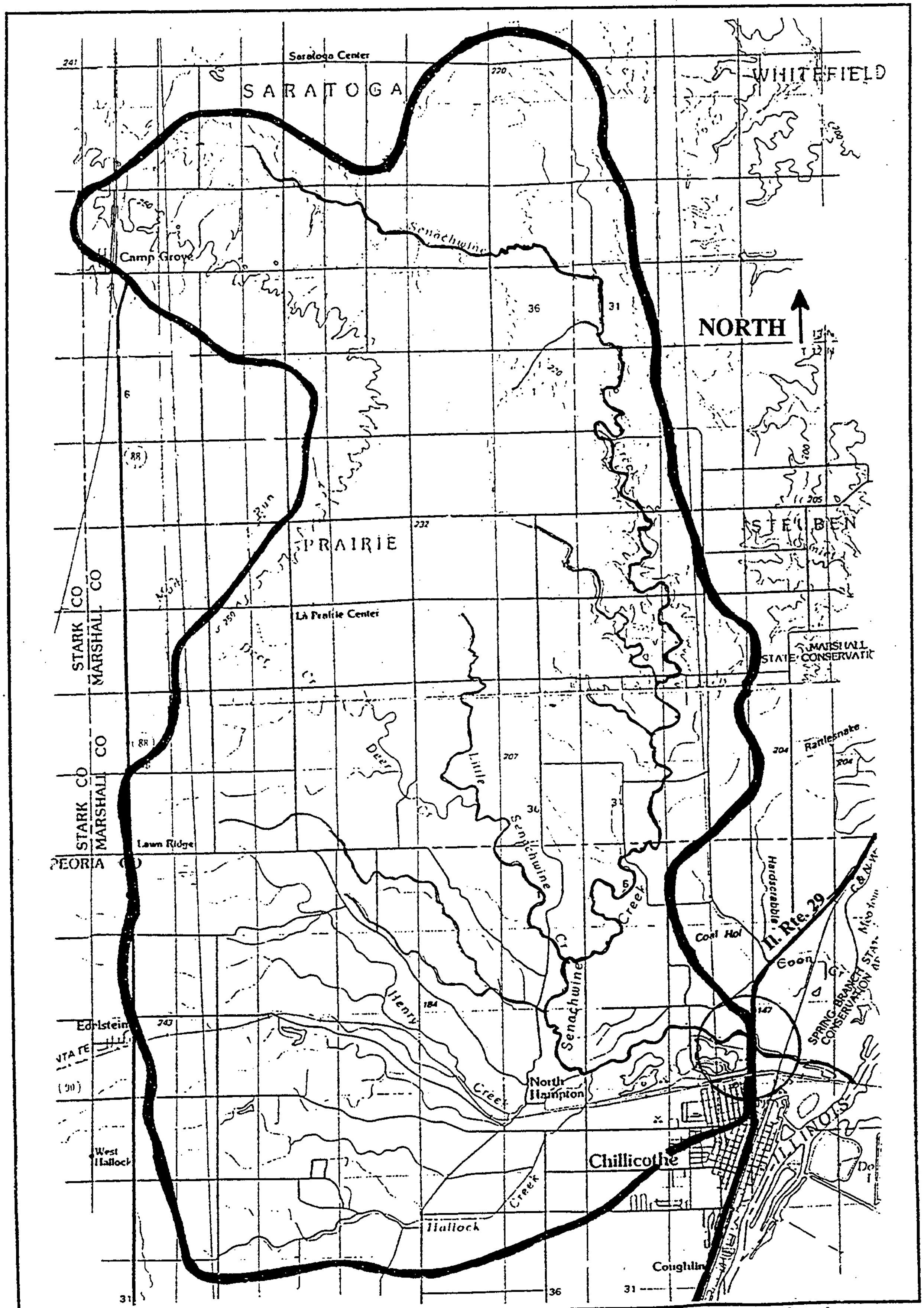

Figure 4-20. Outline of Senachwine Creek case study site drainage basin.

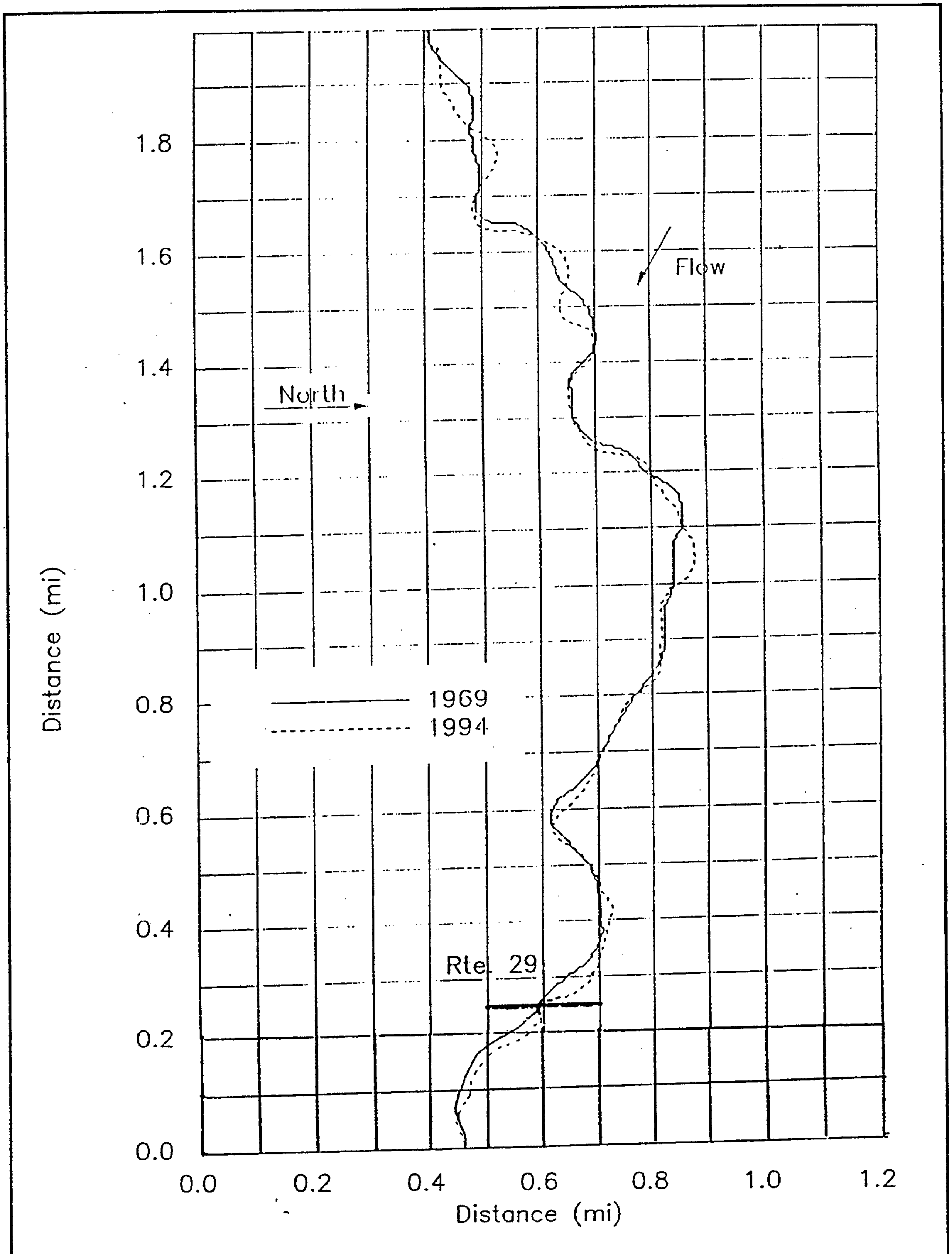

Figure 4-21. Orientation of Senachwine Creek in vicinity of study area.

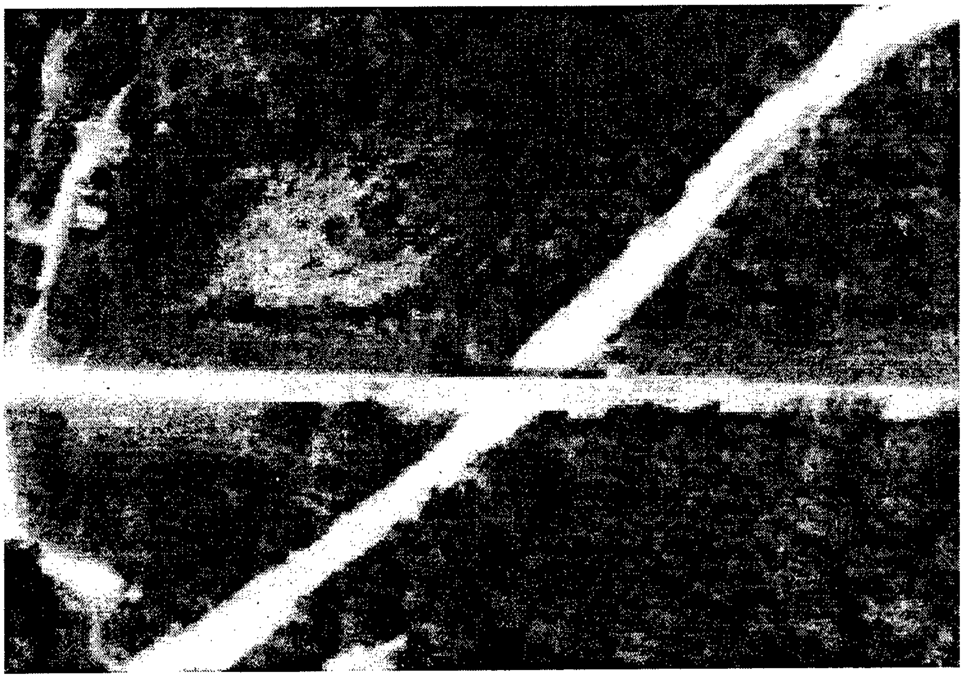

Figure 4-22. Aerial photograph of Senachwine Creek basin - 1969.

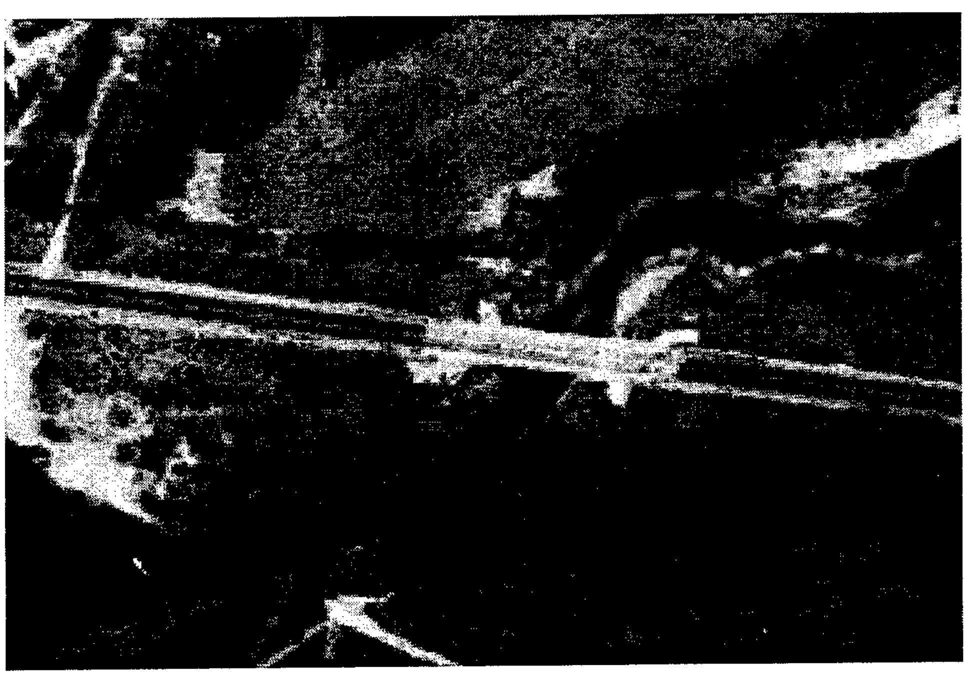

Figure 4-23. Aerial photograph of Senachwine Creek basin - 1994.

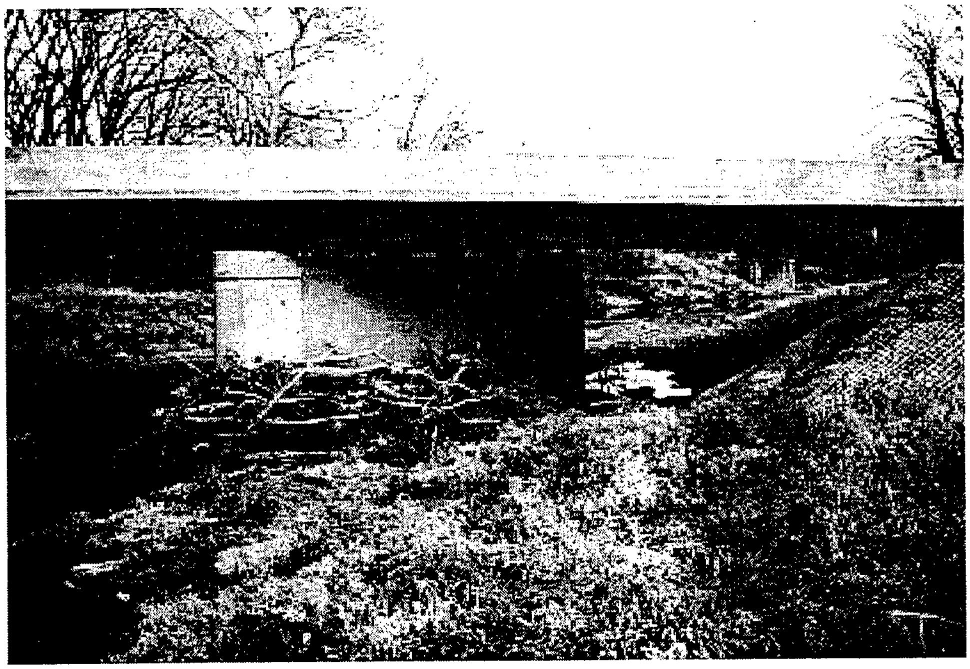

Figure 4-24. Senachwine Creek site - taken during February 26, 1996 site visit.

Figure 4-25. Senachwine Creek site - taken during April 4, 1997 site visit.

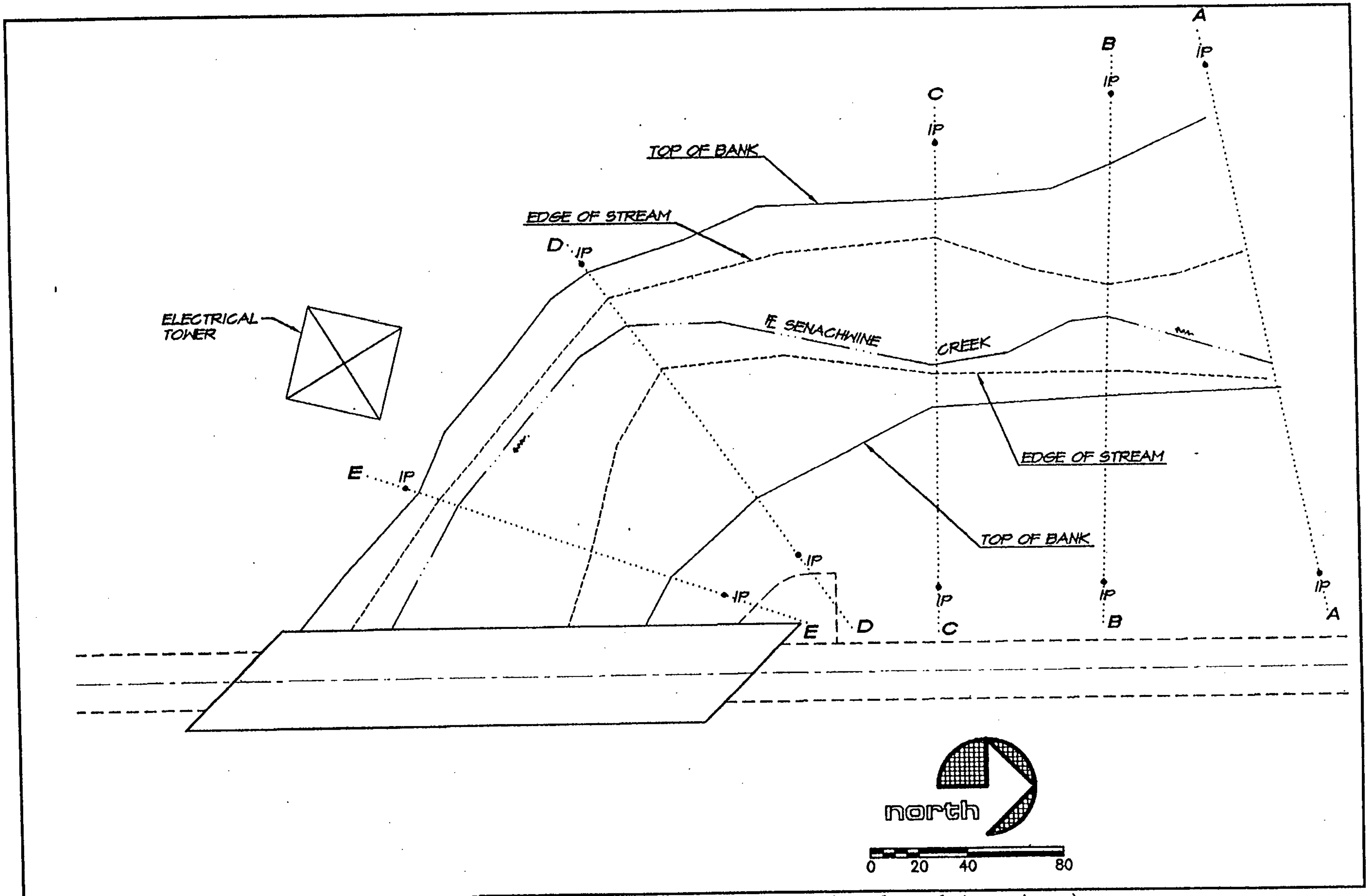

Figure 4-26. Plan survey of Senachwine Creek, July 1996. (Source: Upchurch and Associates)

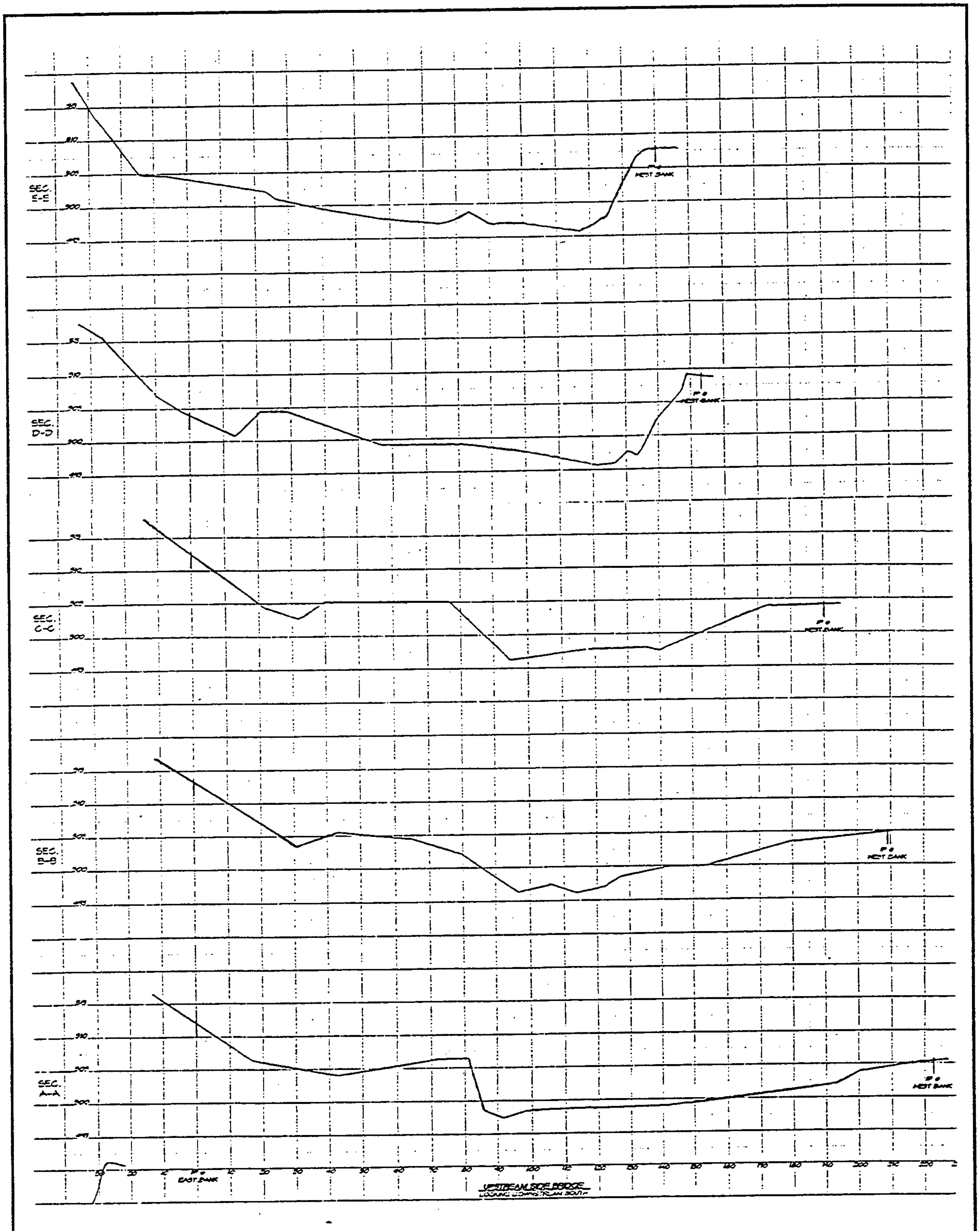

Figure 4-27. Cross-sectional survey of Senachwine Creek, July 1996. (Source: Upchurch Associates)

Spoon River

The Spoon River begins at its western and eastern forks in the vicinity of the Stark-Bureau County line. The forks converge to form the river just north of the town of Modena. From here, the river takes a circuitous path through Stark, Peoria, Knox, and Fulton Counties where it empties into the Illinois River just upstream from Havana. The site to be investigated is immediately upstream of the Route 17 bridge (#088-0002) near Wyoming in Stark County (Figure 4-28).

Hydrologically, the site drains approximately 203.4 square miles with a relatively gradual average slope of 3.9 feet per mile. The basin has been outlined in Figure 4-29, and Figure 4-30 shows the time lapsed orientation of the Spoon River case study site. Table 4-7 summarizes basin soils.

Table 4-7. Major soil associations within Spoon River site drainage basin.

Soil Association	Brief Description	K[1]
Tama-Muscatine	• Moderately well to well drained • Moderately permeable • Gently sloping to very steep	0.34
Tama -Ipava	• Moderately well drained • Moderately permeable • Nearly level to moderately sloping	0.34
Rozetta-Hickory	• Moderately well to well drained • Moderately permeable • Gently sloping to very steep	0.36
Ipava-Sable	• Poorly drained • Moderately permeable • Nearly level	0.28
Elburn-Plano	• Somewhat poorly to well drained • Moderately permeable • Nearly level to gently sloping	0.35
Catlin-Flanagan	• Moderately well to poorly drained • Moderately permeable • Nearly level	0.37
Lenzburg-Rapatee	• Well drained • Moderately slowly to slowly permeable • Gently sloping to very steep	0.30

1. The erosion factor, K, indicates the susceptibility of a soil to sheet and rill erosion by water. Values of K range from 0.05 to 0.69; the higher the value, the more susceptible the soil is to erosion by water. For purposes of this table, K is the arithmetic mean of the erosion factors across the soil association.

Flood magnitudes and frequencies for the Spoon River at Route 3 have been calculated using the USGS estimating techniques for rural Illinois Stream (Curtis, 1987). For more information concerning this technique and its accuracy, see Appendix III. The resulting flood data are presented in Table 4-8.

Table 4-8. Estimate of flood frequency and magnitude at Spoon River site.

Flood Frequency (yr.)	Flood Magnitudes (cfs)
2	2,790
5	4,720
10	6,070
25	7,850
50	9,140
100	10,430

Inspection of the aerial photographs (Figure 4-31 and 4-32) show the evolution of Spoon River. Land use within the basin appears to have changed little over the past 35 years.

The land surrounding the Route 17 site is used for farming. Active farm plots encroach along the banks of the meander just upstream of the Route 17 bridge. Some bank locations exhibit tree, grass, and brush vegetation. Along the outside of the meander, the banks are very steep to nearly vertical. Soil on-site is Huntsville Silt Loam. Huntsville Silt Loam is a black to dark gray soil. It is moderately well drained with slow run-off, and flooding occurs occasionally. An average erosion factor of 0.36 (K scale 0.05-0.69) indicates a moderately high susceptibility to erosion. Figure 4-33 graphically portrays trends in estimated annual sediment yield for the site. These data have been obtained by correlating sediment transport data for a nearby and similar Spoon River site (Demissie,1992) to the case study site through the use of a simple drainage basin area ratio. The extent of geomorphological change on-site can be directly related to the amount of sediment transport experienced. Here, it is possible to see that wide variations in transport are not uncommon depending upon the number and severity of high flow events that occur. The basin is undergoing severe erosion; in fact, the Spoon River, while only a minor source of water, is the largest contributor of sediment to the Illinois River (Demissie, 1992). Figure 4-34, a photograph taken during the February 26, 1996 site visit, depicts the site.

Upon choosing the site for further study, a detailed survey was completed. Figures 4-35 and 4-36 summarize the survey efforts. The plan view illustrates the meander encroaching on the Route 17 bridge. The steep banks, which are characteristic of this site, are easily identified in sections A-A and B-B.

While the meander in the vicinity of the Route 17 bridge appears to be moving slowly, there is an unmistakable motion toward the bridge. Averting further erosion before the meander becomes a major threat is necessary. The banks on the bend are steep and are being reshaped by mass failure and toe undercutting.

This site is promising for a low cost bank protection solution. The meander has not yet been altered severely by human intervention, and the Spoon River at this site is small enough to allow consideration for low cost stabilization measures.

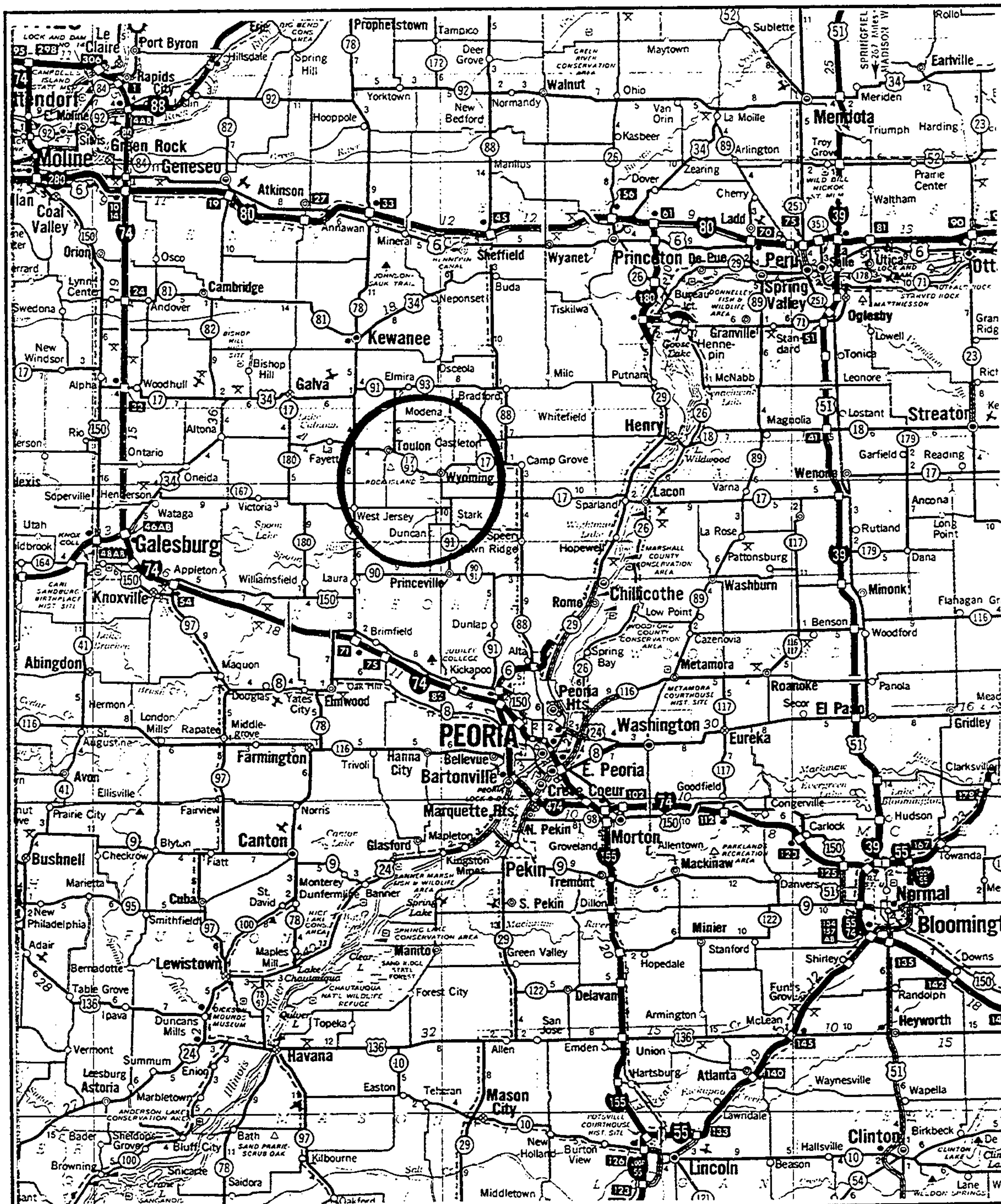

Figure 4-28. Location map of Spoon River site.

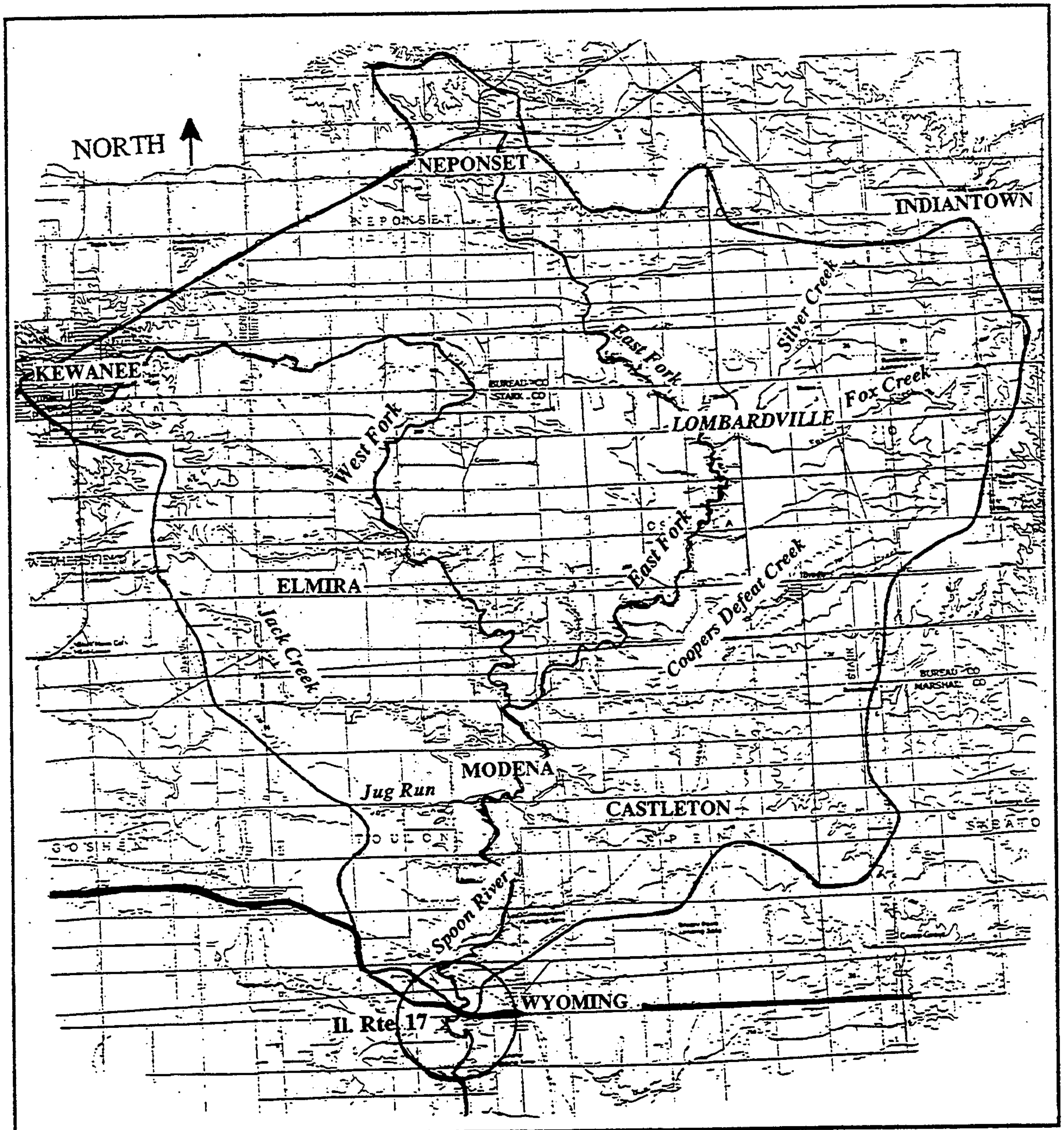

Figure 4-29. Outline of Spoon River case study site drainage basin.

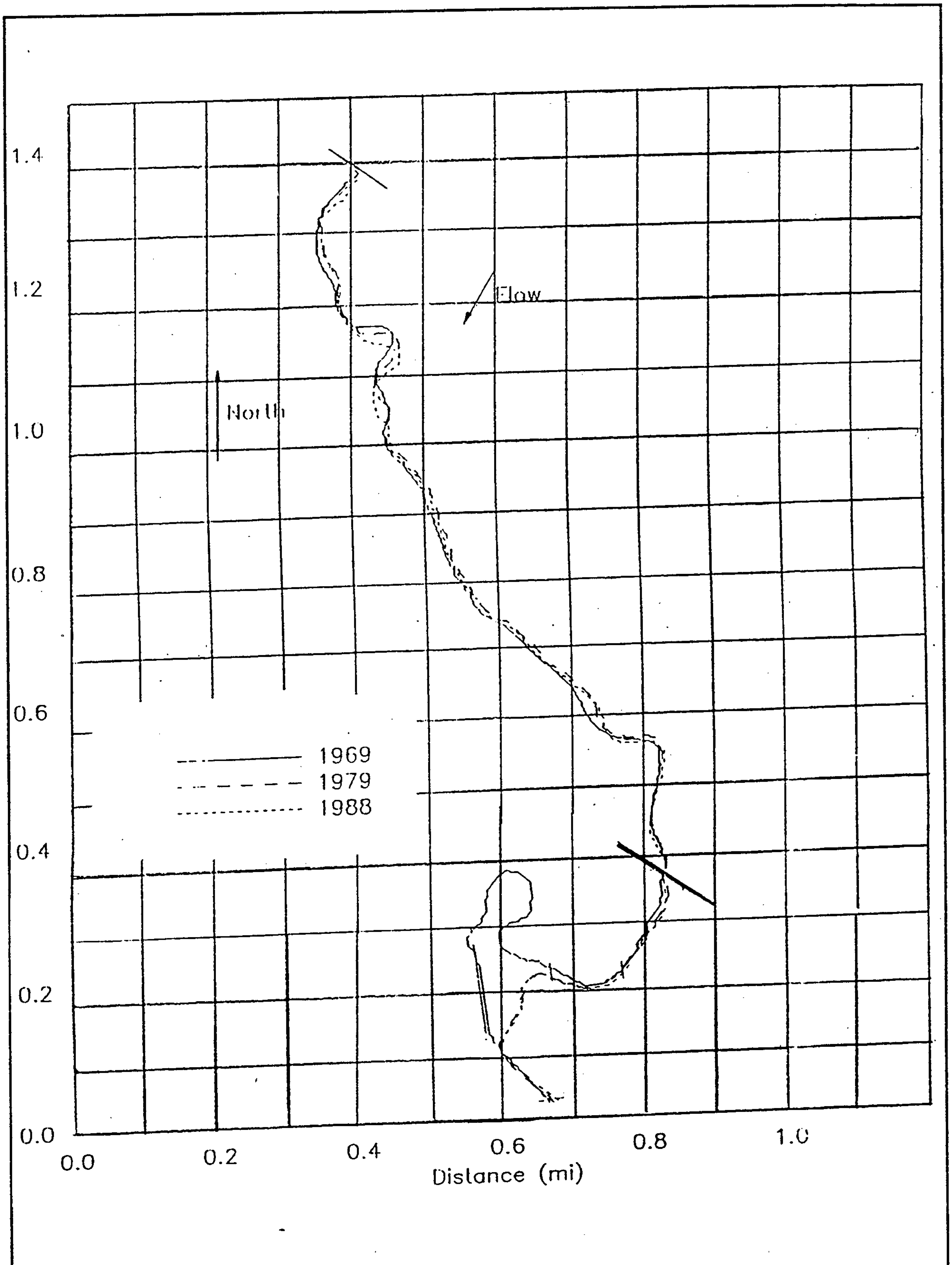

Figure 4-30. Orientation of Spoon River in vicinity of study area.

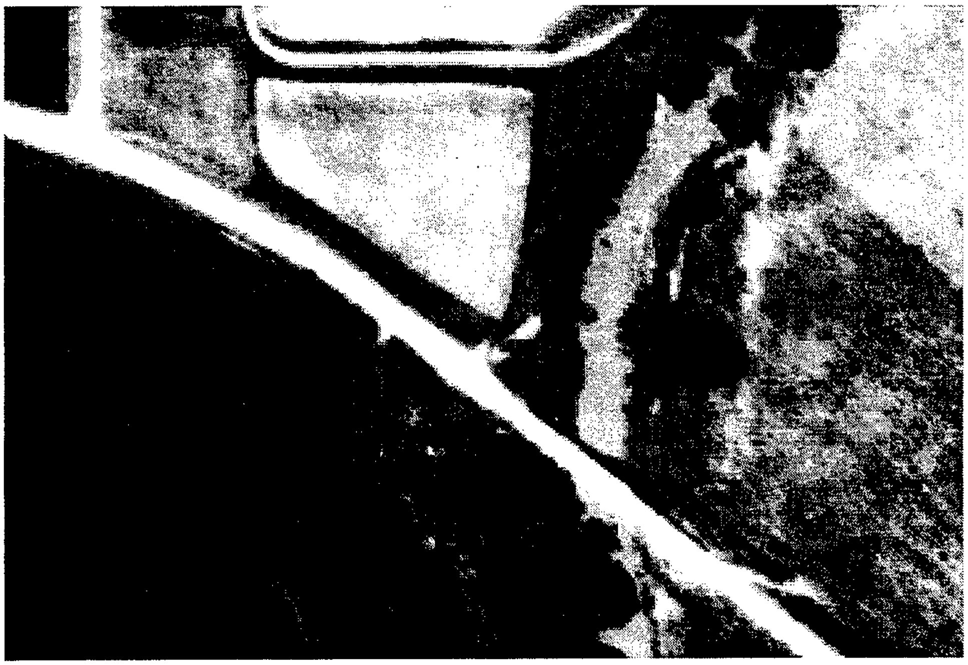

Figure 4-31. Aerial photograph of Spoon River basin - 1969.

Figure 4-32. Aerial photograph of Spoon River basin - 1988.

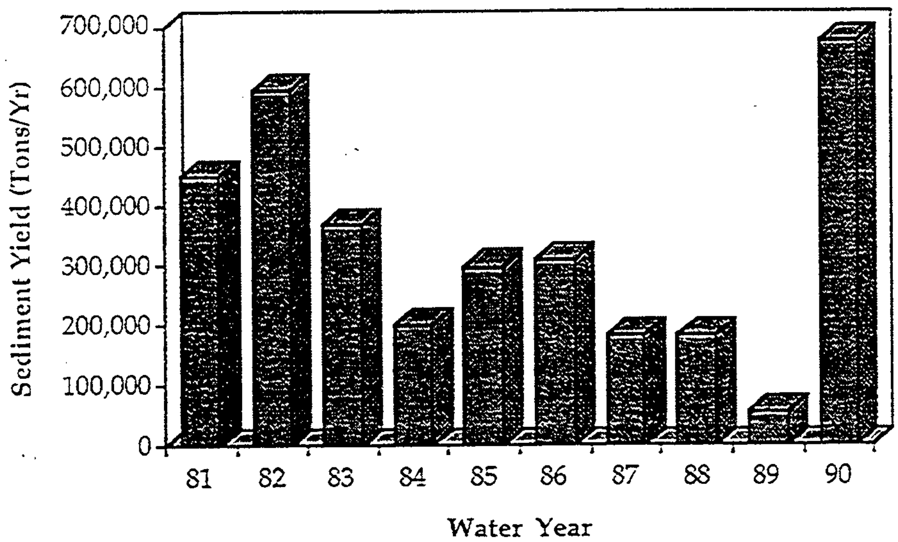

Figure 4-33. Estimated annual sediment yield at Spoon River site.

Figure 4-34. Spoon River site - taken during February 26, 1996 site visit.

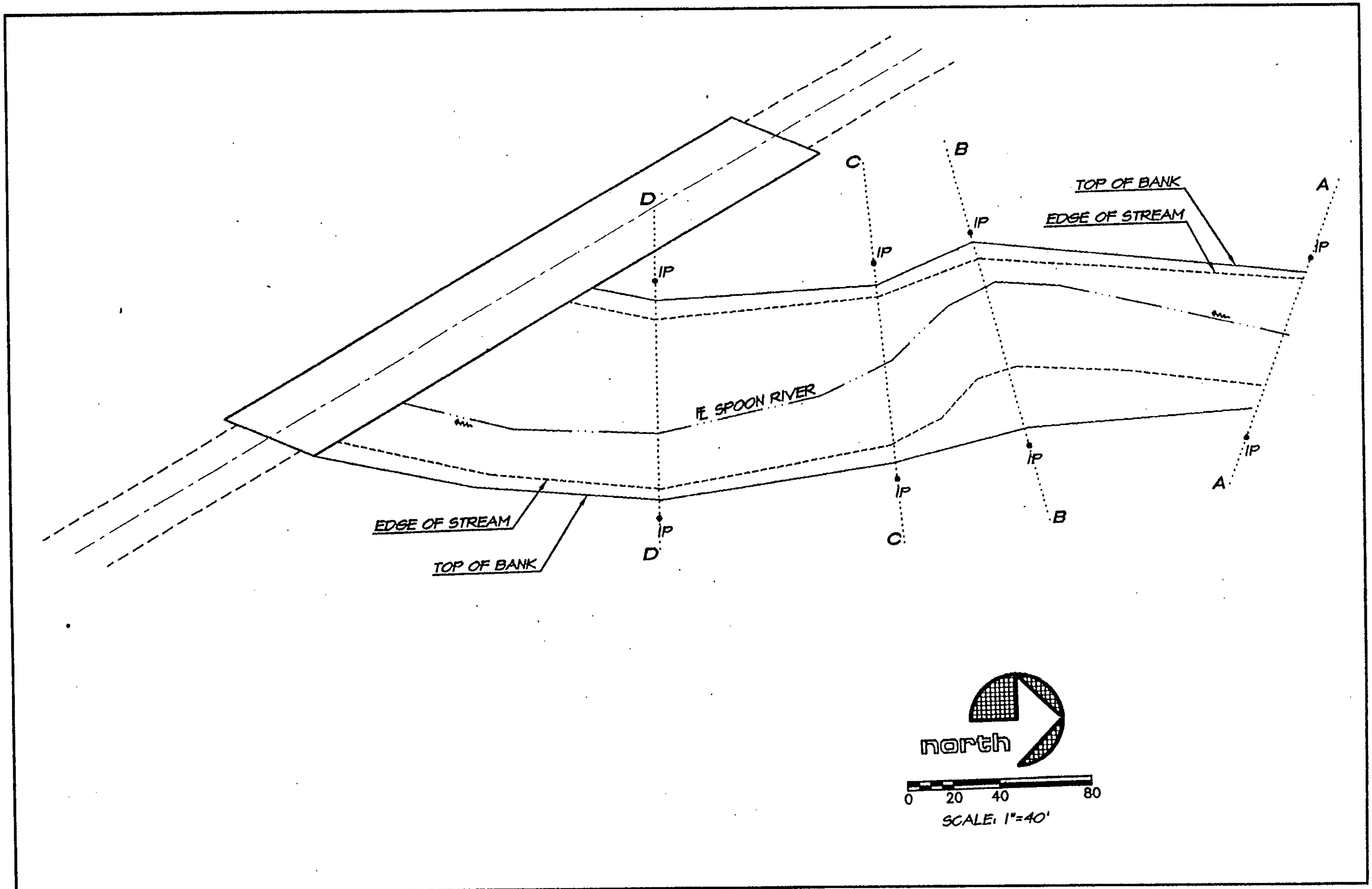

Figure 4-35. Plan survey of Spoon River, July 1996. (Source: Upchurch and Associates)

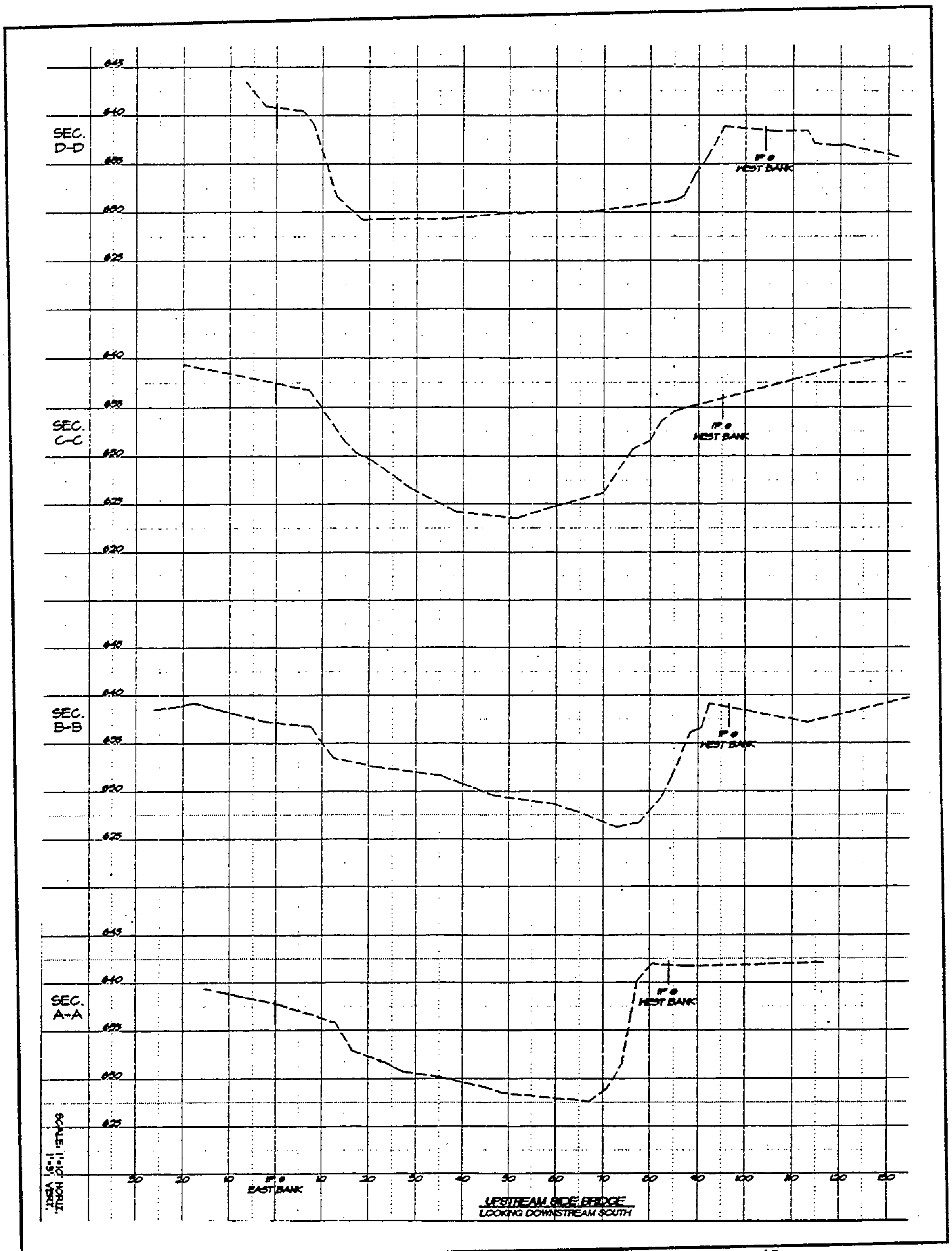

Figure 4-36. Cross-Sectional survey of Spoon River, July 1996. (Source: Upchurch and Associates)

5. Hydrologic and Hydraulic Considerations of Channel Migration

Introduction

Streams are dynamic systems in both temporal and mechanic senses. Migration of meanders within a river valley is in fact a part of the natural stream channel morphologic process. Control of channel migration would be simple if the flow were constant in time, the channel were of uniform cross-section, and the sediment were uniform in size and spherical in shape. However, the contrary is true. Meander migration is the result of a combination of complicated hydrologic and hydraulic factors.

Any built structure in the channel, including bridges and culverts, essentially imposes a control on the channel that prevents a natural migration process. The consequence is a change in deposition or erosion of sediment upstream and downstream of the structure. Associated with this is a corresponding channel pattern adjustment. This effect occurs mostly during high flows; during low flow events, little morphological change occurs. For a relatively flat land like Illinois, the bridge or culvert essentially acts as an obstacle producing flood ponding and backwater effects upstream. Lowering the flow velocity upstream due to ponding will result in sediment deposition. On the other hand, the flow downstream of the structure often has relatively high velocity resulting in downstream erosion. The basic cause of channel migration, with or without built structures, is the dynamic fluvial process. Sediment erosion, deposition, and transport is but one consequence of this cause. Thus, if one wishes to preserve the geomorphologic conditions near the structure, one must first determine the significant floods forming the fluvial pattern. Then, the backwater effects of these floods must be minimized while developing a means to maintain the channel pattern immediately upstream without hindering the transport of sediment downstream.

Undoubtedly, there are various ecological, environmental, economic, social, and other non-technical implications of the fluvial process in either natural channels or channels with human imposed structures. Nevertheless, the essential element in solving the problem of channel migration impact on bridge approaches and conveyance is a clear understanding of the fluvial hydraulics and stochastic surface hydrology combined with proper and skillful application of the theories and procedures available. While sediment erosion and deposition in the vicinity of a bridge may appear to be a local problem, a long-term efficient solution can only be achieved by a holistic approach considering the hydrology and hydraulics of the fluvial process of a large portion, if not the whole, of the stream.

Hydrology

Stream flow has its annual, seasonal, daily, and flood event variations. The most commonly considered annual variations are the frequencies of extremal floods and droughts. These extremal floods are important contributing factors to the channel morphology. It is well known that bankfull-stage floods, which have a return period (average recurrence interval) usually between 2 and 3 years are the most important flows in shaping the channel morphology. Slightly bigger floods, those having a return period of 10 to 100 years, are most often responsible for meander cutoffs. Such floods carry high flow long enough to attack the banks at the meander bend neck and to soften the soil there. Spatially, for these floods, the immediate effect on channel shape and position is of the scale on the order of 0.1 miles. Extraordinary floods, roughly those with frequency about 5000 years and over, have the potential to reshape the landscape, restarting channel geomorphologic process (Baker, 1973). In these cases, effect of the event is on the spatial scale of miles and higher. For bridge protection as considered in this project, floods with return periods less than 100 years are usually considered. The flood frequency of Cahokia creek at Edwardsville, Illinois based on the annual maximum series of 1969-95 is shown in Figure 5-1 as an example.

Conversely, smaller floods, having a frequency of several times a year, are significant in forming the local geomorphologic details within a channel cross section on scale about the width of the channel or less. After the geomorphologic reshaping of the channel by a major flood, usually with a return period exceeding 10 years, the more frequent small floods modify the local sand bars and cross sections through local erosion, sediment transport, and erosion by the flow. Occurrence of major floods as well as smaller floods are probabilistic. Therefore, the local channel change is a chaotic process, and the channel geomorphologic process is stochastic. Theories to evaluate such stochastic channel morphology process for bridge protection exist, but their adoption is beyond the scope of this project.

As an example of the stochastic nature of floods acting on a stream, the variation of flow of Cahokia Creek at USGS Edwardsville gauging station is shown in Figure 5-2, and the flow duration curve (exceedance probability curve) is given in Figure 5-3. The data used are the USGS average daily flow data covering the period from August 1969 to September 1993. In Figure 5-2, each bar represents the average discharge over a 5-day period averaged over 24 years of record. For the 6 leap years during this period, the day of February 29 are merged with those for the 5 days of 2/25-3/1 and the sum of the flow is divided by 126 instead of 120 as for a regular 5-day period. A flow duration curve (Figure 5-3) would allow a pure probabilistic estimation of the stochastic process of channel migration whereas an average hydrograph like Figure 5-2 would permit a stochastic estimation.

Hydraulics

In channel migration control for bridge protection, there are two aspects of hydraulics worth special attention. The first is the characteristics of flow in a meandering channel bend. The second is the hydraulic action of the flow with a discharge different from the design flow, particularly for larger discharges.

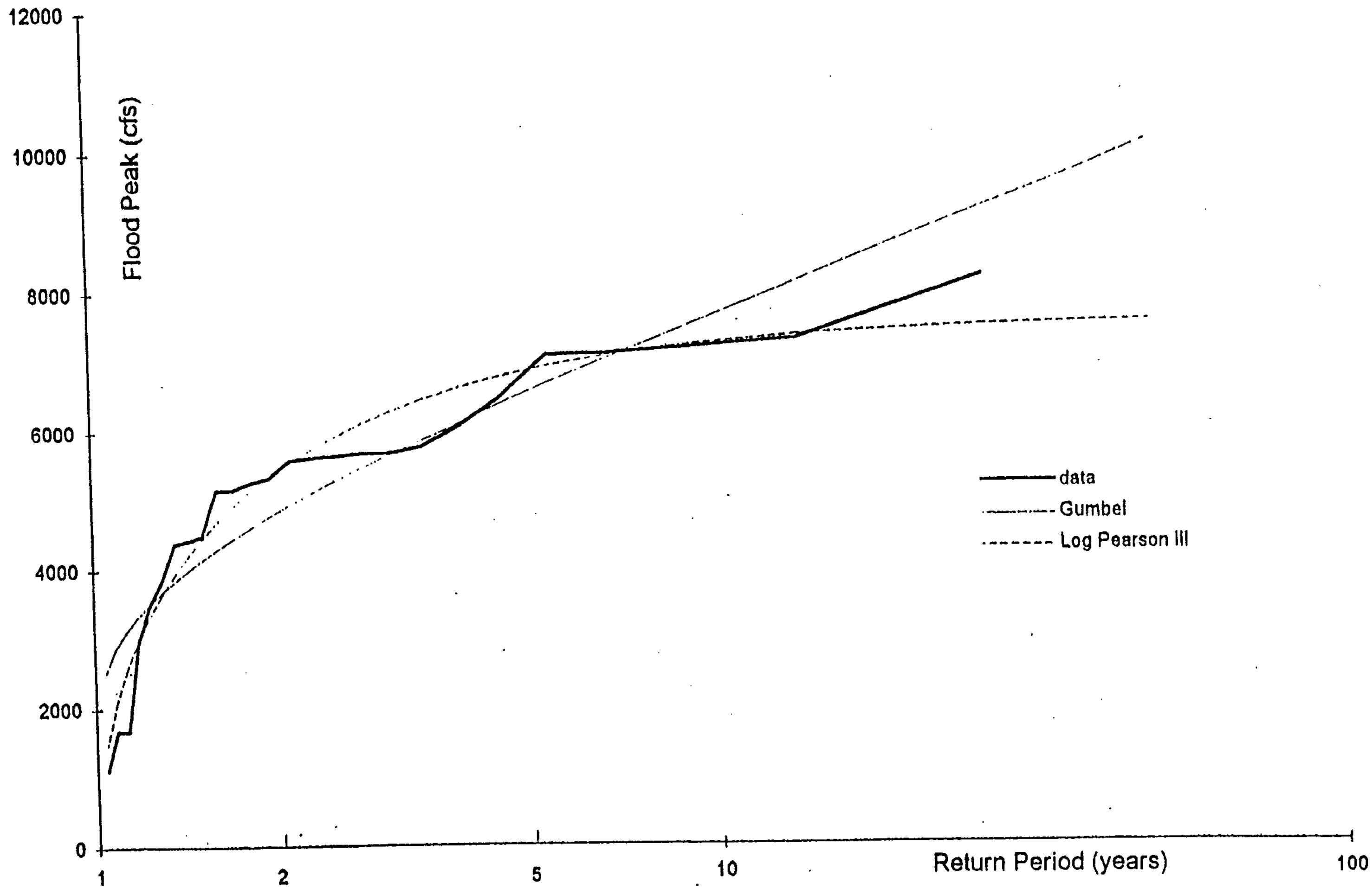

Figure 5-1. Example flood frequency curve for Cahokia Creek at Edwardsville, Illinois.

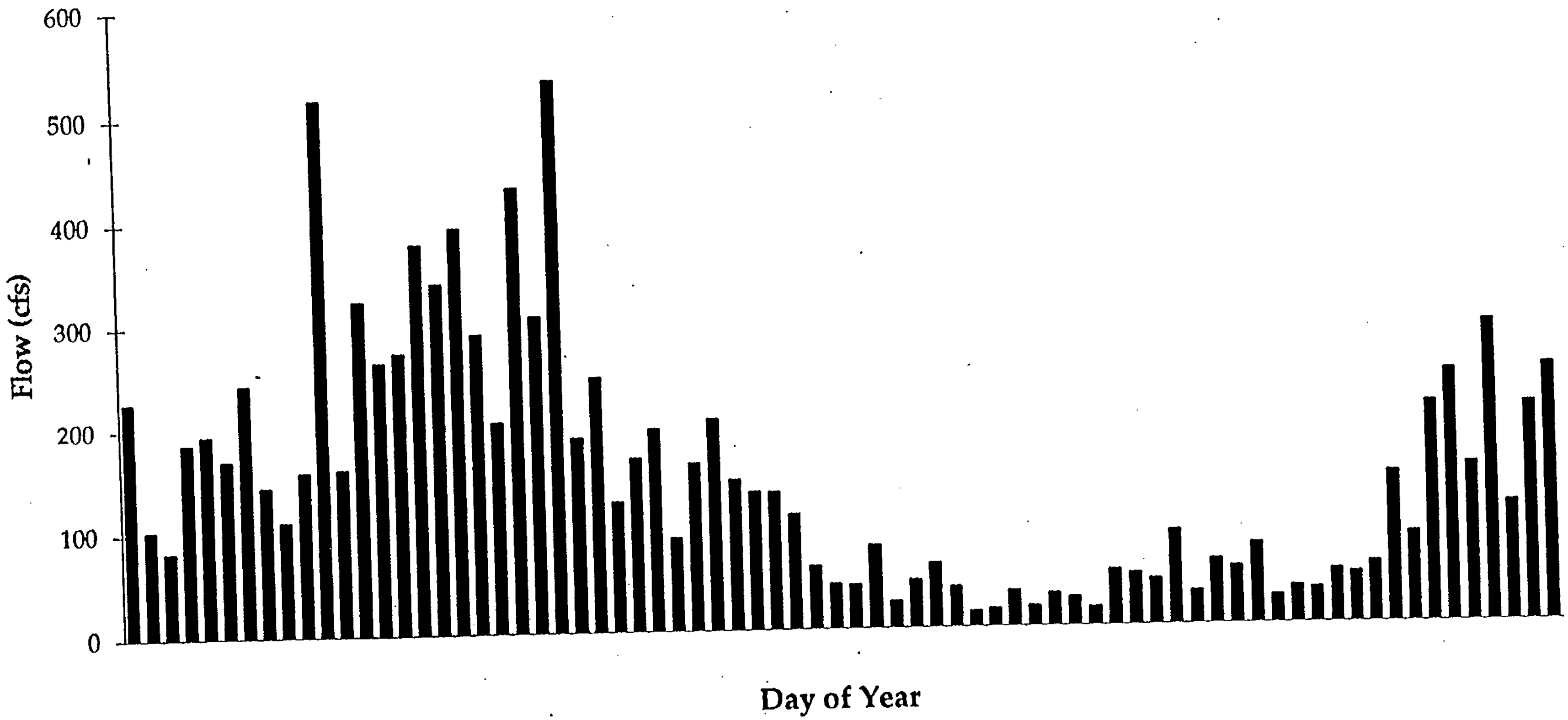

Figure 5-2. Expected annual variation of 5-day average flow for Cahokia Creek at Edwardsville, Illinois.

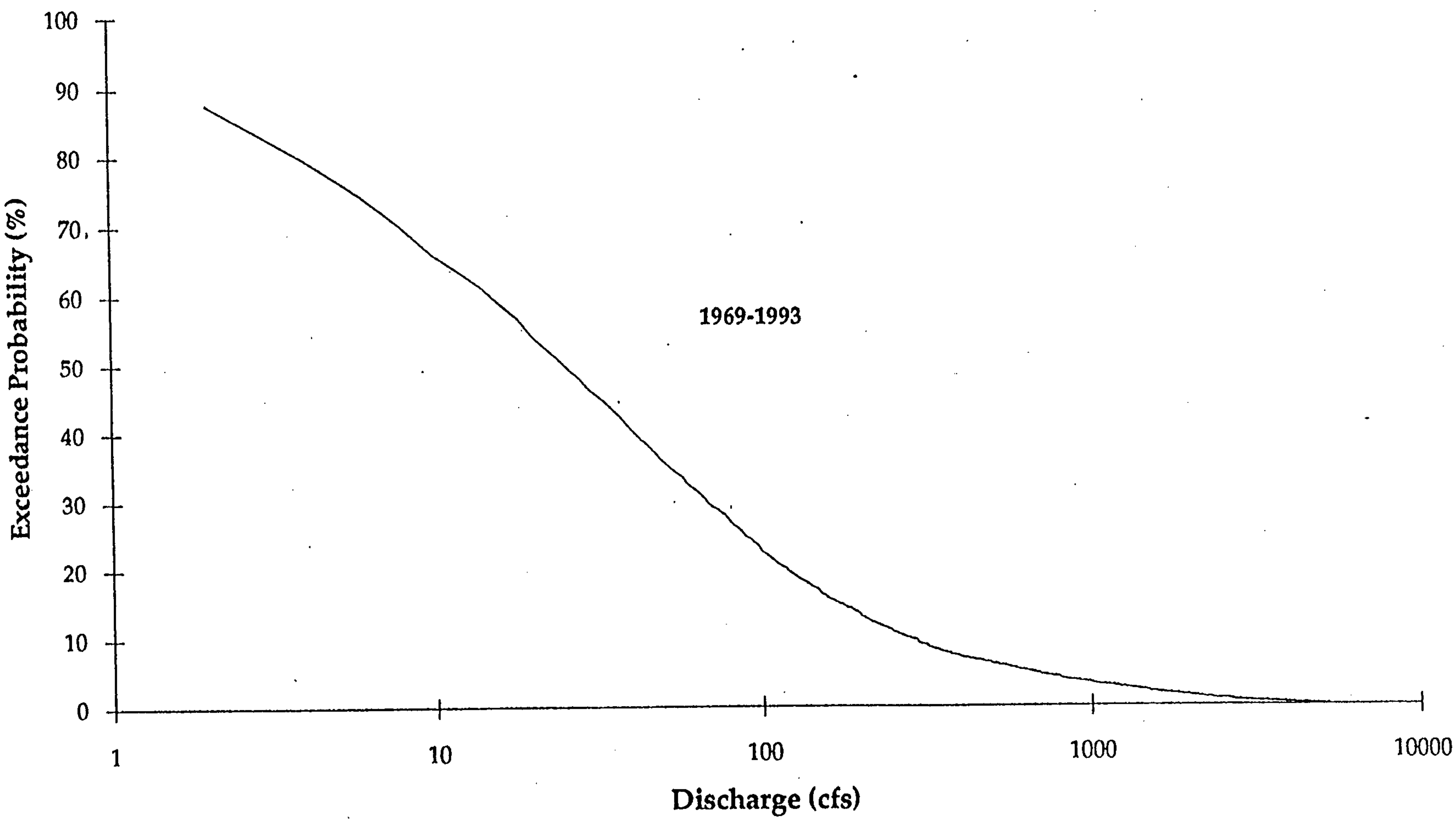

Figure 5-3. Flow duration curve for Cahokia Creek at Edwardsville, Illinois.

Compared with flow in a straight channel, flow in a channel bend has four major characteristics (Rozovskii, 1962; Yen, 1965,1967): (a) superelevation of water surface towards outer bank; (b) existence of spiral trajectory of the flow in the bend; (c) lateral discharge moving water towards outer bank with the bend; and (d) steep bank slopes and deep pools along the outer bank in the downstream part of the bend. Characteristic (b) and (d) are particularly important in controlling meander migration (Yen, 1975). As shown in Figure 5-4, the spiral motion enhances erosion along the outer bank and sediment deposition and sand bar formation on the inner bank. Some counter measures such as bendway weirs and Iowa vanes are constructed to modify or inhibit the occurrence of spiral motion near the outer bank. Understanding the strength of spiral motion and the angle of attack of the flow near the bank is important for effective control of channel migration. The erosion rate and speed of meander migration is also affected by the ability of the flow to move the eroded sediment from the outbank to the opposite bank.

Principles of sediment transport govern the occurrence of erosion and deposition within a channel. Among the different methods which exist to predict sediment movement, The Sheilds Curve, Figure 5-5 (ASCE, 1975), is the one most often applied. The curve represents a probabilistic mean threshold for sediment transport based on two dimensionless parameters. The abscissa is in terms of the boundary layer Reynolds Number which is a function of the flow conditions (shear velocity and viscosity) and the sediment particle diameter. The ordinate is in terms of the dimensionless Shields Stress defined by the bed shear stress divide by the product of the submerged specific weight and sediment diameter. Physical situations which produce a point above the Shields Curve indicate in a statistical mean sense the ability of the flow to transport sediment. Points below the curve result in no sediment transport.

Diagram's like Shields' is useful in evaluating the erodibility of soil or riprap on a flat bed. For a meandering channel, bank erosion is also a function of bank slope and the spiral motion shown in Figure 5-4. Along the outer (concave) banks, near the exit of a bend, the shear stress is high and the downward direction of the spiral motion assists the bank soil to move, enhancing the erosion (Yen, 1995). The strength of the spiral motion is a function of the curvature of the bend and the discharge in the channel. Along the inner (convex) bank, around the entrance to the bend, the depth is small and the spiral motion is weak. Sediment eroded from upstream is carried to this region and deposited in this region, and the point bar grows. Small floods may not be able to transport the majority of the eroded sediment at the outer bank near the bend exit and move it across the channel to the outside of the opposite bank; hence, for such floods, bends tend to grow sharper and bigger. For large floods, eroded sediment are mostly carried across the channel and move downstream. For such floods, there tends to be more migration downstream than growth in bend. Figure 4-3 shows a typical growth and migration of a meander. For bridge protection, it is important to control the meander at the bridge site, and understanding of this hydro-geomorphologic factor is important.

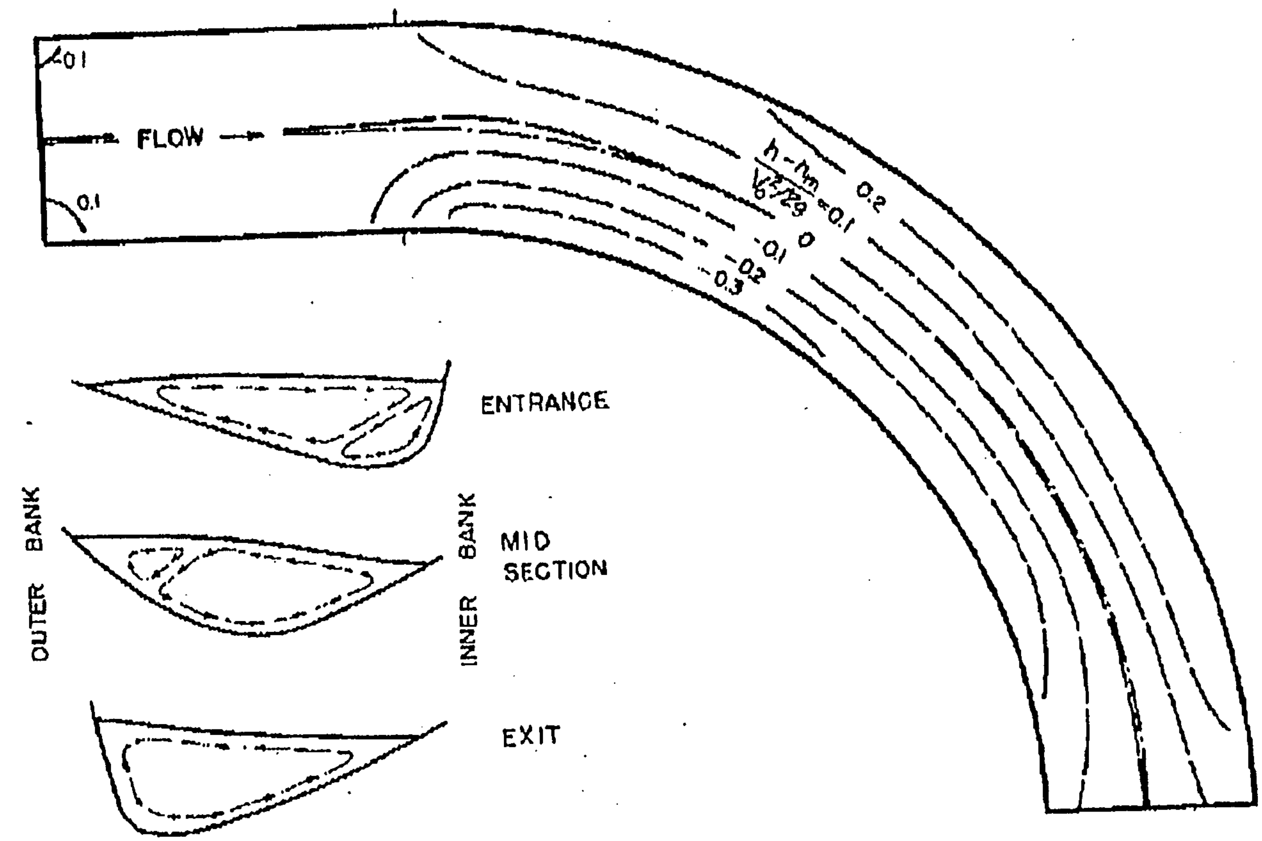

Figure 5-4. Example drawing of spiral motion in three cross sections of a bend and relative water surface configuration.

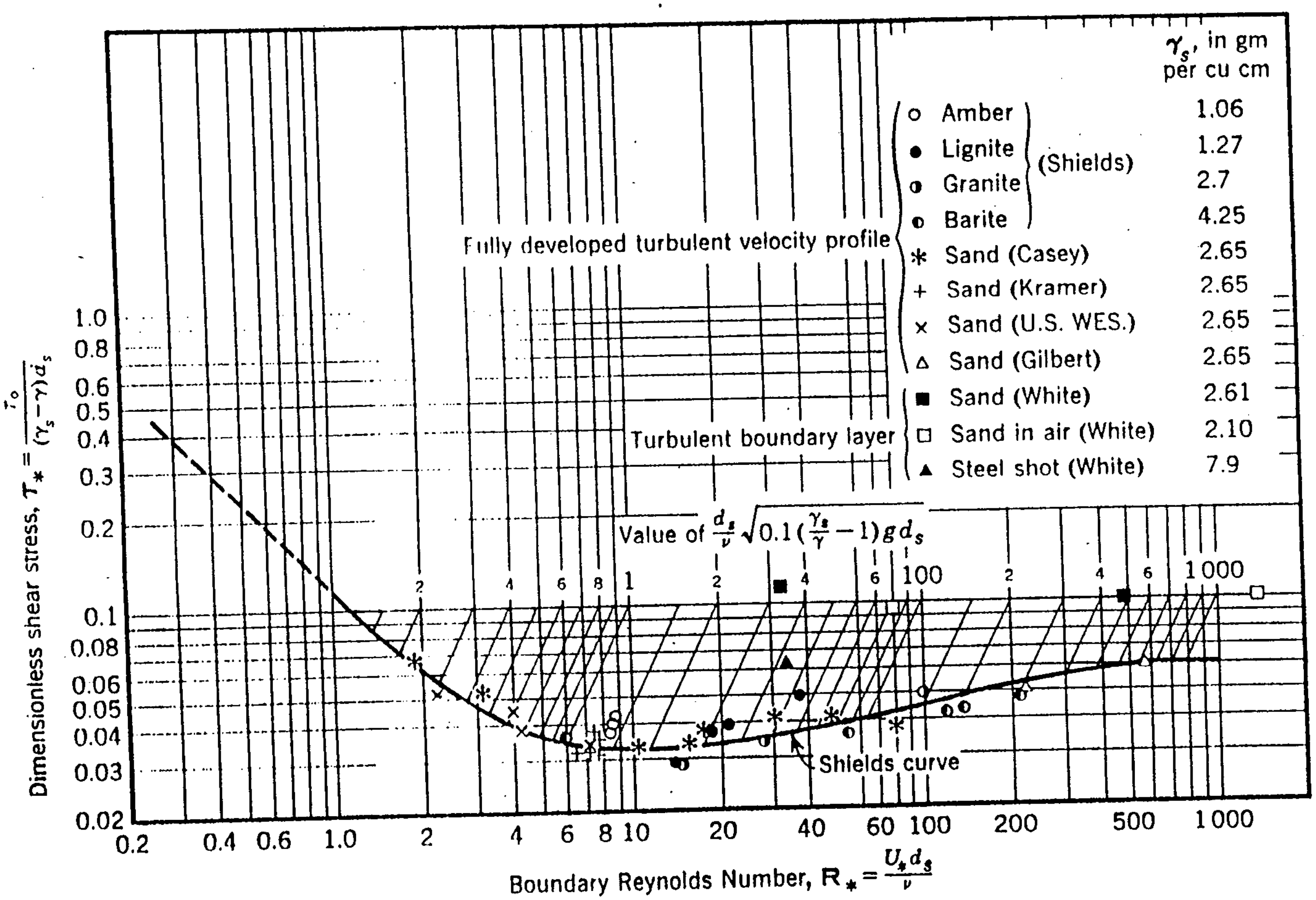

Figure 5-5. Shields diagram. (Source: ASCE, 1975)

Figures 5-1 to 5-3 highlights the variable nature of channel discharge as it depends upon hydrology. One must keep this in mind when considering the hydraulic behavior of the control measures at different possible discharges. For example, a downstream facing spur dike will guide the flow to the desired alignment and promote sediment deposition immediately upstream near the banks. But for large floods, when the dike is totally submerged, there is the potential of severe scour at the downstream side of the dike, especially near the toe. If not properly designed, one overtopping during a large flood could totally negate the earlier gains of the control measure. In general, stream channel alignment control requires the use of instream structures such as spur dikes and bendway weirs. Revetments and various bioengineering application to banks, such as willow posts and grass or brush growth, are mostly for bank protection.

Technology now exists to combine hydrology and hydraulic considerations for optimal design of channel migration control. It requires application of a deterministic sediment-laden channel simulation model such as that of Garcia et al. (1994) to the probabilistic hydrology of the streamflow together with cost information to find the least total cost alternative. Implementation of this integrated idea to an actual site could be a useful development project in the future.

Risk Considerations

For a given bridge or culvert site that requires channel control and erosion protection, usually several protection and control alternatives exist. These alternatives range from an "active" to "passive" approaches. An active approach included the building and laying of designed structures, refilling scour holes, and positively guiding the flow and setting the meander alignment. A passive approach includes planting, laying simple obstacles to the flow at strategic locations, and expecting the flow to gradually improve the channel alignment through the abatement of erosion and positive sediment deposition. Generally, the active approach is more costly but has a higher chance of success; whereas, the passive approach is less costly but has a higher chance of failure. In other words, usually the control measures associated with active approach can withstand larger floods of higher return period while the measures associated with passive approach may fail under the attack of smaller, more frequent floods. This is particularly true if the passive counter measure involves bio-growth. Initially, the effectiveness of the protective measure is low, and the risk is high until the growth achieves a certain stage.

In view of the stochastic nature of the floods one should recognize that any stream channel protection measures, no matter how careful they are designed and constructed, are always under the risk of failure. One would be lucky if the flood with the magnitude of design level does not occur during the service life of the control facility. Conversely, there is the possibility that the year after a 20-year bank protection measure is completed a major flood of the magnitude of 200-year return period occurs and totally destroys the protection facility. The exceedance probability, or risk, as a function of return period of a specified event and the project service life is shown in Figure 5.6.

Considering the risk and associated damage cost, the optimal design alternative is often not the alternative with the lowest initial capital cost. This phenomenon is shown graphically in Figure 5.7 (Yen, 1990). In this figure, considering all the costs, economically, the optimum design has been labeled O. Projecting upward, it is seen that the initial capital cost (C) does not correspond to the lowest initial cost alternative (the most passive approach). Also, this optimum (lowest total cost) alternative (O) results in considerably lower risk (R) than would be assumed if a lower initial cost alternative were chosen. It should be noted that the concept described in Figure 5-7 is straight forward but the quantification values of some components are not easy to determine.

As a simplified hypothetical example, consider two alternatives. For the "low cost" option, the initial capital cost is $5,000, and it can withstand a four year flood. For the "high cost" option, the initial capital cost is $50,000 against a 25 year flood. Should the project fail, the damage cost on the bridge and other associated losses is $700,000 plus the cost of rebuilding the protection measure which is assumed to be the same as the initial cost. A calculation summary for the hypothetical site over a 25 year project service life is provided in Table 5-1.

Table 5-1. Hypothetical example comparing "low" and "high" cost options considering risk cost.

	"Low Cost" Option	"High Cost" Option
Initial Capital Cost	$5,000	$50,000
Design Return Period	4 yr	25 yr
25 Year Failure Probability	0.99925	0.6396
Failure Damage Cost	$700,000 + $5,000	$700,000 + $50,000
Risk Cost	$704,470	$479,700
Total Cost	$709,470	$529,700

The probability of failure is assumed constant from year to year, and the value for a 25 year service period can be determined form Figure 5-5. The Risk Cost is the product of the Failure Probability and the total Failure Damage Cost. Here, it is seen that the "Low Cost" option results in a higher total cost over a 25-year service period and is the less desirable alternative.

In reality, it is often incorrect to assume a constant probability of failure, especially when using low cost bioengineering techniques. These counter-measure may fail if a critical flood occurs soon after the vegetation/tree is planted and before it has a chance to grow and takes a strong root. Whereas, the design would succeed if the same flood occurs much later. The stochastic nature of failure is depicted in Figure 5-8. This figure presents three hypothetical cases subject to the same total number of floods of identical magnitude but with different sequence of occurrence in a period of the same number of years. Two protection/control alternatives, one high and one low initial cost, are considered. The high initial cost option is, for simplicity, considered to have no biological

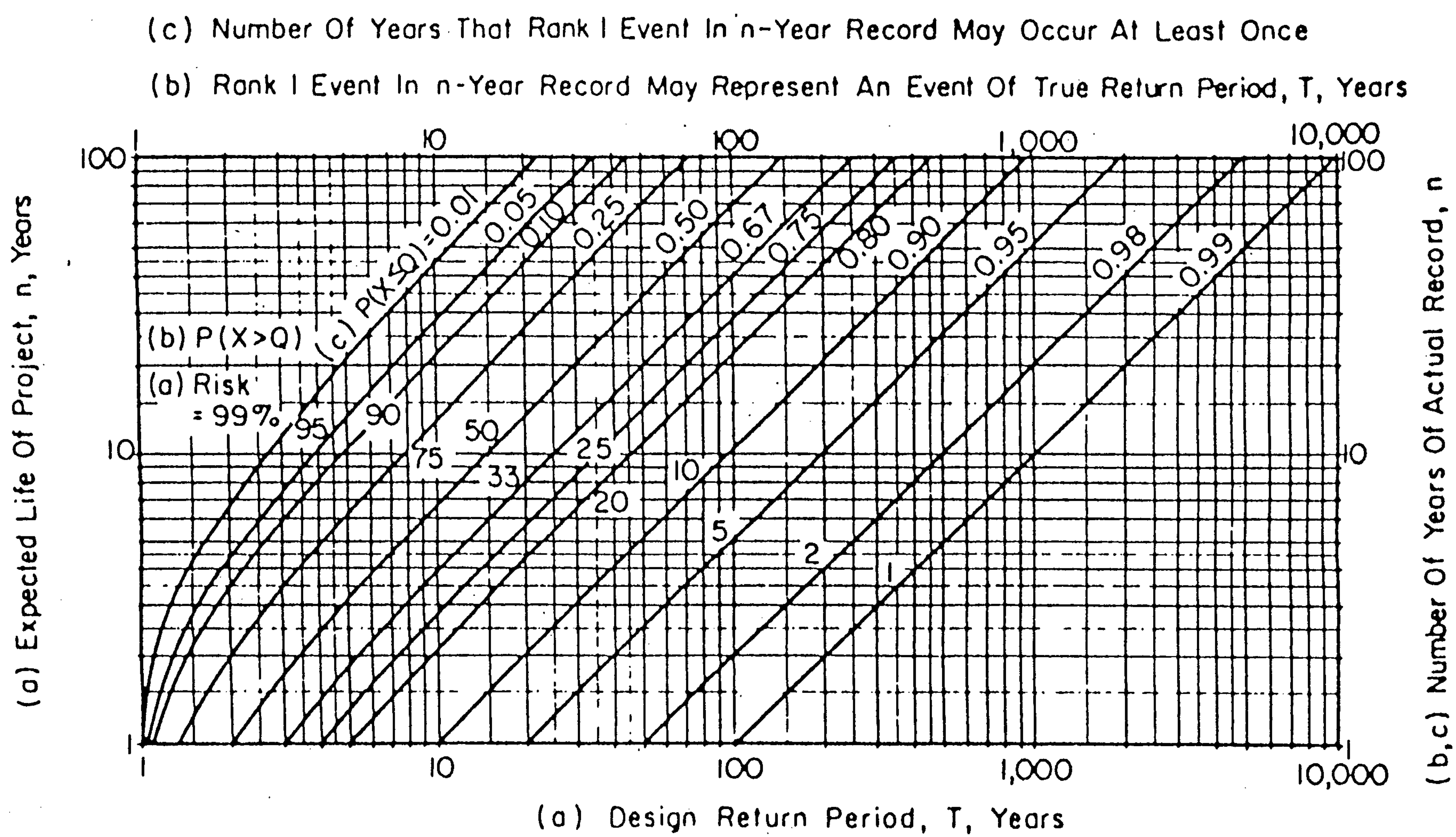

Figure 5-6. Relationship between return period, project life and risk (Yen, 1970)

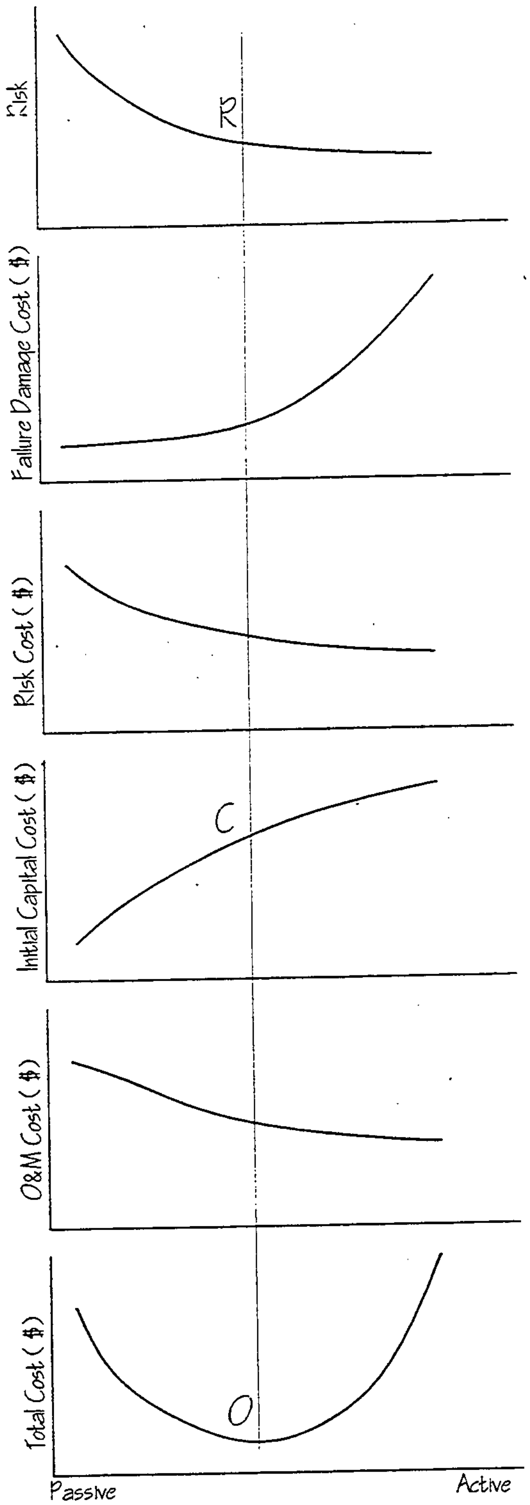

Figure 5-7. Risk based optimum design among alternatives. (After: Yen,1990)

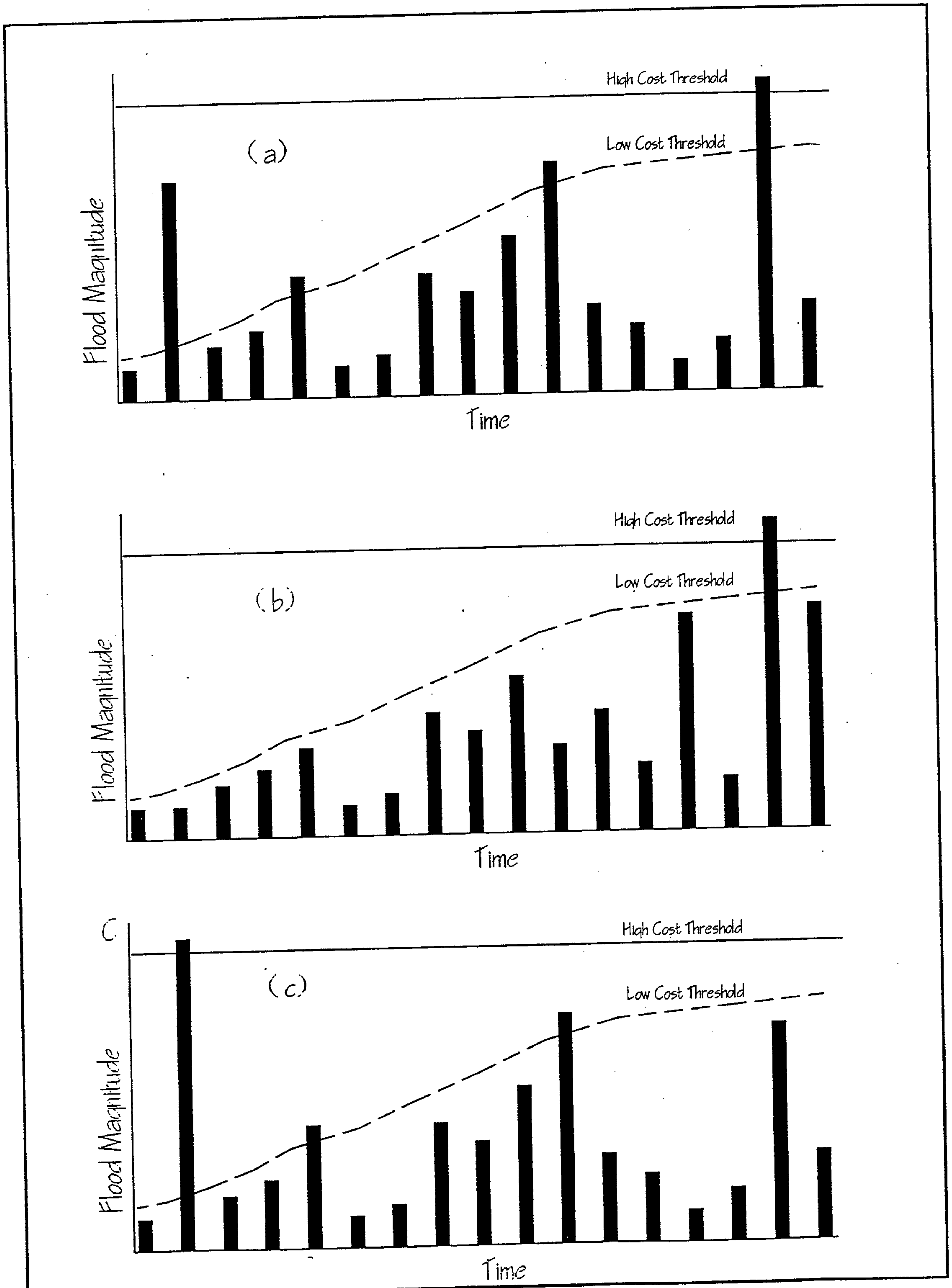

Figure 5-8. High and low cost risk thresholds and dependency of failure with flood sequence.

elements, and it is assigned a constant risk threshold. The low initial cost alternative is of the bioengineering type, and has a variable risk threshold. For case (a), a number of critical floods with magnitudes below the high-cost threshold but above the low-cost threshold occur in a row at early stages of project growth. While the high cost alternative is not affected by these floods, the low cost alternative may repeatedly fail. The second to last flood event surpasses both high and low cost alternatives; either alternative would require repair and replacement after such an event. For case (b), smaller floods occur at early stages such that the vegetation has a sufficient time for growth, and no critical floods threaten the project until the low cost alternative reaches maximum design performance. Here, either low or high initial cost alternative are equally successful. The third and least fortunate case, (c), shows the situation when a flood greater than the design flood occurs soon after counter-measure implementation. In this case, both high and low cost alternatives will fail and require repair and/or replacement.

More specific theory accounting for such risk factors for design of bank protection already exists. However, development of the theory to application procedures for channel migration control for bridge protection is not a simple task to be undertaken within the scope of the present project. It is a task worthwhile for the future.

6. Monitoring

Introduction

One may identify three distinct site monitoring levels: (1) informed site observation, (2) site survey and investigation, and (3) research and development monitoring.

While Level 1 monitoring is, by its nature, the simplest, it also is perhaps the most important step in the monitoring sequence. Level 1 requires no specialized equipment and entails only minimal, if any, extra costs. In general, Level 1 consists of local engineers and officials who are well informed as to what constitutes a problematic site. Then, these officials are able to make informed visits and site observations on a periodic basis; the time between visits can be on a set interval or dependent upon flood events. Level 1 allows for identification of problem sites. Protection measures can then be implemented earlier and at lower costs.

Level 2 monitoring requires an in-depth site survey and investigation. Specialized equipment and expertise are required, and, as a result, Level 2 costs are higher than that of level 1. Methods commonly used for Level 2 monitoring have been described at the end of Chapter 2. This type of monitoring is conducted before and after protection measures are designed and built. Prior to the installation of protective measures, Level 2 monitoring results provide information necessary for a successful design and implementation. Following the installation of bank protection measures, it is important that the site be carefully monitored. Monitoring allows for a thorough and objective evaluation of the success of the protection measures. At the same time, monitoring maintains a level of alertness which allows for the recognition of sudden changes in flow and channel characteristics which could jeopardize project stability. Many bank protection methods require periodic maintenance, and monitoring promotes the development of an organized maintenance schedule dictated by monitoring results.

Level 3 monitoring is of the research and development type. Here, a more detailed site investigation must be conducted. Research and development type monitoring might include hydraulic and hydrologic studies, detailed soil analysis, etc. Research and development monitoring may result in more innovative methods of bank protection. Of course, more innovative and unproven protection techniques require Level 3 monitoring to assess their success and applicability.

Monitoring Recommendations

In keeping with the low cost objectives of this project, monitoring must be completed in such a way that level 1 monitoring can be conducted by local highway personnel with minimal technical skills required, whereas for level 2 monitoring it should minimize both field work and data analysis time. The goal is to develop an effective surveillance to ensure safety of highway bridges. Recommended items relevant to level 1 monitoring are given in Table 6-1. The observations are mostly to be done visually, with the aid of local landmarks or simple erosion pins if necessary. Sedimentological observations, in terms of scour and degradation,

Table 6-1. Low cost monitoring for small streams stability.

Site Problem	Stream Type (Straight, Bend)	Monitoring Priorities		Observable Changes		
		After Flood	Annual or periodic	Sedimentological	Bank condition	Bed condition
Bank Erosion	S	V	*		V	
	B	V	V	*	V	*
Toe Scour	S	V	*	V		*
	B	V	V	V Degree of degradation	*	V
Bend Scour and Alignment	B	V	*	V	V	V

V = high priority
* = Secondary priority

require special attention, although deposition should also be watched for at the concave bank in monitoring bend alignment changes. Remedial actions should be taken if danger is obvious. Gradually developing threats may require more extensive level 2 monitoring.

Concerning the observable changes in Table 6.1 for low cost level 1 monitoring, the specifics are more site dependent and the emphasis is on common sense judgment. Clear changes and damages to the countermeasures installed should always be reported. Descriptions on a few typical situations are given below.

(1) Sedimentology -- The monitoring is to observe if there is any noticeable change due to sediment deposition or scour, especially for the latter. For banks and toe, deposition usually can be viewed positively as reinforcement. For the bends deposit could mean growth of the meander. Conversely, noticeable scour, whether on banks, toes or bends should be viewed with alert.

(2) Bank condition -- Monitoring is to observe if erosion occurs on the bank. Erosion to the degree of near soil wet angle of repose is always a dangerous sight. Observation should also be made on changes of implemented bank protection measures. For bioengineering bank protection measures attention should be paid to the bio-growth. Desirable growth is that which would enhance bank resistance to erosion without significantly obstructing the flood flow. Among the categories of bank condition, bed condition and sedimentology, bank condition is the one most amenable to level 1 non-structural monitoring.

(3) Bed condition -- Monitoring of bed condition is most convenient to be conducted at low flow and most important for bends. Changes of bed forms and shifting of pools and riffles are an indication of changing meander alignment.

Figure 6-1 presents a sample monitoring plan flow chart. This chart attempts to identify the major steps required in all monitoring plans. At some stages, examples have been provided. These examples have been developed in consideration of the Spoon River site. However, each site presents a unique condition and results in a unique monitoring plan.

In level 2 monitoring, cross profiling and planimetric survey satisfy both cost and quality goals for this project. The erosion pin method, while providing greater accuracy, requires extensive field work both during pin placement and reading. The data obtained during measurement provides only point measurements which must be analyzed in order to develop an overall picture of bank movement. In short, for cases where extreme accuracy is not necessary, the excess effort required for the erosion pin method is not justified. Technologically demanding photographic techniques are extremely expensive and beyond the scope of a low cost effort. Photography, in the simple sense of 35-mm pictures, while low cost, has the disadvantage of providing only a qualitative view of creek bank movement. As a supplement to planimetric and cross profiling surveys, however, photography will help strengthen an intuitive feel for the forces and trends causing stream bank movement.

Having identified repeated cross profiling and planimetric survey in conjunction with simple photography as the most effective monitoring techniques for low cost stream bank protection projects, there remains a need for development of a successful monitoring plan. One must realize that site conditions; creek geometry, flow conditions, erosion characterizations, and proximity of structures, for example, dictate monitoring requirements. The recommendations below provide guidelines for development of monitoring plans for sites similar to the case study sites presented in this report. The unique characteristics for each site must be considered carefully when developing a detailed monitoring plan.

Planimetric survey gives a clear view of lateral bank movement. The area to be surveyed must be carefully defined; it should contain the full project extents as well as some area up and downstream of the site. Areas in the immediate vicinity of a bank protection project can be drastically affected by implemented protection measures. The extents surveyed up and downstream of the site is completely dependent upon site conditions. In some cases, including a length both up and downstream along the creek equal to one half the length of the extent of the project site will be sufficient. For some sites, lengths up and downstream equal or greater than the extents of the bank protection measures should be included in the survey.

Cross-section profiles help quantify changes in channel geometry while clearly depicting breakdown in slope. Profiles should be surveyed along the entire length chosen for plan survey. The important decisions in the development of a profiling plan are the number and positions of sections to be surveyed. As a guideline, cross-sections are usually taken every 50-100 feet along the creek centerline. Cross-sections should be taken more frequently in areas where creek channel geometry changes drastically. Locations where the channel is particularly susceptible to erosion, scour, or bank failure should also be surveyed more aggressively. As the site evolves, it may be necessary to add new cross-sections. Other sections may be determined to be stable and dropped from the survey effort; this decision, however, should be made with care.

In general, it is recommended that the same survey organization and, if possible, crew conduct the repetitive surveys. Familiarity with the site will decrease field time; at the same time, a crew familiar with the site and project goals will be more likely to recognize subtle changes and suggest monitoring improvements. Photography of the channel's major features can help those not involved in field work to identify similar changes. Time between surveys must also be determined on a site by site basis. A repeated survey which shows minor or no changes can indicate either a successful protection measure or a time interval too short to resolve change. The first survey should be done immediately after the bank protection measures have been implemented. A second survey should be conducted when the level 1 monitoring described in Fig. 6.1 recommends such a level 2 survey.

Survey data can be easily interpreted. Most survey organizations use some form of computer aided drafting. Repeated surveys can be easily overlaid with previous plots. Such presentation provides immediate understanding of relative bank movement over time. Most CAD software can be implemented to provide

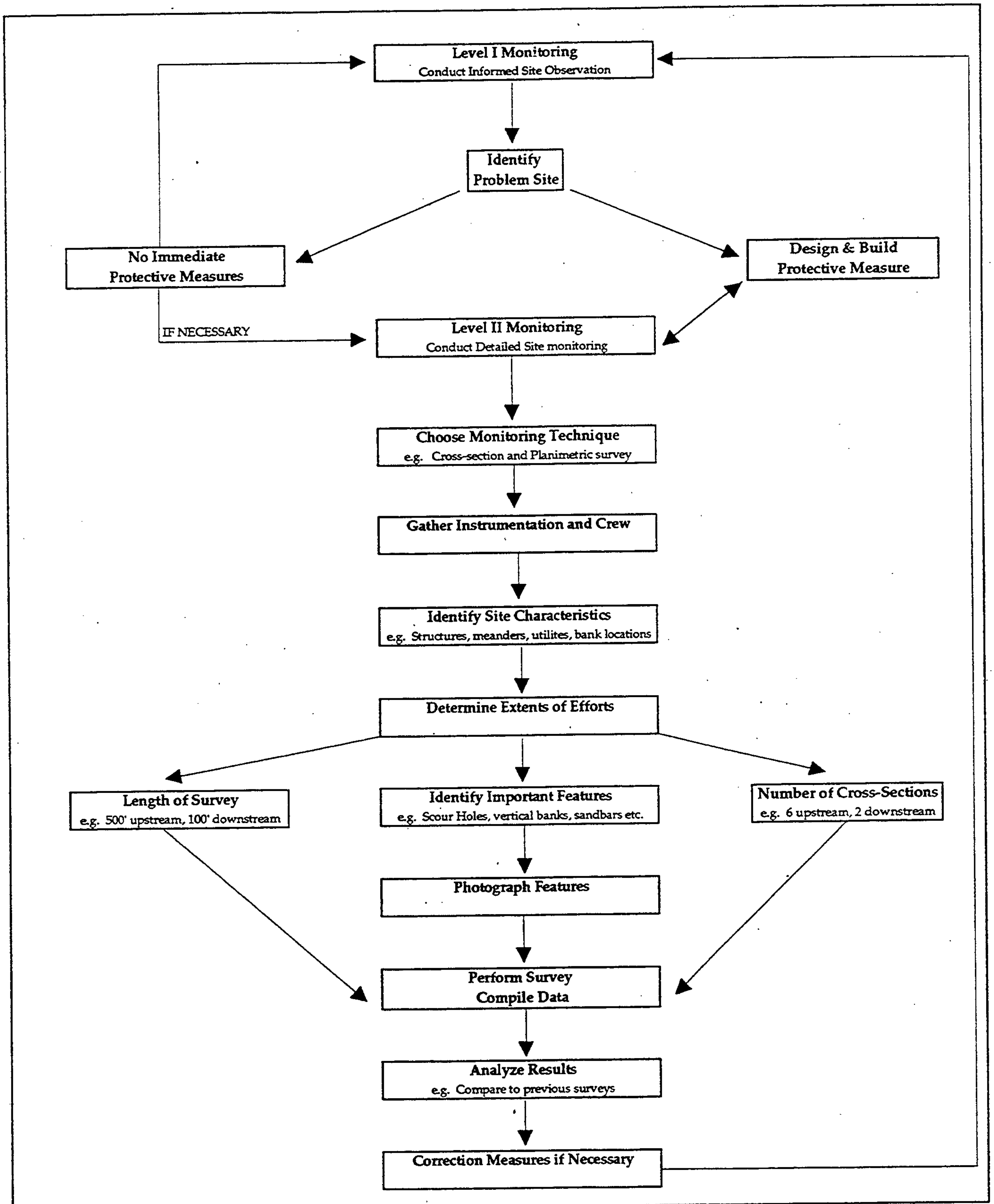

Figure 6-1. Sample monitoring plan flow-chart. (Examples developed for Spoon River site)

output for numeric changes in both bank position and average slope. Scaling directly form hard copy survey plots can yield similar results.

Introduction

In this chapter, a description of design alternatives for the four study sites detailed in Chapter 4 will be provided. For each site, the applicability of the protection techniques reviewed in Chapter 2 and the hydraulic, hydrologic, and risk factors discussed in Chapter 5 are considered. In all cases, a major objective is to implement measures to stabilize and improve the meander pattern in order to ensure long-term bridge protection and to prevent meander migration both up and downstream. Another objective is to protect streambanks from local erosion. A qualitative comparison of the merits of the methods discussed in Chapter 2 is summarized in Table 7-1 as a simple guide. Items given in this table have been derived from various sources in the literature information. Paricularly, the cost ranges are based on those given in an Army Waterways Experiment Station Report (Nunnally and Shields, 1985) modified with recent local information gathered in this project and adjusted to 1998 dollars. Some of the items (e.g., failure risk, water quality impact) are based on best current estimates and could be adjusted in the future when more data becomes available. This table is intended to be used as a preliminary rough start, and more specific conditions should be considered for a given project.

Following a brief discussion of the site characteristics (for more detailed information, see Chapter 4), several schematic design alternatives are suggested. The size of the selected streams and the advanced degrees of bank erosion and meander problems preclude very low-cost remedy measures for these sites. In fact, each of the sites involve channel alignment problems and, hence, instream correction structures are desired. Real low cost alternatives would be more likely feasible for smaller streams and for incipient stages of problem development. In the following, the first alternative presented for each site provides the highest level of protection; consequently, this design can be expected to require somewhat higher initial investment cost in comparison to those design alternatives which follow. Each design following the primary alternative requires decreased intervention at a lower initial cost. In general, the first alternative for each site provides an "active" measure (as described in Chapter 5) for bridge protection and channel correction; whereas, the last alternative yields only a "passive" measure where protection will be achieved only over a long period of time if the measure is not prematurely destroyed by a large flood. The failure probability is increasingly higher for lower cost alternatives. The concepts of Figure 5-7 and 5-8 should be considered in selection of the optional alternatives. The initial costs of the first alternative is relatively less time-dependent and can be estimated with confidence. Contrarily, the exact quantities of placement of materials required for the lower cost alternatives are highly dependent on the channel conditions at the time the measure is implemented. Total costs for the "low cost" alternatives under present conditions are highly unreliable as discussed in the Risks section of Chapter 5. Hence, recommendations for these alternatives are more qualitative than quantitatative. Details of the design, more accurate estimates of work quantities, and costs should be

Table 7-1. Summary of low cost bridge scour and stream instability contermeasures.

Countermeasure	Bank Protection	Toe Protection	Stream Alignment	Interference with Flow	Technical Skills Required		Cost [1] (1998 $)	Anticipated Life [2]	Failure Risk [3]	
					Design	Implementation			Initial	Long-Term
Instream Structures										
Bendway Weir		P	P	significant in guiding flow	moderate-high	low-moderate	$35–$45/cu yd	moderate	low	moderate
Cross Vanes		P	P	moderate	high	high	$20-$30/ft	long	low	low
Spur Dikes		S	P	significant in guiding flow	high	high	$35-$45/cu yd	long	low	low
Revetments										
Riprap	P	P		negligible	moderate	low-moderate	$20-$35/sq yd	long	low	low
Masonry blocks	P	S		negligible	low-moderate	moderate	$30-$40/sq yd	moderate-long	moderate	high
Bioengineering Methods										
Live Stakes (Willow posts, etc.)	P	S		moderate-low	low-moderate	moderate-high	$10-$20/ft	moderate-long	moderate	(4)
Tree and Other Biolog Revetment	P	P		moderate-low	low-moderate	moderate	$20-$40/ft	short-moderate	moderate	(4)
Bio Cribs and Live Gabions	S	P		low	moderate-high	moderate-high	$10-$20/ft	short	moderate	(4)
Bundled Bio Mass (wattles, mattresses)	P			low	low	low-moderate	$20-$30/ft	short	moderate	(4)
Brush or Woody Vegetation	P			moderate-low	low	low	$2-$5/sq yd	short-moderate	moderate	(4)
Grass and Legume Growth	P			negligible	low	low	$0.50-$10/sq yd	short-moderate	moderate	(4)
Geosynthetic Membranes	P			negligible	moderate-low	moderate	$30-$60/sq yd	moderate	moderate	(4)

P Primary Purpose
S Secondary Purpose

Table 7-1. (Continued) Summary of low cost bridge scour and stream instability countermeasures.

Countermeasure	Maintenance		Relative [5] Maintenance Cost	Aesthetic Value	Aquatic Habitat Impact	Water Quality Impact	Other Advantages	Other Disadvantages
	Initial	Long-Term						
Instream Structures								
Bendway Weir	low	moderate	low-moderate	slightly negative to fair	neutral-positive	none	allows native vegetation to establish	difficult and sensitive design, local scour may occur between weirs
Cross Vanes	low	moderate	low	slightly negative to fair	neutral-positive	low		
Spur Dikes	low	moderate	low	negative	neutral-positive	low		
Revetments								
Riprap	low	moderate	low-moderate	none to slightly negative	negative-neutral	none	resistant to ice flows, permits vegetation in time	
Masonry blocks	moderate	moderate	moderate-high	none to slightly negative	negative-neutral	none		
Bioengineering Methods								
Live Stakes (Willow posts, etc.)	moderate	low	moderate	moderate-good	positive	favorable	increase wildlife habitat, natural soil stabilization	
Tree and Other Biolog Revetment	moderate	low	moderate	fair	positive	none		
Bio Cribs and Live Gabions	moderate	low	low-moderate	slightly negative	positive	low		
Bundled Bio Mass (wattles, mattresses)	moderate	low	low-moderate	slightly negative	neutral	low		
Brush or Woody Vegetation	moderate	low	low-moderate	good	neutral	favorable		dependent on vegetation survival
Grass and Legume Growth	moderate	low	low-moderate	good	neutral	favorable		dependent on vegetation survival
Geosynthetic Membranes	moderate	low	low-moderate	low-fair	negative-neutral	none		

NOTE:

(1) The true cost of any method should be assessed over its total life.
(2) The anticipated life and risk of failure are associated with design parameters such as frequency of protection, duration, depth of flow, velocity, soil type, channel stability, etc.
(3) The risks of failure over long term use may increase for bioengineering methods if creek alignment changes over time.
(4) Bioengineering methods are a relatively new concept and there are few installations with sufficient history to adequately document anticipated life, long-term risk of failure, and long-term maintenance requirements. However, the long-term failure risk of the bioengineering countermeasure itself is anticipated to be low.
(5) Bioengineering methods may have higher initial maintenance costs until well established. Long-term maintenance may be low since vegetation requires little attention. Structural methods may have low initial maintenance, but higher long-term maintenance as the structure deteriorates.

carried out just before the time of construction. These design alternatives have associated with them an increased risk and maintenance cost which, over the long term, may make them undesirable. Following each site discussion, a summary table of preliminary estimates of work quantities for each design alternative is provided. These tables provide a means of comparing the initial capital cost for each alternative. More detailed estimates must be made at the time of construction.

Specific monitoring recommendations are included after each design suggestion to help evaluate the success of that alternative. If, under monitoring, it is determined that the chosen alternative is not providing sufficient protection, more extensive measures should be taken.

Cahokia Creek

Conditions at the Cahokia Creek site are such that the formation of a natural cut-off is imminent (See Figure 4-3). Such a cut-off will cause a bed degradation wave to travel upstream, perhaps threatening the bridge. Also, the local landowner has expressed concern and interest in preventing the creation of an oxbow lake and isolation of his land. A land bridge of only 60 feet is all that remains between the two bends in the Creek.

The precarious situation at this site has already lead to extensive protection measures. In mid February of 1995, the downstream bend was treated with longitudinal peaked stone toe protection and willow posts. The total cost for this effort was nearly $55,000. Site inspections have indicated that this bend appears to be well stabilized. Over the duration of this project, the upstream bend on the site has been treated with longitudinal peaked stone toe protection and "minimal-stone" bendway weirs. This protection also appears to be having a positive influence.

Design Alternative 1.

Design recommendations for this site consist of some suggestions for additional protection on the upstream bend for positive reinforcement of existing measures. Two main concerns exist. First, there is some question as to whether the existing weirs will be able to structurally withstand particularly high flow conditions. A Shields' Diagram analysis indicates that a two year flood would be capable of moving riprap having a size less than 4 inches. While most material used in the weir construction (5-6") is larger than this critical size, it is important to realize that the Shields Diagram is only applicable for straight channels under ideal conditions. The local shear stresses on the outside of a bend such as that found on the Cahokia site are more difficult to estimate but are generally considerably higher.

The second concern for the upstream Cahokia site is more critical. High flows will overtop the level of existing longitudinal peaked stone toe protection, and the banks above this level are completely unprotected and unvegetated. Under these conditions, the possibility of extensive scour above and behind the toe protection is considerable. This sort of erosion will lead to bank failure and continued migration of the upstream bank toward cut-off. Considering the clear success of the downstream protection, willow planting is highly recommended. During flood conditions, these will induce deposition and the establishment of a naturally stable bank.

Figures 7-1 and 7-2 represent the surveys collected for this project with the relevant portions of protection design shown. Steps for design implementation are listed below:

(1) Willow planting requires bank reshaping to a slope no greater than 1:1.5 (See Appendix II). Figure 7-1 shows the reshaped bank line. This effort will require the use of a track hoe in order to pull back the oversteep banks to the appropriate slope. Reshaping should begin at the top of the pre-existing toe protection and continue to the natural land surface. It is estimated that the excavation of approximately 150 cubic yards of soil will be needed. Materials removed from the bank will be needed to fill a small area between section CC and DD. The use of heavy equipment on this site must be done with great care in order not to endanger the existing land bridge.

(2) Willow posts should be planted in rows with approximately 3 feet spacing. This planting should proceed in accordance to recommendations supplied in the Literature Review and Appendix II: Willow post guidelines. Treatment of the full bend can be expected to require approximately 750 willow post. These willows should be taken from local sources in order to insure best survival rates.

(3) A 4 inch layer of gravel material around the willows is suggested in order to provider further protection until the plantings become well established. A volume of approximately 125 cubic yards is required for this layer. As a cost cutting measure, this layer may be omitted.

This site should be monitored monthly through the first year of willow growth. Observers should check willows and keep records of survival rates. After planting, the willow posts may not immediately take hold. Several months, depending upon weather conditions, should be allowed before "failed" posts are removed and replaced. The banks around the willow posts must be monitored for scour development. If extensive scour develops, willows will be uprooted and die. If this type of scour is observed, protective measures discussed in step 3 above can be retrofit and should mitigate scour. Once the willows have survived the first year, regular monitoring procedures should be adopted (Chapter 6) to insure continued site stability.

There are a series of other design approaches which may be substituted for design alternative 1. They all have the same goal of protecting the banks above the line of longitudinal peaked stone toe protection already on site. These approaches do not provide as thorough protection to the banks; however, considerable initial investment savings may be gained. A preliminary estimate of the work quantities required for all design alternatives is presented in Table 7-2.

Design Alternative 2.

No excavation - Willow planting is recommended only for slopes less than 1:1.5. This site presents slopes slightly steeper than recommended. However, to avoid excavation, it may be possible to place willow posts without bank reshaping. Some bank material can be expected to crumble from the steep-sloped sections. This soil may naturally provide bank stabilization; however, loosening this soil increases the risk that, during high flow events, large quantities of bank materials will be washed downstream. Also, it has been documented (Derrick, 1995) that slopes greater than 1:1.5 result in higher willow mortality rates.

Monitoring suggestions for this alternative are the same as those for alternative 1. At these steeper slopes, extra care should be taken to observe scour progress. If extensive scour scaring is visible, measures should be taken to secure the soil around the willows.

Design Alternative 3.

Brush Matting - Securing live or dead brush to the bank with wire and stakes as shown in Figure 2-7 may protect the banks and promote sediment deposition. The brush matting would extend from existing riprap toe (no biolog toe is necessary) to the top of the bank. In material cost, this technique is quite inexpensive. Labor costs provide the biggest obstacle to this design. Also, a considerable amount of brush will be require for such a design.

Monitors of brush matting should visit the site regularly (monthly during flood season) until it is decided that the brush layer is well silted in. It is especially important to assure that the brush remains fastened to the banks, particularly after a high flow event.

Design Alternative 4.

"Do Nothing"- Conditions on this site warrant the consideration of a "do nothing" alternative. The downstream bend of this pending oxbow lake has been well stabilized with willow post and riprap toe. The existing upstream protection in the form of riprap toe and "minimal stone" bendway weirs may provide sufficient protection.

If this alternative is chosen, careful monitoring must be practiced. After high flow events, the banks above the level of the stone toe should be examined for extensive scour and mass failure scaring. If continual observations of these types of countermeasures indicate that the stream is continuing to progress towards cut-off, then further protective measures will be needed to protect the site. Also, observers should note and evaluate the response and condition of the weirs on site after high flow events.

Table 7-2. Preliminary estimate of work quantities for Cahokia Creek site.

Alternative Number	Soil Excavation (yd^3)	Willow Posts (units)	Gravel Layer (4 in.) (yd^3)	Brush Layer (6 in.) (yd^3)
1	150	750	125*	--
2	--	750	125*	--
3	--	--	--	250

* Gravel layer is optional.

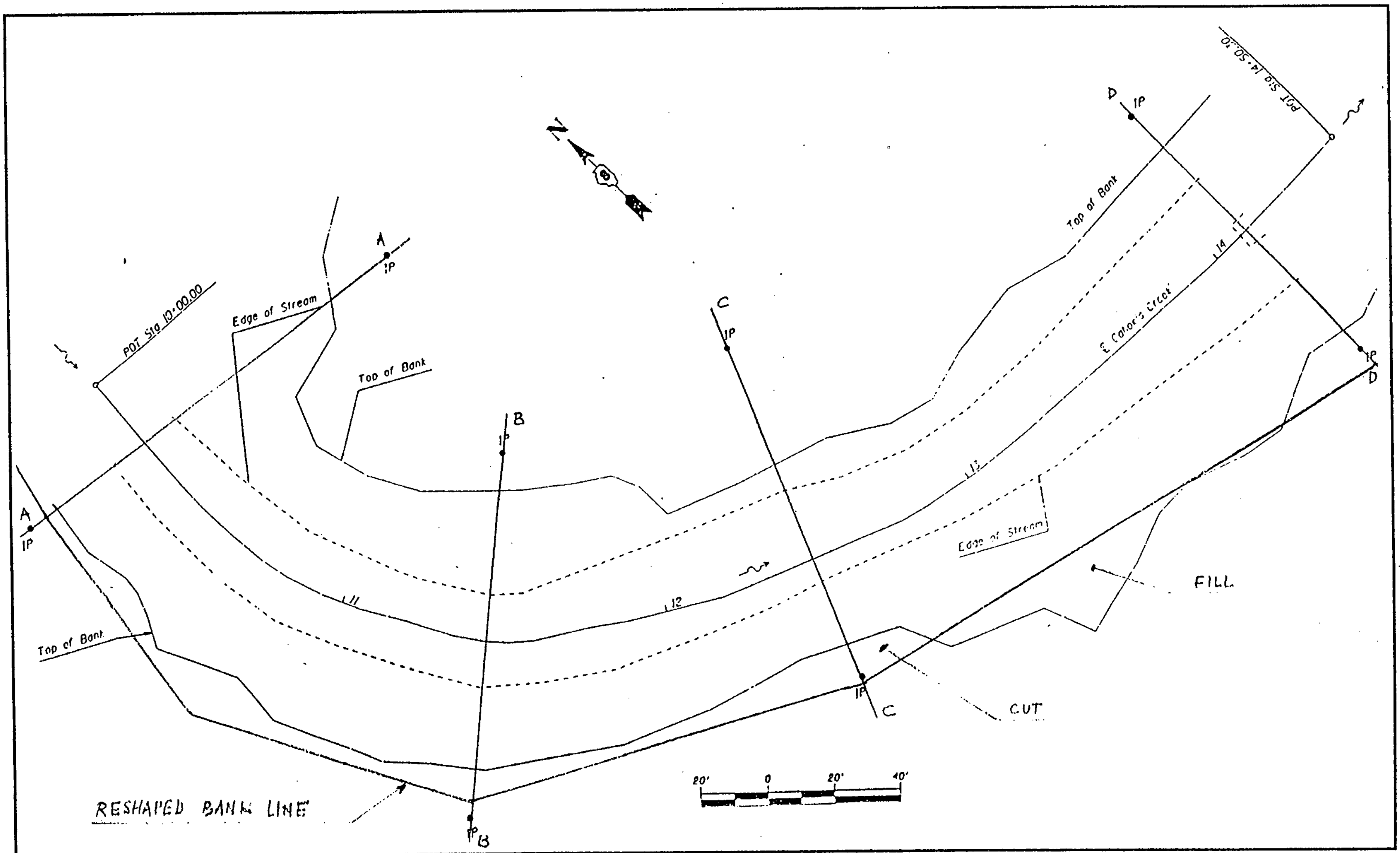

Figure 7-1. Cahokia Creek site alternative 1 design schematic (plan view).

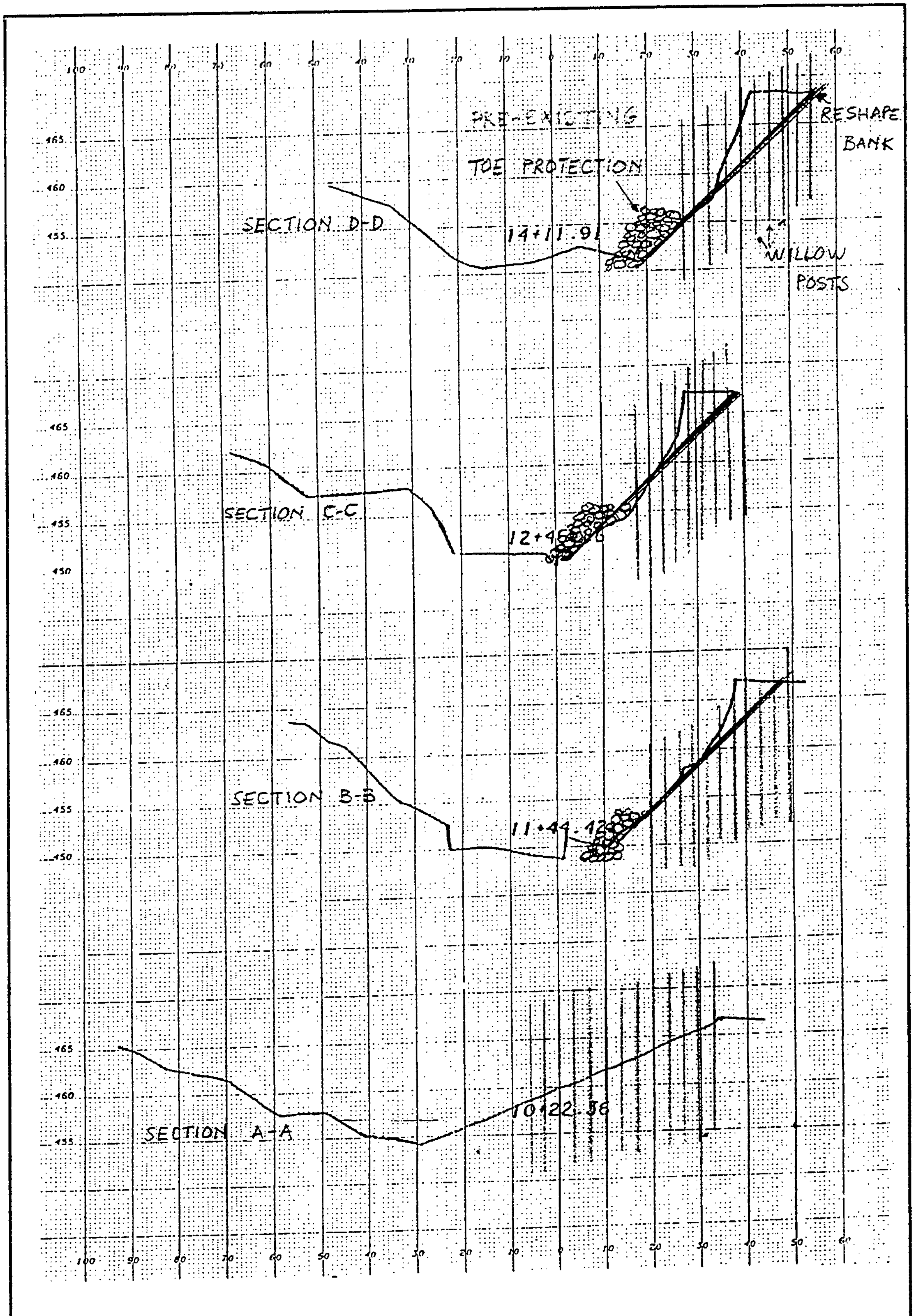

Figure 7-2. Cahokia Creek site alternative 1 design schematic (cross-section view).

Piasa Creek

The Piasa Creek has a tight meander just upstream of the IL. Rte. 3 bridge. In this region, there are three areas exhibiting strong deposition accompanied by locations of extensive scour and toe undercutting. Those sections undergoing erosion are in critical condition; the east bank shows huge bank scour holes. Here, the bank's last lines of defense consist of a number of large trees, many of which are on the verge of falling into the stream. The channel itself is already heavily cluttered with debris, trees, and an island sandbar. Any protective measure must be preceded by a channel cleanup; some of the material taken from the channel may be used in the following revetment plan.

Design Alternative 1.

Considering the severity of conditions on site, a multiple measure approach to bank protection is again suggested. The primary goal of the following design is to stabilize the meander, preventing further growth and improve channel alignment. This will provide long term protection to the bridge. The defense of the east bank just upstream of the bridge will require a strong stone toe with considerable backfilling. The most extensive design presented below also seeks to modify the Creek's approach alignment to one more favorable to the existing bridge.

The bend threatening the site has its origin well upstream of the bridge; therefore, countermeasures must begin upstream. The survey compiled for this project does not encompass the extent of the protection design. For this reason, a trace of the available 1994 aerial photo (Figure 7-3) has been used to illustrate a schematic layout. Significant geometric changes since 1994 are likely. Design implementation will require a detailed field survey of the area depicted on the photo trace. This site is known to be in a dynamic state of evolution. The position, angle, and spacing of the dikes as presented below are based on the recent situation. The design must be re-evaluated to reflect site conditions at the time of construction.

Figures 7-4 and 7-5 represent the surveys collected for this project with the relevant portions of protection design depicted. Steps for design implementation are listed below:

(1) A stone toe is to be placed from section EE to just upstream of section AA along the east bank. The toe should be composed of RR-5 gradation riprap, and this is estimated to require 300 tons of stone. This toe, as shown in Figure 7-4, is set well away from the bank between section AA and CC. Here, the area behind the toe must be backfilled. Some of the materials garnered from channel cleanup may be used for this process. A total of 650 cubic yards of material is estimated for backfill. From section CC to section EE, the toe protection blends into bank riprap bank revetment. This will require approximately 200 tons of riprap. The riprap blanket will serve as strong protection in the immediate vicinity of the bridge.

(2) It is suggested that 3 to 4 rows of willow posts be used to fortify the filled areas; approximately 100 post will be needed. These will provide strength to the new bank while promoting deposition and natural bank development.

(3) Five stone dikes angled downstream have been specified in an attempt to direct the flow towards a more favorable channel alignment while stabilizing the existing meander through sediment deposition at the downstream face of the dikes (See Chapter 5). Dikes often submerged during floods, especially those at the convex bank, may be angled upstream. Dikes at the upstream portion of a concave bank, especially those rarely submerged, are more effective angled downstream to guide the flow. In this case, the flow separation zone downstream of the dike as well as the slow flow regime upstream and behind the dike promote sediment deposition. Therefore, more detailed analysis, considering hydrology as well as hydraulics, should be made on the dike heights and angles at the time of design and construction. The in-stream beginning point and angle of inclination of these dikes have been specified on Figure 7-3. The coordinates presented are to be measured from the northwest bridge abutment. Figure 7-3 is a trace of a 1994 aerial photo, and the exact bank locations are uncertain. For this reason, dike lengths have not been estimated. In general, dikes should extend from the beginning point to the *stable* streambank where the dikes should be keyed into the bank. The stone toe "wall" just downstream of dike A serves as a final flow deflector. All dikes should be approximately 5 feet high. A rough estimate indicates approximately 900 tons of stone will be required for dike construction.

For monitoring, initial visits to this site should be conducted on a bi-annual basis. Monitors should observe the extent of sediment deposition downstream of the dikes and note any undesirable scour in and around the dikes. Progress of sediment deposition will be slow; however, there should be noticeable change within six month time intervals. The fill and plantings between sections AA and DD should be monitored for scour and willow post survival rates. The stabilization of this area is key to the protection of the bridge. Until the willows are well rooted and natural vegetation begins to thrive, special monitoring visits should be conducted after high flow events to evaluate the extent of erosion between AA and DD. If plantings are unable to be established, a more traditional method of protection (riprap or gabions) may be required for this newly filled bank area.

There are a series of other design approaches which may be lower cost substitutions for design alternative 1. They all have the same goal of stabilizing the approach meander to the bridge. These approaches provide neither thorough protection to the banks nor meander alignment improvement. However, considerable initial investment savings may be gained. A preliminary estimate of the work quantities required for all design alternatives is presented in Table 7-3.

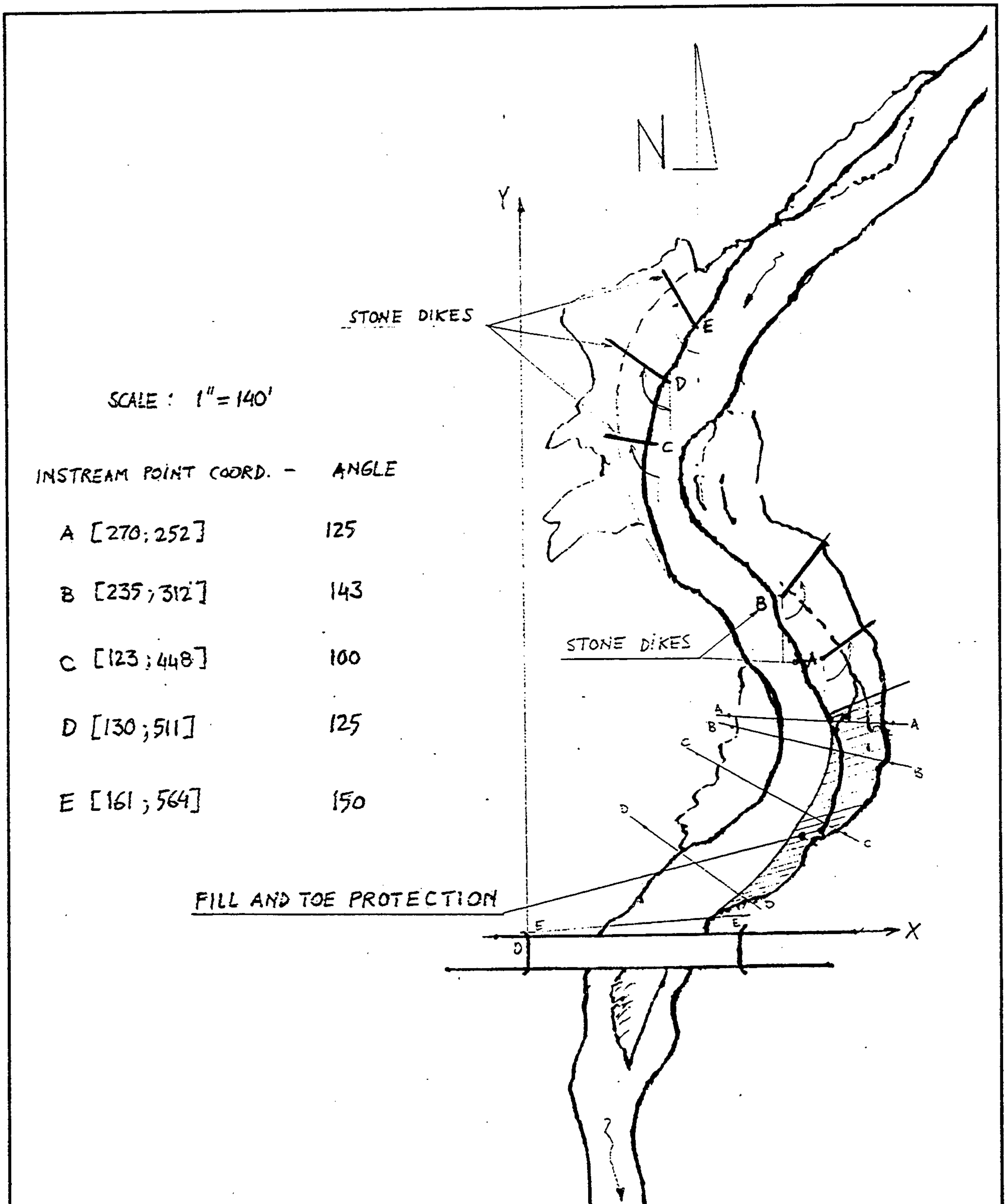

Figure 7-3. Piasa Creek alternative 1 design overview (aerial photo trace - 1994).

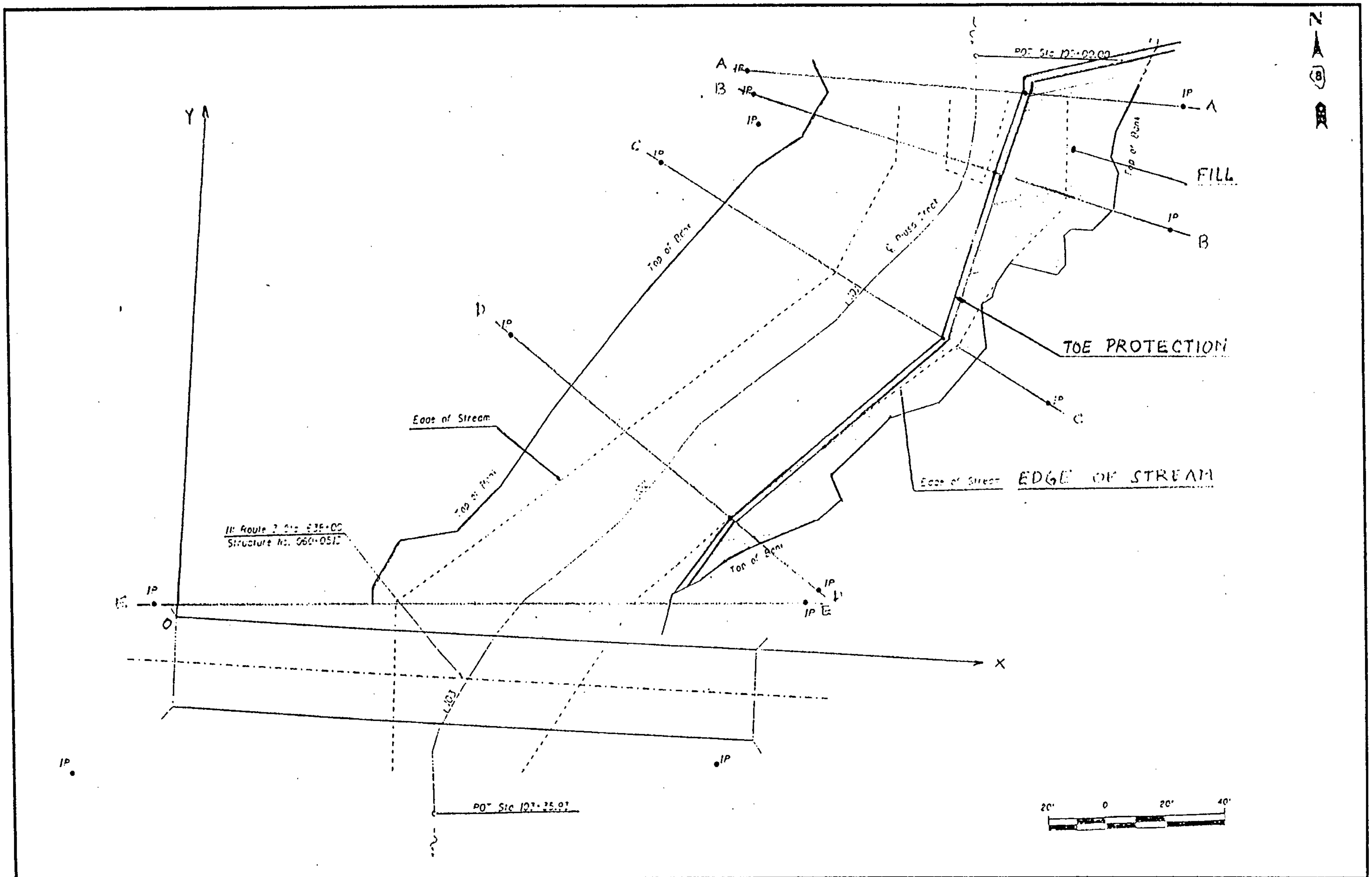

Figure 7-4. Piasa Creek site alternative 1 design schematic (plan view).

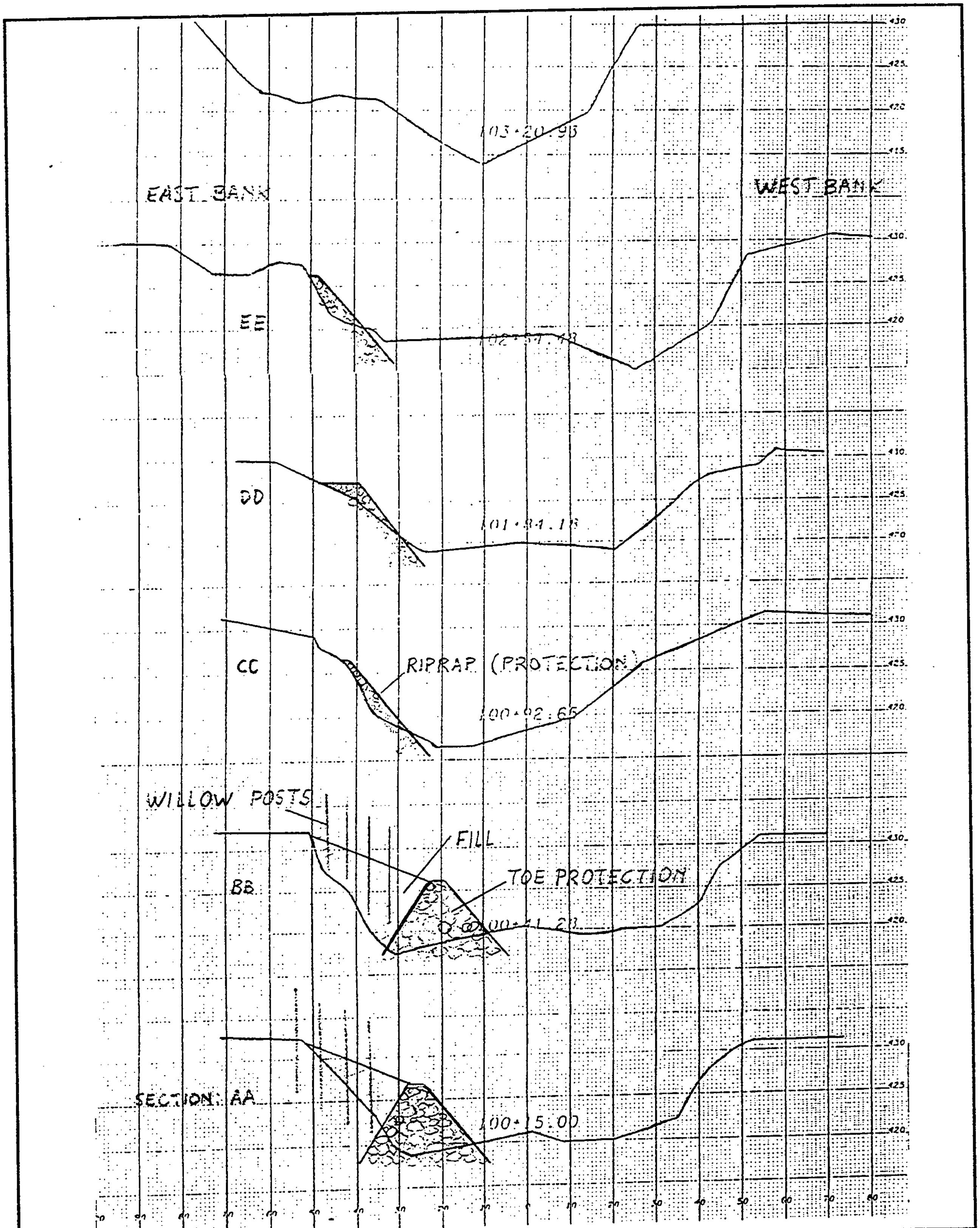

Figure 7-5. Piasa Creek site alternative 1 design schematic (cross-section view).

Design Alternative 2.

No riprap toe protection - The dikes are designed to promote deposition along their downstream face. For the majority of flow conditions, these dikes should protect the downstream toe from erosive forces. As a cost cutting measure, the stone toe may be eliminated between section AA and CC.

Removing the toe foundation between section AA and CC makes filling this area impractical. Instead, it may be possible to set willow posts along the bank line and within the scour holes themselves without fill. During high flow conditions, these areas experience erosive eddies. The willow posts may serve to break these eddies, slowing the water and allowing for long term deposition. The severity of scour on-site, however, makes this alternative uncertain.

It is also possible to replace some of the upstream stone dikes (C,D,E of Figure 7-3) with dikes built from dead trees removed from the channel. These should be securely anchored to the banks and bed. If sufficient material is available and the dikes are well constructed, tree dikes will function as well as stone dikes.

If this alternative is chosen, initial monitoring should be conducted every three to six months. The frequency can be reduced to semi-annual or annually in late summer or early fall if the protection measure proves to be effective and silting progresses. Special attention should be given to the east bank just upstream of the bridge. If continued scour and willow fatality is noted, more extensive protection measures should be taken immediately.

Design Alternative 3.

Dikes Only - If the toe protection and filling is eliminated as in alternative 2, it may also be possible to eliminate the planting of any willow posts. The large trees still standing between section AA and CC are the east bank's last line of defense. If the dikes are installed as suggested, these trees should be relieved of considerable erosive forces. The dikes will promote deposition, and the trees may re-establish themselves, providing considerable bank protection.

The stability of the trees, however, is in question. If they were to fail, the east bank would be virtually unprotected. Therefore, alternative 3 is high risk.

For this design, monitoring should be quite aggressive on a monthly basis. The area between section AA and CC should be observed. If considerable deposition does not occur, the trees may fall. In this case, more active protection must be installed. If the trees hold and silting develops, the monitoring frequency may be decreased to three to six months.

Table 7-3. Preliminary estimate of work quantities for Piasa Creek site.

Alternative Number	Stone (Dikes) (tons)	Riprap (Toe) (tons)	Riprap (Banks) (tons)	Fill (yd^3)	Willow Post
1	900	300	200	650*	100
2	900#	–	200	*	100
3	900#	–	200	*	–

* Channel cleanup not included.

Tree dikes if fallen trees available on-site.

Senachwine Creek

This site provides a typical example of geomorphologic development of a stream threatening a highway bridge. Figures 4-31 and 4-32 indicate the drastic movement of the Senachwine over the past 30 years. The original channel configuration was aligned straight and inclined to the bridge at the time of its construction. Over time, a meander has approached the bridge from upstream. The progression of this meander has been constrained by the bridge piers, and the river is responding by developing a tight radius bend immediately upstream in order to circumvent the bridge. With the bridge as the control point, a meander has also developed downstream in response to the evolution upstream. The tighter the meander becomes, the more rapid the rate of meander progression can be expected. Therefore, protection of the bridge requires control of the meander.

This site poses particular difficulty in the selection of a protection measure because the creek threatens two significant structures. There is evidence that, in recent years, the east bank has been even more rapidly approaching the bridge abutment and road embankment. This bend, in turn, forces flow energy toward the west bank in the vicinity of the large power line tower. These two structures are in danger of being undermined. At both locations, there is little room that would allow for bank reshaping. As a result, protection measures such as willow posts bank protection, which require reshaping, must be rejected.

Low head bendway weirs are not suggested for bends exhibiting such tight radii. Any protection measure adopted here must be quite extensive and "active" in order to counteract the severity of the problem. At this stage of meander development, low cost measures will not be effective. This site, however, does present one advantage for design implementation: it is known to undergo frequent periods of "no flow" conditions. During these periods, design implementation can progress with relative ease.

Design Alternative 1.

The goal of the protection measure recommended below is twofold. First, it is necessary to armor the critical bank areas against further erosion. The second and long term goal is to train the flow of the river towards a more acceptable bridge approach path. These goals are to be accomplished in this alternative through a combination of measures including, riprap toe protection, riprap revetment, stone dikes, and willow planting.

The bend threatening the site has its origin well upstream of the bridge; therefore, countermeasures must begin upstream. The survey compiled for this project does not encompass the extents of the protection design. For this reason, a trace of the available 1994 aerial photo (Figure 7-6) has been used to illustrate a schematic layout. As already stated, significant geometric changes since 1994 are observed. Design implementation will require a detailed field survey of the area depicted on the photo trace. This site is known to be in a dynamic state of evolution. The position, angle, and spacing of the dikes as presented below is based on the recent situation. The design must be re-evaluated to reflect site conditions at the time of construction.

Figures 7-7 and 7-8 represent the surveys collected for this project with the relevant portions of protection design depicted. Steps for design implementation are listed below:

(1) A riprap toe must extend 50 feet upstream of the inside end of dike *e* to just downstream of section CC along the east bank. This toe is estimated to require 1200 tons of RR-5 gradation riprap. In general, the toe height should be maintained at 5 feet, sloping to 3 feet near the ends. The instream toe slope should be approximately 1:1.5. In the location between dike *a* and *c* the toe protection has been specified well away from the bank. The region behind the toe should be stabilized with fill sloping from the top of the toe protection to the existing bank. Approximately 900 cubic yards of fill material will be needed. To protect the new bank, two or three rows of willow posts (See Appendix II for willow post planting guidelines) should be planted; these will promote deposition and begin the natural rebuilding process. A total number of 200 willows is estimated.

(2) Five dikes are planned for the east bank. These dikes will promote sediment deposition at their downstream face during most periods of flow(See Chapter 5). Each begin at the toe protection and extend into the stream at a constant height of 5 feet. The locations of the dikes have been specified on Figure 7-6 using an origin located at the intersection of the road and bridge abutment as marked. The instream beginning of each dike is labeled with points A-E. Dike angles are measured from the vertical projection of this point to the road. The following table summarizes the dike specifications. Lengths presented are approximate; constructed dikes must extend from instream point to toe protected banks. Exact lengths will depend on further field surveys. The dikes are expected to require approximately 650 tons of stone for construction.

Dike	Point Coordinates (ft)	Angle (deg)	Length (ft)	*Height (ft)*
a	A{170,165}	30	30	5
b	B {265,220}	60	65	5
c	C {380,305}	80	60	5
d	D {490,410}	105	50	5
e	E {565,505}	133	40	5

(3) The series of dikes will force the flow against the west bank in the vicinity of the electrical tower. Therefore, this region must be heavily protected. A stone toe with full riprap revetment extending from the bridge abutment to 100 feet upstream of section DD is recommended. When constructing this toe, sufficient stone for scour settling must be incorporated. The final resulting toe should supply approximately 5 feet of protection, sloping to 3 feet at the ends. Upbank of the toe, riprap revetment is suggested. The bank slope in this region will require no reshaping, and a riprap blanket of approximately 12-18 inches is suggested. Approximately 550 tons of stone is

necessary for this effort. Consult the literature review and Appendix I for further guidelines on placement of riprap protection.

The toe and revetment must be placed against the *far* west bank for its entire length. This bank does not appear on the current surveys for sections AA, BB, and CC. Between these sections, the line marked "Top of Bank" actually denotes a banked sand bar formed in an intermediate stage of the creek's evolution. The IP for these sections of the survey has actually been set within the sand bar. Therefore, the exact location of the revetment between sections DD and CC is unknown, and Figure 7-7 and 7-8 represents an estimate of its location.

For monitoring, initially visits to this site should be conducted on a bi-annual basis in mid-spring and late fall. Monitors should observe the extent of sediment deposition downstream of the dikes and note any undesirable scour in and around the dikes. Progress of sediment deposition will be slow; however, there should be noticeable change within six month time intervals. The fill and plantings should be monitored for scour and willow post survival rates. The stabilization of this area is key to the development of a more favorable meander approach to the bridge. Until the willows are well rooted and natural vegetation begins to thrive, special monitoring visits should be conducted after high flow events to evaluate the extent of erosion from the newly filled bank. The riprap protection in the area of the electric tower should be monitored at its ends to assure that it does not begin to unravel.

There are a series of other design approaches which may be substituted for design alternative 1. They all have the same goal of protecting the electric tower and road embankment while developing a more favorable meander alignment to the bridge. These approaches do not provide as thorough protection; however, considerable initial investment savings may be gained.

There are very few alternative designs for the southwest bank near the electric tower which can be suggested with any confidence. Therefore, all of the designs do incorporate riprap protection of this bank. A preliminary estimate of the work quantities required for all design alternatives is presented in Table 7-4.

Design Alternative 2.

Willow Toe Protection - The dikes will provide considerable protection to their downstream toe. It may be possible to replace the more expensive rock toe line with a line of tightly spaced willow post. If these are planted during the "no flow" period, they may have sufficient time to establish themselves before flood season. If this alternative is chosen, the area marked as fill in Figure 7-6 may be planted with several rows of willows. These will cause flow velocities to decrease and promote deposition behind the willow toe line.

As this alternative depends upon willow survival, initial monitoring is suggested to occur every month from March to October until the willows are deemed established. Observers should check willows and keep records of survival rates. After planting, the willow posts may not immediately take hold. Several months, depending upon weather conditions, should be allowed before "failed" posts are removed and replaced. The bed and banks around the willow posts must be monitored for scour development. If extensive scour develops, willows will be uprooted and die. Once the willows have survived the first year, regular monitoring procedures should be adopted (Chapter 6) to insure continued site stability.

Design Alternative 3.

No Toe Protection - The dikes will provide considerable protection to their downstream toe. Cost savings may be achieved by eliminating toe protection along the suggested line. If this alternative is chosen, no fill would be required, and dike B should be extended all the way to the stable banks.

For this alternative, monitoring visits should be made approximately every three months. Here, the main observation should be between the dikes for excessive scour. If scour progresses too far, the dikes may be circumvented and rendered useless.

Table 7-4. Preliminary estimate of work quantities for Senachwine Creek site.

Alternative Number	Stone (Dikes) (tons)	Riprap (Toe) (tons)	Riprap (Banks) (tons)	Fill (yd^3)	Willow Post
1	650	1200	550	900	200
2	650	--	550	--	550
3	650	--	550	--	--

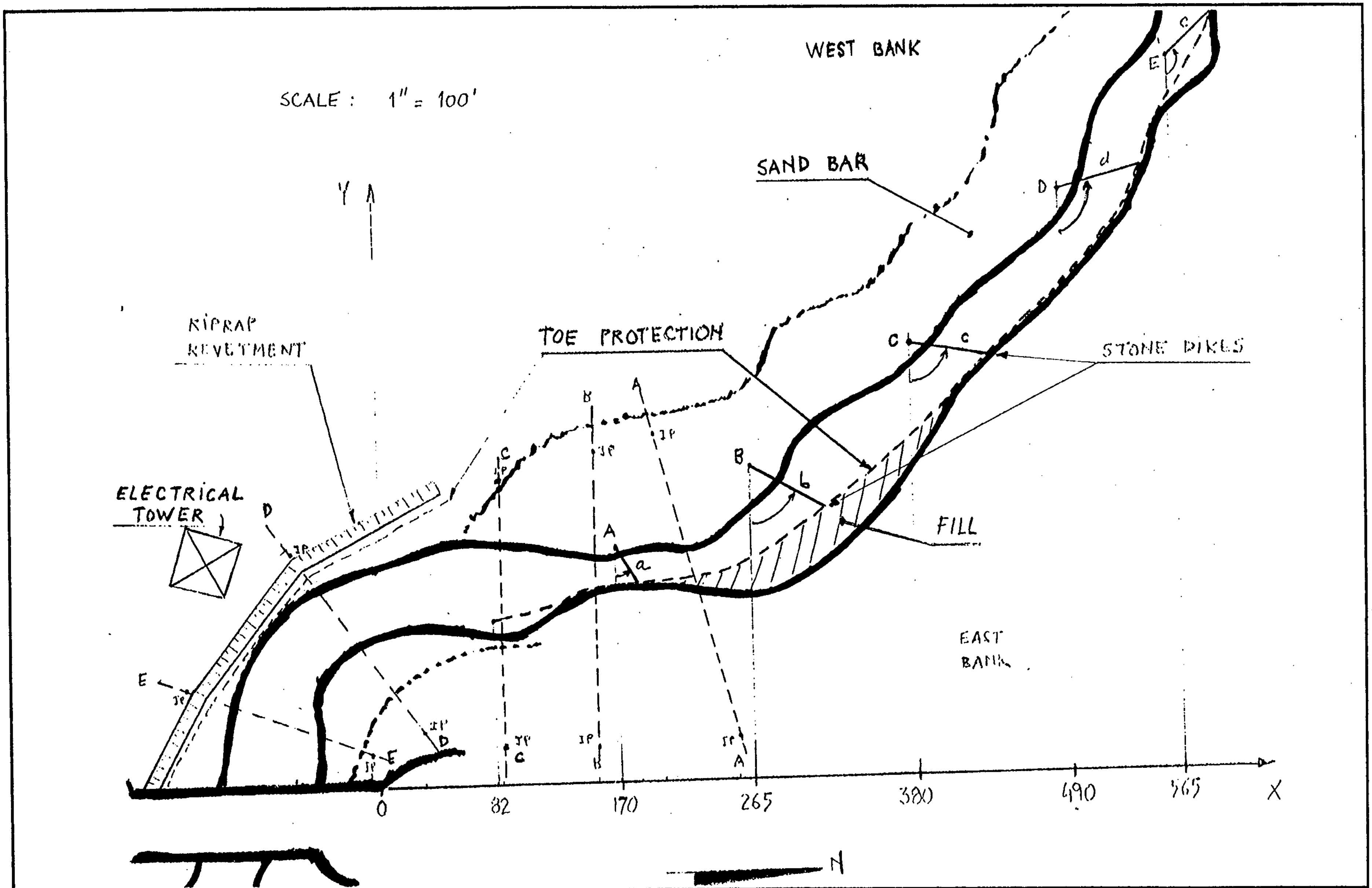

Figure 7-6. Senachwine Creek alternative 1 design overview (aerial photo trace - 1994).

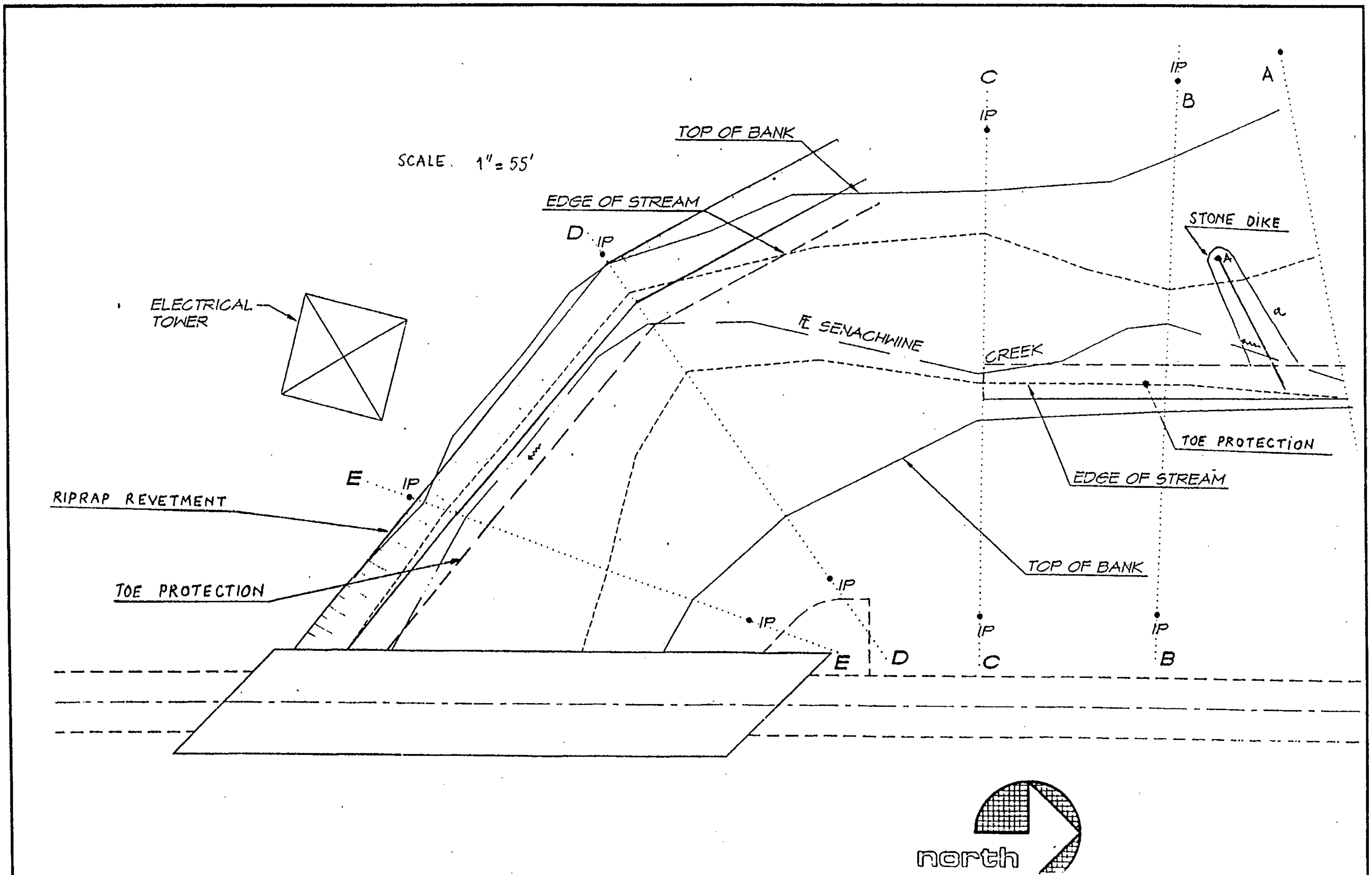

Figure 7-7. Senachwine Creek site alternative 1 design schematic (plan view).

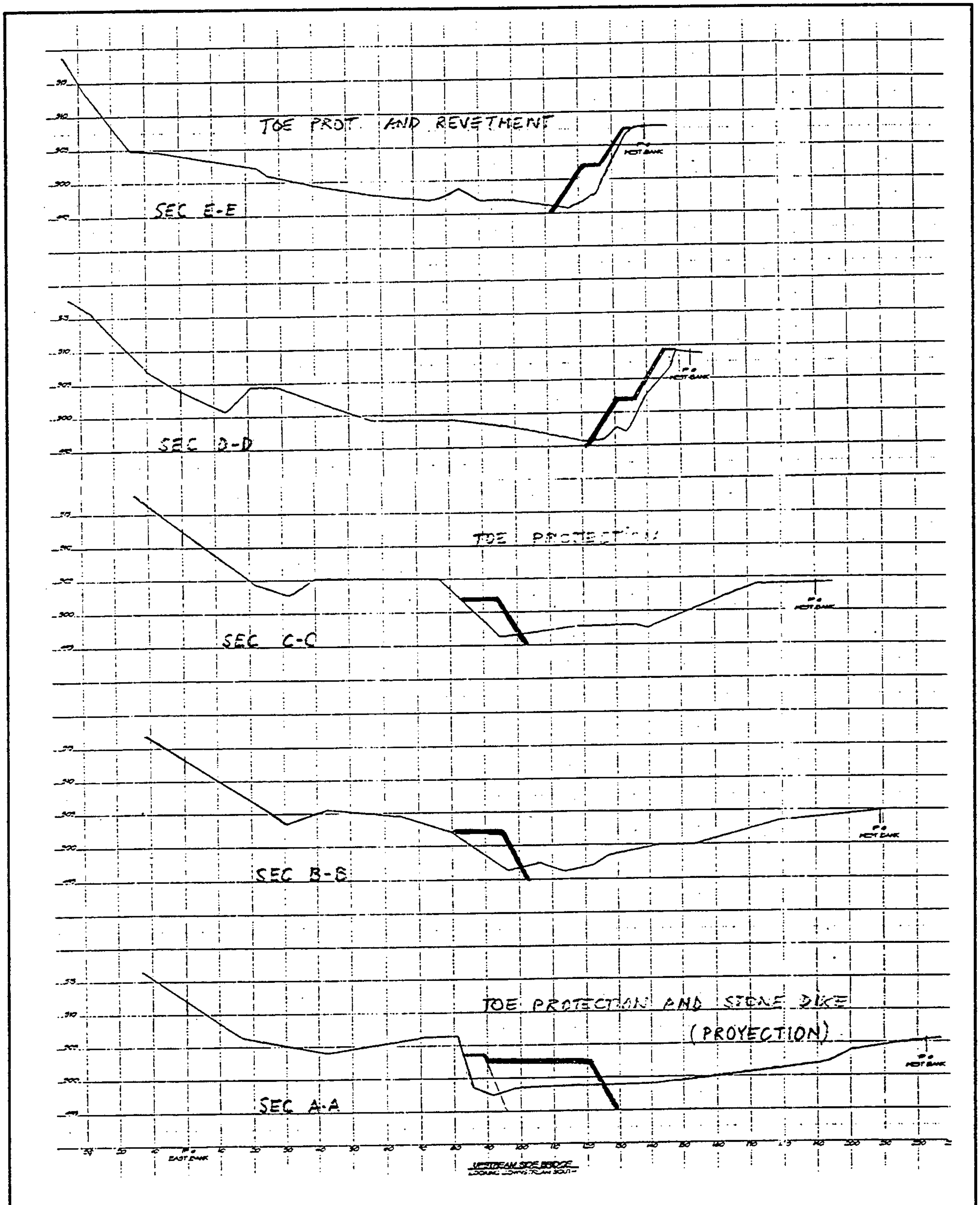

Figure 7-8. Senachwine Creek site alternative 1 design schematic (cross-section view).

Spoon River

The Spoon River's approach to the IL. Rte. 17 bridge shows the development of a mild meander (See Figure 4-30). The loam soils on-site contain considerable amounts of cohesive meander, and therefore, the banks are able to sustain near vertical slopes. However, visible scarring shows that toe undercutting and mass failure are prevalent problems along the west bank just upstream of the br:dge. If this condition is allowed to persist, farmland will continue to be lost, and over time, the bridge abutment will be threatened. Therefore, the objective of the counter measure is to prevent meander growth and to protect from local bank erosion.

A protection measure which appears promising is the use of willow posts. Willow posts alone, however, are not expected to be sufficient due to severe undercutting at the Spoon River. For this reason, a multi-measure "bioengineering" technique would be required. This design consists of willow posts combined with longitudinal peaked stone toe protection. A design of this type, however, has the disadvantage in that it requires three distinct treatments: bank shaping, riprap toe protection, and extensive planting. Also, maintenance can be expected to be more time and cost intensive. As a result, a solution of this type has been deemed economically unfeasible. Here, the important design criteria is the protection of the toe in order to halt bank propagation through toe undercutting followed by mass failure. This goal can be accomplished through several techniques.

Design Alternative 1.

The one bank protection option which requires little or no bank reshaping is the use of a weir or dike system to train the flow away from the endangered banks. The basic hydraulic conditions on-site suggest dikes as an excellent option. The Spoon River's high sediment load (See Chapter 4) can be used as an advantage to rebuild the toe and banks on site. The plan survey of the Spoon river (Figure 4-26) shows the stream's thalweg impinging on the bank at the outside of the bend approaching the IL Rte 17 bridge (near section BB). Low head riprap dikes can be used to gradually guide the thalweg towards the center of the channel while riprap toe protection will prevent further undercutting of the banks.

A schematic design of the type described above is shown in Figure 7-9 in plan view. Figure 7-10 represents a typical cross-section of the remediated Spoon river site. Steps for design implementation are listed below:

(1) Longitudinal peaked stone toe protection should be applied along the extent of the west bank from approximately 70 feet upstream of section AA to just downstream of section DD. From the upstream beginning of toe protection to section AA, tentatively, it is suggested that the height of the stone toe gradually increase from 3 to 5 feet at section AA. From section AA to CC the toe remains 5 feet high. From section CC to DD, the height should gradually return to 3 feet. The exact height of the stone toe should be further investigated at the design time considering stream hydrology. The toe protection must be keyed into the bank at either end of the project site in order to prevent scour behind the toe barrier.

(2) Three hand-placed dikes composed of riprap material are suggested as follows:

	Location	Length
1	centered on section AA	8 feet from the end of toe protection
2	76' downstream of AA	8 feet from the end of toe protection
3	at section BB	8 feet from the end of toe protection

All dikes gradually decrease in height from 4 feet at their intersection with the stone toe to 2 feet at their end. This portion of the project will require 10 cubic yards of riprap. Again, the exact dike height should be determined at design time considering hydrology.

(3) The east bank of the Spoon River in the vicinity of the bridge (Section DD) is quite steep and showing preliminary signs of under-cutting. The bank here, however, is well vegetated and appears to be stable. Implementing the above protection measure on the west bank will likely re-route significant flow energy against the east bank in the area of the bridge. In order to protect against de-stabilization of this bank, application of longitudinal peaked stone toe protection along the east bank is suggested for a length of 160 feet upstream of the bridge. The height of the toe should gradually increase from 3 feet at section DD to 5 feet at the bridge. Approximately 800 tons of RR-5 gradation riprap (See Appendix I) will provide the required toe protection on both east and west banks.

Conditions at this site are not as severe as at Piasa and Senachwine. For this reason, it should be sufficient to conduct monitoring on an annual basis. At this time, the extent of deposition along the west bank should be noted. Also, condition of the dikes should be monitored after particularly high flow events to insure they are capable of sustaining high flow erosive forces. A preliminary estimate of the work quantities required for all design alternatives is presented in Table 7-5.

Design Alternative 2

Bendway Weirs -A second alternative which is lower cost and should perform nearly as well as that suggested above is a series of low head "minimal stone" bendway weirs. Here, the goal shifts from attempting to develop a natural toe to directing the flow energy away from the threatened bank. These weirs would be angled slightly upstream. When weirs are submerged, the flow will be guided away from the banks, perpendicularly across their top (See Chapter 5). Three weirs, placed similarly to the dikes suggested in design alternative one, are suggested. These weirs provide some savings in that less stone is required for their construction (See Chapter 2 for construction details). Again, for maximum safety, it is suggested that a stone toe protection be installed between the weirs.

construction (See Chapter 2 for construction details). Again, for maximum safety, it is suggested that a stone toe protection be installed between the weirs.

Monitoring of this alternative can also be conducted on an annual basis. Scour between and above the weirs are the main concern for this design alternative. This can be a particular problem with low head bendway weirs because they are submerged for the majority of the operating time. Following large flow events, the integrity of the weirs should be confirmed.

Design Alternative 3

No Dikes - The main criteria for this site is the protection of the toe. At a slightly greater risk, this may be accomplished without installation of dikes or weirs. A single treatment protection plan which consists of a line of stone toe protection along both banks as marked on Figure 7-9 may be sufficient to stabilize this site.

At an even lower cost, a line of tightly spaced willow posts placed at the waters edge could also be used to provide the required toe protection. Of course, this alternative represents even greater risk as it depends upon the survival of the willow posts.

For this alternative, monitoring should be somewhat more aggressive. If a stone toe is used, initial monitoring visits should be made approximately every 6 months. The site should be investigated for erosion and mass failure scarring above the level of toe protection. If the banks continue to fail above the toe protection, more active means of protecting the banks from erosive forces (dikes or weirs) is suggested.

If a line of willows is place, success is dependent upon willow survival. Initially, the willows should be monitored monthly from March to October to evaluate their survival rate until it is deemed they are well established. Afterwards, monitoring procedures described in Chapter 6 should be adopted. If willow fatality is high, dikes or weirs should be installed to shelter them from the flow's full force.

Design Alternative 4.

"Do Nothing"- Condition on this site warrant the consideration of a "do nothing" alternative. The bridge is not in immediate danger, and the meander currently appears to be in a relatively stable phase of evolution.

If this alternative is chosen, careful monitoring must be practiced. After high flow events, the banks should be examined for extensive scour and mass failure scaring. Continual observations of these type indicate the stream continues to progress towards the bridge, and some protective measures will be needed to effectively protect the structure and maintain the channel alignment.

Table 7-5. Preliminary estimate of work quantities for Spoon Rive site.

Alternative Number	Riprap (Toe) (tons)	Riprap (Weirs) (tons)	Willow Posts
1	800	10	–
2	- 800	2	–
3	800*	–	300*

* Alternative 3 - Either riprap OR willow post toe protection.

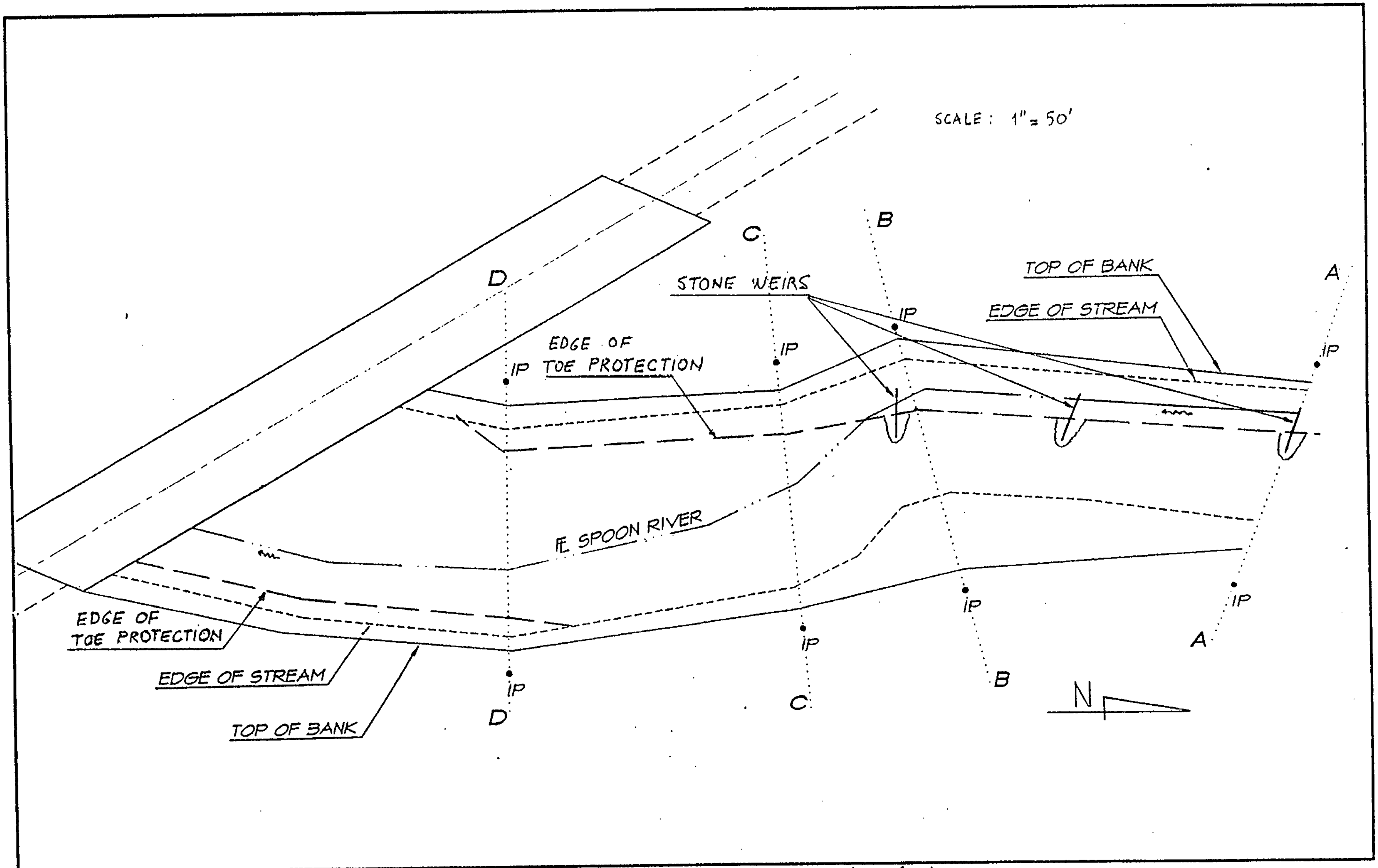

Figure 7-9. Spoon River survey showing alternative 1 schematic protection design.

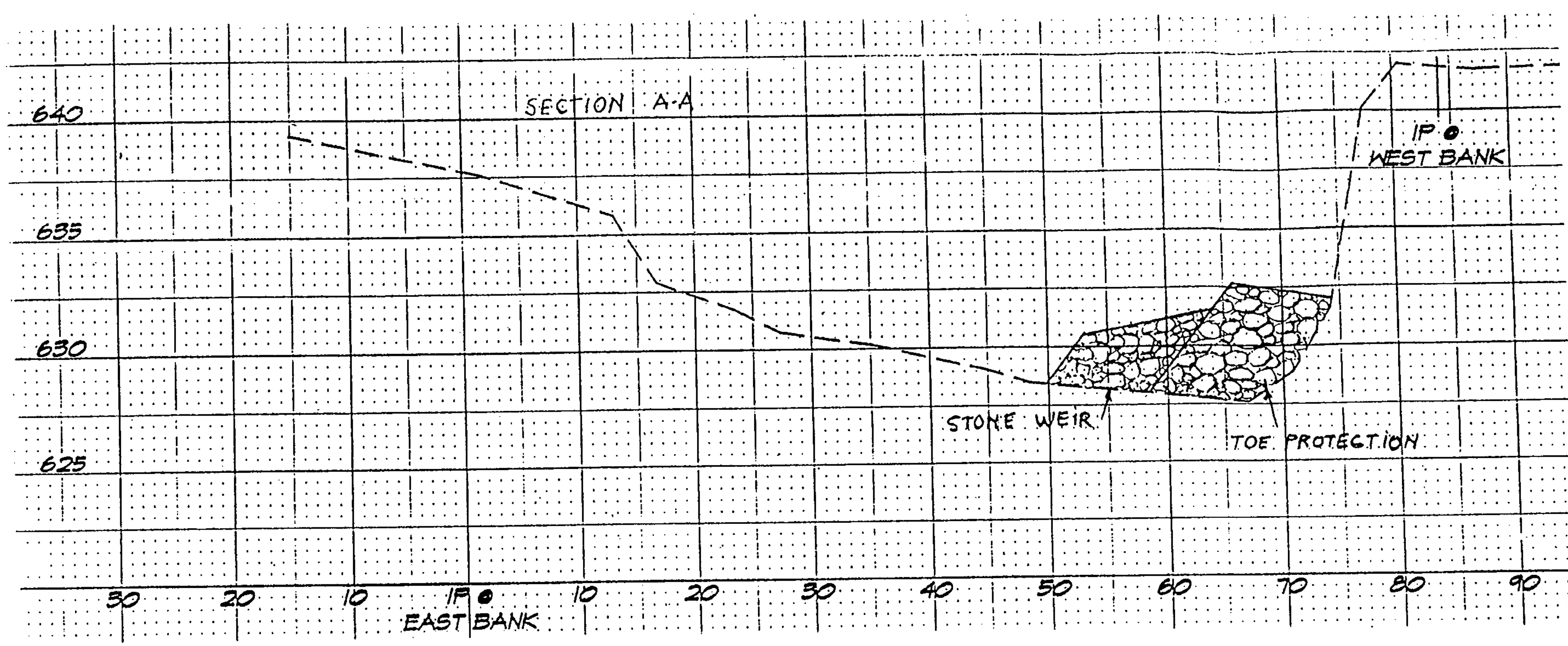

Figure 7-10. Spoon River alternative 1 protection design - typical cross-section (Section AA).

Recommendations for a Field Stream Laboratory

One of the most striking facts resulting from this investigation, is the need to conduct more basic research on stream bank protection techniques under well controlled conditions. This could be accomplished with the help of a field stream laboratory. A meandering stream could be constructed in the field, having dimensions similar to those commonly observed for small streams in Illinois. With this set up, different techniques for controlling stream migration could be readily tested and monitored. Since the flow discharge would also be controlled, both in magnitude and time variation, it would be possible to test bank protection techniques over a wide range of flow conditions in a rather short period of time. This cannot be accomplished in nature, where one is at the mercy of the watershed characteristics and hydrologic conditions.

Another advantage of having a field stream laboratory, is that smaller streams are the ones that often experience problems associated with stream bank migration. Such streams are also the ones that stand to benefit the most from low-cost protection measures, which can be truly tested only under well controlled conditions. For example, benway weirs are being favored as a low-cost technique for bank protection, but they have not really been tested over a range of conditions wide enough to allow for the development of design recommendations based on the hydraulics, hydrology, sedimentology, and morphology of a given stream.

The field stream laboratory could also be used for teaching purposes. Stream gaging and monitoring techniques could be taught in this facility, giving the students very valuable hands-on experience in stream hydraulics, hydrology, and sedimentation.

The purpose of this study is to investigate and recommend low-cost, low-maintenance methods for protection of bridges in small streams subject to channel migration threats. In this project, various available control techniques are reviewed, and the results serve as a reference for future designers as they try to match solutions to unique stream migration problems. In the process of selecting case study sites, a partial inventory of Illinois streams with channel migration problems has been developed. In general, the main type of problem encountered throughout Illinois is that of toe undercutting and bank loss through drop-off or mass failure. Four example sites were selected, and countermeasures to channel migration are proposed in a level compatible with current practice. These preliminary designs serve only to begin the design process and show use of those techniques discussed throughout the literature review. Furthermore, factors affecting channel migration are highlighted, and potential for improved methodologies is suggested. All the example sites have stream alignment problems and hence instream correction structures are desired, together with the size of the stream at the sites the effective coutermeasures may not be low cost.

This report attempts to approach stream migration from a practical viewpoint by identifying channel migration problems and posing solutions based on fundamental understanding. To select a means toward protecting any site, it is important to completely understand the site. In order to fully characterize a locale, information on the site's soil, land use, hydrology, geometry, and history is necessary. The use of aerial photos greatly helps to track the evolution of a stream over time. What the stream has done in the past can give clues to its future orientation. Each site is different, and a careful study of the area must be completed before a solution can be implemented.

A discussion of monitoring techniques is included in this report because these provide the only means to evaluate the success of bank protection measures. Some features of low-cost monitoring are summarized in Table 6-1. Cross-profiling and planimetric survey have been recommended because of their overall economic and practical feasibility. After stable protection is established, surveys are recommended on an interval of every 2-5 years. Again, each site's unique characteristics will dictate monitoring requirements.

The mechanisms which contribute to stream channel migration are many and complex. True understanding of bank migration requires integration of knowledge in hydrology, hydraulics, and geomechanics. Each site encountered presents a wide range of conditions. For these reasons, full comprehension of migration phenomena has yet to be attained, and a good deal of academic research must still be conducted. Nevertheless, the science has advanced sufficiently that now a holistic countermeasure integrating state-of-the-art technology accounting for both temporal and spatial variations of the flow and channel is possible for cost-effective low maintenance control of channel migration and bridge protection. Important features of low cost control measures are summarized in Table 7-1 for preliminary quick reference. Selection of a representative problem site, such as

Spoon or Mackinaw Rivers near Congerville, to develop a demonstrative effort of such techniques will be beneficial.

The development of a field stream laboratory for studying channel migration and stream bank protection techniques at the University of Illinois is also a possibility worth considering. A number of techniques could be tested under well controlled conditions at a scale representative of the field environment.

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Appendix I: IDOT Riprap Guidelines

The following documents have been take from the Illinois Department of Transportation's *Drainage Manual* . They serve to define riprap gradations (RR-5, etc.) and practices discussed in this report for those unfamiliar with IDOT standard procedures.

10.302.021 Riprap

According to F.H.W.A. Hydraulic Engineering Circular No. 11 (HEC 11), "Use of Riprap for Bank Protection," riprap is defined as: "A flexible channel or bank lining or facing consisting of a well graded mixture of rock, broken concrete, or other material, usually dumped or hand-placed, which provides protection from erosion."

Stone shape is an important factor in the selection of an appropriate riprap material. In general, riprap constructed with angular material has the best performance. Round material can be used as riprap provided it is not placed on slopes greater than 3:1. Flat slab-like stones should be avoided since they are easily dislodged by the flow. Broken concrete must be adequately fragmented before it is acceptable as a riprap material. An approximate guide to stone shape is that neither the breadth or thickness of a single stone should be less than one-third its length.

The successful use of rubble as riprap requires good control on material quality. The shape of rubble riprap is often a problem (particularly concrete rubble). The length to width ratio of any riprap material should be 3:1 or less.

The recommended placement method for rubble riprap is plating. Plated riprap is placed on the bank with a skip and then tamped into place using a steel plate, thus forming a regular, well organized surface. Plating permits the use of smaller stone sizes when compared with loose riprap.

The potential for scour at the toe of riprapped slopes should be studied because stabilizing a channels' banks will in most instances cause a deepening of the channel. Bank stabilization has been observed to increase the maximum-to-average depth ratio to approximately 1.7.

The longitudinal limits of protection required for bank protection at bends in channels are recommended to be a minimum distance of 1.0 channel width upstream and a minimum distance of 1.5 channel widths downstream with reference lines that pass through tangents to the bend at the bend entrance and exit. Review of existing bank protection sites has revealed that a common misconception in stream bank protection is to provide protection too far upstream and not far enough downstream (See figure 10-302.021).

The design height of a riprap installation should be equal to the design highwater elevation plus some allowance for freeboard. As a minimum, it is recommended that a freeboard elevation of 3 feet be used in unconstricted reaches and 4 feet in constricted reaches.

Typical channel cross sections that show recommended toe and flank details for riprap installations are on Figures 10-302.021a and b. Extra care should be taken to anchor the top of the riprap slope if overtopping of the banks is predicted.

Slopes to be protected by riprap should be free of brush, trees, stumps, and other objectionable materials and be dressed to a smooth surface. All soft or spongy material shall be removed to the depth shown on the plans or as directed by the engineer and replaced with approved native material. Filled areas will be compacted and a toe trench as shown on the plans shall be dug and maintained until the riprap is placed.

A tentative design relationship for the design of riprap is the permissible velocity procedure. Under the permissible velocity approach the channel is assumed stable if the computed mean velocity is lower than the maximum permissible velocity. This relationship is given in the following equation for riprap at bridge piers and abutments.

$$D_{50} = \frac{.692(1.9V)^2}{(S-1)2g}$$

where D_{50} = average stone diameter (ft)
V = the average velocity in the main channel
S = specific gravity of riprap material (normally 2.65)
g = 32.2 ft./sec.^2

Although, velocity relationships such as this have been used in the past, a more realistic model of detachment and erosion processes is based on tractive force concepts. These concepts are explained in H.E.C. 11, "Use of Riprap for Bank Protection."

The design procedure and sizing of riprap for raodside channels are presented in FHA Hydraulic Engineering Circular No. 15 (HEC 15), "Design of Roadside Channels With Flexible Lining" and the sizing and length of riprap for use downstream of culverts are presented in FHA Hydraulic Engineering Circular No. 14 (HEC 14), "Hydraulic Design of Energy Dissipators for Culverts and Channels".

The gradation of riprap specified for highway projects shall meet one of the gradation separations in Table 10-302.021 and shall meet the requirements of Article 705.01 of the Standard Specifications. This required size should then be converted to one of the standard gradations. The standard gradation (comparing the 50% passing size to the computed size) which equals or exceeds the computed size should be selected.

The standard gradations specify the design riprap size in terms of weight (W_{50}) instead of equivalent diameter, D_{50}. The following equation provides a relationship for converting the equivalent diameter to the corresponding rock weight.

$$W_{50} = D_{50}^{3} \cdot \gamma_s \cdot \gamma_w / 1.91$$

where

γ_s = the specific weight of the riprap material. (Typically 2.6 lbs./ft.^{3} to 2.7 lbs./ft.^{3})

γ_w = the specific weight of water (62.4 lbs./ft.^{3})

This equation is presented in nomograph form in chart 10.302.021d.

STONE GRADATION AND SIZE FOR EROSION PROTECTION

BEDDING MATERIAL

Gradation Number	Sieve Size	Percent Passing	Gradation Number	Sieve Size
RR 1	3" 1-1/2" #4	100 53±23 8±8	RR 2	4" 2" #4

STONE RIPRAP

Gradation Number	Rock Size (lb.)	Percent Passing	Gradation Number	Rock Size (lb.)
RR 3	50 10 0.5	100 50±20 8±8	RR 6	600 170 6
RR 4	150 40 1	100 50±20 8±8	RR 7	1000 300 12
RR 5	400 90 3	100 50±20 8±8		

Table 10-302.021

The thickness of the riprap layer is vital to the successful performance of riprap placed for erosion protection. The minimum thickness of the stone riprap layer to be used on highway projects is specified in Section 601 of the Standard Specifications for Road and Bridge Construction as follows:

Gradation	Min. Thickness	Bedding Thickness
RR1 & RR2	6 inches	-
RR3	8 inches	-
RR4	16 inches	6 inches
RR5	22 inches	8 inches
RR6	26 inches	10 inches
RR7	30 inches	12 inches

Additionally, the following criteria apply to the design of the riprap layer thickness:

1. It should not be less than the spherical diameter of the upper limit D_{100} (W_{100}) stone, or less than 1.5 times the spherical diameter of the upper limit D_{50} (W_{50}) stone, whichever results in the greater thickness.

2. It should not be less than 12 inches for practical placement.

3. The thickness determined by either 1 or 2 should be increased by 50 percent when the riprap is placed under water to provide for uncertainties associated with this type of placement.

4. An increase in thickness of 6 to 12 inches, accompanied by an appropriate increase in stone sizes, should be provided where riprap revetment will be subject to attack by floating debris or ice, or by waves from boat wakes, wind, or stream bed forms.

5. For rock riprap, the Manning n value varies with mean stone size, as follows:

$$n = 0.0395\ D_{50}^{1/6}$$

Thus, the following n values apply for common stone sizes:

D_{50} (ft.)	n
0.25	0.0314
0.50	0.0352
0.75	0.0377
1.00	0.0395
1.50	0.0423

Where n = Manning coefficient of channel roughness; see Chapter 4 "Open Channel Flow" for a discussion of the Mannings Equation for velocity of flow in open channels.

10-302.022 Design of Riprap Bedding or Filter Fabric

A granular bedding or filter fabric is to be placed under all riprap designed for erosion protection. Article 601.04 of the Standard Specifications requires that a filter fabric be placed under riprap gradations RR4, RR5, RR6 and RR7 and that a layer of granular bedding material be placed on the fabric before the riprap is installed.

A filter is a transitional layer of gravel, small stone, or fabric placed between the underlying soil and the structure. The filter prevents the migration of the fine soil particles through voids in the structure, distributes the weight of the armor units to provide more uniform settlement, and permits relief of hydrostatic pressures within the soils.

In the following relationships, filter refers to the overlying material and base refers to the underlying material. The relationships must hold between the filter blanket and base material and the riprap and filter blanket. Filters designed by the following criteria have been field evaluated and found to perform very well.

For a granular filter bedding the following criteria should be met:

$$\frac{D_{15}\text{ filter}}{D_{85}\text{ base}} < 5 < \frac{D_{15}\text{ filter}}{D_{15}\text{ base}} < 40$$

and

$$\frac{D_{50}\text{ filter}}{D_{50}\text{ base}} < 40$$

The filter gradation in Table 10-302.021 which best satisfies the above relationship should be specified for design. For riprap gradations RR6 and RR7, it may be necessary to use two filter gradations.

In selecting an engineering filter fabric, the fabric should be able to transmit water from the soil and also have a pore structure that will hold back soil. The following properties of an engineering filter fabric are required to assure that the performance is adequate as a filter under riprap.

1. The fabric must be able to transmit water faster than the soil.
2. The following criteria for the apparent opening size (AOS) must be met:
   (a) Soil with less than 50 percent of the particles by weight passing a U.S. No. 200 sieve, AOS < 0.6 mm (0.024 in) (greater than #30 US. Std. sieve).
   (b) Soil with more than 50 percent of the particles by weight passing a U.S. No. 200 sieve, AOS < 0.297 mm (0.012 in) (greater than #50 US Std. Sieve).

The above criteria only applies to non-severe or non-critical installations. Severe or critical installations should be designed based on permeability tests.

A 4 to 6 inch layer of aggregate bedding should be placed on top of geotechnical fabric to protect it during the placement of riprap. The use of gravel for protection of the geotechnical fabric is not recommended due to the greater potential for slippage of the riprap. Crushed rock is preferred for protection of geotechnical fabric due to its angular characteristics which provide greater friction to prevent slippage.

The use of woven fabrics is not recommended for use under riprap due also to the potential for slippage of the riprap on it. Non-woven fabrics are preferable for use under riprap because of their greater friction characteristics.

Caution should be used when using filter fabric under riprap at locations where a concentrated flow of water can be expected to be directed onto the riprap (such as at the downstream end of culverts or where side ditches intersect streams). Failures can occur at these locations due to slippage of the riprap on the filter fabric, especially when gravel is used for bedding and/or the channel slopes are steeper than 3:1. To alleviate the slippage of the riprap, in such cases, a concrete slurry grout or fabric concrete revetment mats are recommended.

The use of geotechnical fabric should be considered for installations that would require two or more layers of granular bedding. The use of geotechnical fabric has also been determined to be cost effective in areas where a good source of suitable bedding is not convenient.

Geotechnical Fabric Placement. To provide good performance, a properly selected cloth should be installed with due regard for the following precautions:

- Heavy riprap may stretch the cloth as it settles, eventually causing bursting of the fabric in tension. A 4 inch to 6 inch aggregate bedding layer should be placed beneath the riprap layer to protect the fabric.

- The filter cloth should not extend channelward of the riprap layer; rather, it should be wrapped around the toe material as illustrated in figure 10-302.021b.

- Adequate overlaps must be provided between individual fabric sheets. For lightweight revetments this can be as little as 12 inches, and may increase to as much as 3 feet for large underwater revetments.

- A sufficient number of folds should be included during placement to eliminate tension and stretching under settlement.

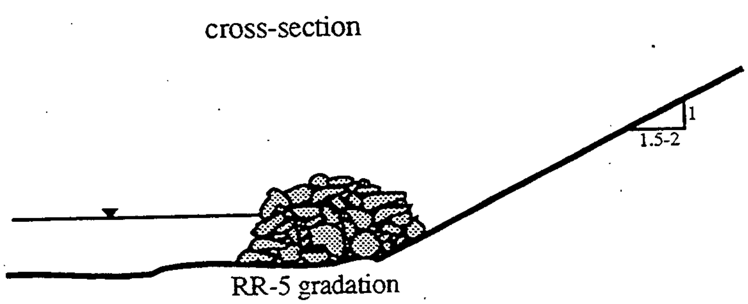
- Securing pins with washers are recommended at 2- to 5-foot intervals along the midpoint of the overlaps.

- Proper stone placement on the filter requires beginning at the toe and proceeding up the slope. Dropping stone from heights greater than 2 feet can rupture fabrics (greater drop heights are allowable under water).

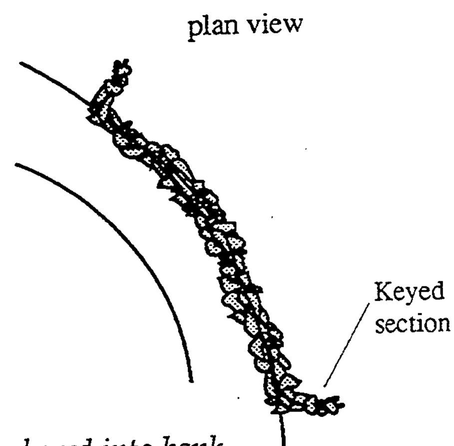
I. Site Preparation

Bank Shaping. The existing stream bank should be reshaped to a slope of 1.5H:1V or flatter using a track hoe or similar equipment. A bank slope of 2H:1V has been used successfully in many projects involving willow posts. Steeper slopes often result in large willow mortality rates .

Toe Protection. Stream bank toe protection is essential to the success of willow post stabilization projects. Longitudinal peaked stone toe protection (LPSTP) has been used successfully in many willow post projects. LPSTP involves piling riprap at its natural angle of repose along the foot of the stream bank. RR-5 gradation riprap is usually used and is applied at the rate of 1 ton per lineal foot of stream bank. At the upstream and downstream project ends, the riprap is keyed into the bank to prevent the flow from scouring behind the riprap.



LPSTP method of toe protection



Riprap keyed into bank

II. Plant selection and handling

Species. Use native willows in good condition. Black Willow, sandbar Willow, and Carolina Willow have all been used with success when planted in ground that is usually saturated (region of bank near stream).

Size. Posts should be long enough to ensure that they will extend below the water table when planted. Generally, a post length of 10-14 ft is specified for willow post stabilization projects in Illinois. The post top diameters are usually greater than 3"; top diameters in projects range from 1.5" to 6".

Cutting. Posts meeting the specifications above should be cut and planted within 48 hours. The willows must be cut when dormant, which is the period after the leaves have fallen off but before new leaf buds appear. In Illinois this dormancy generally falls between January 1 and March 1. Once cut, the willows should be pruned of any branches and a marker should be placed on the tops of the posts to ensure upright planting. Between cutting and planting, the willows should be soaked in water.

III. Planting

Location. The willows are usually planted in the lowest portions of the stream bank. Sometimes willows are planted submerged in the stream bed. The extent of the willows up the stream bank depends on the degree of protection desired and the slope of the bank (water table); the willows should extend to the water table to ensure survival.

Spacing. The willows are planted in a grid, either staggered or evenly spaced. The dimensions of the grid range from 2'x2' to 4'x4'. Three rows of willows, in a 3ft x 3ft grid, are often used, with the first row beginning just above the waterline. The spacing chosen will depend on the degree of protection desires, the size of the willows planted, and the availability of willows for planting.

Holes. The planting holes for the posts are bored using either an auger or metal ram with a flat (not pointed) end. Metal rams attached to excavators have also been used to construct the holes. The hole diameters range from 8" to 12" and are drilled to depths of 5-10 ft, depending on the willow size. The bottoms of the holes should extend below the normal water table.

Placement. The willows should be placed, top side up, into the holes so that no space remains between the bottom of the willow and the hole. Once placed, the holes should be backfilled with dirt, and this backfill should be compacted using a metal rod or similar tool to remove any air pockets. The planted willows should be watered with a hydraulic pump.

IV. Other Considerations

Erosion control.. Designs often include measures to reduce soil erosion around the posts. Grass is often planted among the willows to reduce scour around the willows. Alternatively, dormant cuttings or stakes can be planted among the willows as erosion control measures.

United States Department of Agriculture - Soil Conservation Service
Stream Bank Stabilization Site Evaluation Worksheet
Woodland Technical Note - IL-16

1. Determine channel bottom to low bank height difference:

   A. Elevation of low bank at lowest point on its profile =
   B. Elevation of channel bottom at lowest point of channel scour =
   C. Difference in elevation (A-B) =

2. Determine minimum length of "stake" or "post" required:

   A. If C exceeds 5 feet - Use a minimum length of 7 feet.
   B. If C is less than 5 feet - Use a minimum length equal to C.

3. Determine the ratio of horizontal distance to vertical height of the streambank (low bank side) for row arrangement to be used (see figure 4):

   Example: Horizontal distance of streambank equals 12 feet
   Vertical height of streambank equals 6 feet
   Horizontal distance to vertical height ratio equals 12:6 or 2:1
   Row arrangement would be designed using PRA-2 as a minimum

Horizontal Distance to Vertical Height Ratio	Minimum Planting Row Arrangement to Use
Steeper than 2:1	PRA-1
2:1 to 5:1	PRA-2
Flatter than 5:1	PRA-3

4. Will it be necessary to regrade the streambank to a different ratio?

   If yes:

   A. To what ratio will the completed project be graded? _____:_____
   B. Based on "A" above, what PRA will be used? PRA-_____
   C. What equipment will be needed to complete the regrading?

5. Determine soil type(s) at the project site:
   A. Predominant soil type(s) in the riparian zone for design purposes:________________
   B. Other major soil types present:_______________________________
   C. Determine Woodland Planting Group(s) to use from Section II-F of the Field Office Technical Guide.

	Predominant Soil Type(s)	WPG	Species to be Used
1.	____________________	_____	____________________
2.	____________________	_____	____________________
3.	____________________	_____	____________________

D. Are predominant soil type(s) considered stable if properly protected? _______
   If no, what alternative actions might be needed to assure stability?

6. Determine stream channel bottom stability:

   A. Does stream channel bottom have "overfalls" in or immediately below the project site? ______

   B. Does the stream channel bottom continuously fluctuate in elevation due to scour holes? ______

   C. Does the stream have segments of rushing water in an otherwise tranquil stream flow? _____

If any of the above can be answered "yes," careful evaluation by an experienced hydrologist may be needed before designing or installing this project.

7. Determine cost estimates: 1/

   A. Equipment:_____ hours x _____/hour $___________
   B. Labor: _____ hours x _____/hour $___________
   C. Materials:_______________________________ $___________

   _______________________________ Total $________

1/Average cost of $77/100' length for sloping 12' high bank to 1:1 slope. Cost per hole @ $2.40/6' post and $2.90/9' post. Average of 10 posts/person/hour labor costs for cutting and transporting posts.

Appendix III: Flood Peak Discharge Estimates

The following are excerpts from the U.S.G.S report *Techniques for Estimating Flood-Peak Discharges and Frequencies on Rural Streams in Illinois* (Curtis, 1987). These documents outline the procedures used to estimate floods for sites on ungaged streams such as those described in this report.

Particular attention is drawn to Table 4 which relates the accuracy of this estimating technique.

For the four selected sites in this study the drainage area, basin slope, 2-year, 24 hour rainfall depth and regional factor for different flood frequencies were determined and listed in Table A3-1. Accordingly, the peak discharges for return period (in years) of 2, 5, 10, 25, 50, and 100 can be estimated by using Curtis (1987) Eqs. 1-6, and the results are given in Table A3-1.

Table A3-1. Flood Magnitudes at Sites Investigated Using USGS Method Eqs 1-6 of Curtis (1987)

Site	Area (mi^2)	Slope (ft/mi)	Rainfall (in)	Return Period (yr)	Factor	Peak Discharge (cfs)
Cahokia Creek	151.32	5.29	3.4	2	1.057	4407
				5	1.053	7471
				10	1.053	9586
				25	1.051	12340
				50	1.05	14365
				100	1.048	16378
				500	1.044	21155
Piasa Creek	98.01	5.76	3.4	2	1.057	3258
				5	1.053	5548
				10	1.053	7133
				25	1.051	9193
				50	1.05	10715
				100	1.048	12223
				500	1.044	15806
Senchwine Creek	88.63	10.33	3.1	2	0.931	2667
				5	0.9375	4601
				10	0.945	5974
				25	0.952	7779
				50	0.956	9122
				100	0.959	10459
				500	0.965	13648
Spoon River	203.36	3.93	3.1	2	0.805	2792
				5	0.822	4720
				10	0.837	6073
				25	0.853	7854
				50	0.862	9143
				100	0.87	10434
				500	0.886	13499

TECHNIQUE FOR ESTIMATING FLOOD-PEAK DISCHARGES AND FREQUENCIES ON RURAL STREAMS IN ILLINOIS

By G. W. Curtis

ABSTRACT

Flood-peak discharges and frequencies are presented for 394 gaged sites in Illinois, Indiana, and Wisconsin for recurrence intervals ranging from 2 to 100 years. A technique is presented for estimating flood-peak discharges at recurrence intervals ranging from 2 to 500 years for nonregulated streams in Illinois with drainage areas ranging from 0.02 to 10,000 squares miles. Multiple-regression analyses, using basin characteristics and peak streamflow data from 268 of the 394 gaged sites, were used to define the flood-frequency relation. The most significant independent variables for estimating flood-peak discharges are drainage area, slope, rainfall intensity, and a regional factor. Examples are given to show a step-by-step procedure in calculating a 50-year flood for a site on an ungaged stream, a site at a gaged location, and a site near a gaged location.

INTRODUCTION

The purpose of this report is to provide updated station flood-peak discharges and frequencies and to provide improvement to the previous techniques for estimating flood-peak discharges and frequencies of floods for sites on most streams where flood discharges are not significantly affected by regulation or urbanization. Flood-peak discharges and frequencies are presented for 394 gaging stations in Illinois, Indiana, and Wisconsin for recurrence intervals of 2, 5, 10, 25, 50, and 100 years. A technique using drainage area (A), slope (S), rainfall intensity (I), and regional factor (Rf) was developed for estimating flood-peak discharges at ungaged sites in Illinois. Equations using these variables are applicable for estimating flood-peak discharges for recurrence intervals of 2 to 500 years for drainage areas ranging from 0.02 to 10,000 square miles (mi^2) on nonregulated rural streams. Estimates of future floods are necessary for the proper design of engineering projects such as bridges, culverts, highways, and flood-control structures; for establishment of actuarial flood-insurance rates; and for proper flood-plain management by State and local agencies.

Previous techniques for estimating flood-peak discharges and frequencies in Illinois have been provided by Mitchell (1954), Speer and Gamble (1965), Wiitala (1965), Patterson and Gamble (1968), Ellis (1968), Carns (1973), Curtis (1977a), and Allen and Bejcek (1979). Techniques were developed by Carns (1973), Curtis (1977a), and Allen and Bejcek (1979) using ordinary least squares multiple-regression analyses as recommended by Thomas and Benson (1970). Additional data and improved analytical methods used in this report increase the confidence in estimating techniques over those published in earlier reports.

This report was prepared under a cooperative agreement between the State of Illinois, Department of Transportation, Division of Water Resources, and the U.S. Geological Survey (Survey). Streamflow data were collected in cooperation with the U.S. Army Corps of Engineers and State and local agencies.

TECHNIQUE FOR ESTIMATING FLOOD-PEAK DISCHARGES

Annual peak discharges from gaging stations having a minimum of 10 years of record through the 1985 water year were used to define station flood-frequency relations. Locations of these stations are shown in figures 1 and 2. The map number, identification number, name, geographic location, and station flood-peak discharges for the stations are listed in table 1. All figures and tables are grouped in the back of the report for easy reference.

Station flood-frequency relations were defined using the Hydrology Subcommittee of Interagency Advisory Committee on Water Data (1982), formerly U.S. Water Resources Council, guidelines. These guidelines outline procedures to fit the logarithms of observed annual peak discharges to the Pearson Type III frequency distribution.

Peak discharges of various recurrence intervals and basin characteristics for gaging stations were used in multiple-regression analyses to develop estimating equations for flood-peak discharges and frequencies on nonregulated rural streams in Illinois. Data from stations affected by either urbanization or by regulation were not included in the regression analyses. Relations were developed for estimating flood-peak discharges corresponding to the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year recurrence-interval flood (T-year flood or Q_T). The regression analyses indicated that the independent variables--drainage area (A), slope (S), rainfall intensity (I), and regional factor (Rf)--are the most significant variables to use in estimating flood-peak discharges for Illinois streams. One estimating equation for each recurrence interval provides a straightforward technique to compute flood-peak discharges for both small and large Illinois streams. Flood-peak discharges and frequencies, basin characteristics and other pertinent data, and regional factors are tabulated in tables 1, 2, and 3. In table 1, two sets of station flood-peak discharges are presented for two stations (Nos. 385 and 389) on the Big Muddy River. The first set of discharges are for periods of nonregulated flow and were used in the regression analyses. The second set of discharges are for periods of regulated flow and were not used in the regression analyses.

The reliability of flood-frequency estimates is uncertain for very large recurrence intervals. Because of this uncertainty, the 500-year flood discharges are omitted from table 1. An estimating equation for the 500-year flood is provided primarily for planners who are required to compute this event for special purposes such as flood-insurance studies. Only those stations used in the regression analyses are listed in table 2.

The flood-frequency and the regression analyses, used to develop the estimating technique, are defined in detail in the data-analyses section.

Flood-peak discharge equations, applicable statewide, for estimating Q_T on nonregulated rural streams are as follows:

$$Q_2 = 38.1\ A^{0.790}\ S^{0.481}\ (I-2.5)^{0.677}\ Rf \tag{1}$$

$$Q_5 = 63.0\ A^{0.786}\ S^{0.513}\ (I-2.5)^{0.719}\ Rf \tag{2}$$

$$Q_{10} = 78.9\ A^{0.785}\ S^{0.532}\ (I-2.5)^{0.742}\ Rf \tag{3}$$

$$Q_{25} = 98.2\ A^{0.786}\ S^{0.552}\ (I-2.5)^{0.768}\ Rf \tag{4}$$

$$Q_{50} = 112\ A^{0.786}\ S^{0.566}\ (I-2.5)^{0.786}\ Rf \tag{5}$$

$$Q_{100} = 125\ A^{0.787}\ S^{0.578}\ (I-2.5)^{0.803}\ Rf \tag{6}$$

$$Q_{500} = 155\ A^{0.789}\ S^{0.601}\ (I-2.5)^{0.838}\ Rf \tag{7}$$

The four variables required to solve the equations are drainage area (A), slope (S), rainfall intensity (I), and regional factor (Rf). Drainage area and slope are determined from topographic maps. Drainage area is the area contributing to surface runoff. Slope is determined between points 10 percent and 85 percent of the total distance measured along the low-water channel from the site to the basin divide. The rainfall intensity is determined from figure 3. The regional factor is determined by first selecting the region number from figure 4 and then the appropriate regional factor from table 3.

Flood-peak discharge equations for recurrence intervals between 2 and 100 years, other than those in equations 1 to 7, may be developed by interpolating the regression constant and coefficients from the graphs in figure 5.

APPLICATION OF ESTIMATING TECHNIQUE

The technique for estimating flood-peak discharges and frequencies is applicable to either ungaged or gaged nonregulated rural streams. Figure 6 shows the sequence to follow for estimating a flood-peak discharge at a site. Step-by-step procedures for applying the estimating technique are given in the examples that follow.

Site on Ungaged Stream

Flood frequency estimates at sites on ungaged streams are calculated using equations 1 to 7.

Example 1: Computation of the 50-year recurrence interval flood at a site on an ungaged stream:

1. Determine the size of contributing drainage area (A), in square miles. The area can be planimetered on topographic, county, or other maps suitable for delineating the basin boundary. For this example, assume A = 625 mi^2.

2. Determine the slope (S), in feet per mile (ft/mi). Slope is based on the difference of elevations divided by distance between points 10 percent and 85 percent of the total distance measured along the low-water channel of the stream from the site to the basin divide. For this example, assume S = 2.5 ft/mi.

3. Determine the rainfall intensity (I), in inches, from figure 3. The value of I should be an average for the basin. For this example, assume I = 3.1 inches.

4. Determine the region (R) and the regional factor (Rf) from figure 4 and table 3, respectively. For this example, R is III and Rf is 0.862.

5. Select equation 5 from page 3 and compute the flood magnitude.

$$Q_{50} = 112\ A^{0.786}\ S^{0.566}\ (I-2.5)^{0.786}\ Rf$$

$$= (112)(625)^{0.786}(2.5)^{0.566}(3.1-2.5)^{0.786}(0.862)$$

$$= (112)(157.6)(1.68)(0.669)(0.862)$$

$$= 17{,}100\ ft^3/s.$$

Site at Gaged Location

Flood frequency estimates at gaged sites are combinations of the gaging station frequency curve and the equation estimates. The equivalent years of record concept (Hardison, 1971) was used to obtain weighted estimates of peak flow at gaged sites using estimates obtained from station records and from equations 1 to 7. This procedure was described by the Hydrologic Subcommittee (1982) and is expressed in the equation

$$\text{Log } Q_T = \frac{\text{Yrs of record (log sta. } Q_T) + \text{Eq yrs record (log regional } Q_T)}{\text{Yrs of record + Eq yrs record}}. \quad (8)$$

In equation 8, station Q_T is obtained from the first line of discharge values in table 1 and converted to a logarithm (log). The years of record are determined from table 2. The regional Q_T is computed using the desired regional estimating equation on page 3 or obtained from the second line of discharge values in table 1 and then transformed into logs. The station equivalent years of record (Eq yrs record) for the equation are also given in table 2. The antilog of the result from equation 8 is the weighted estimate of the station flood discharge.

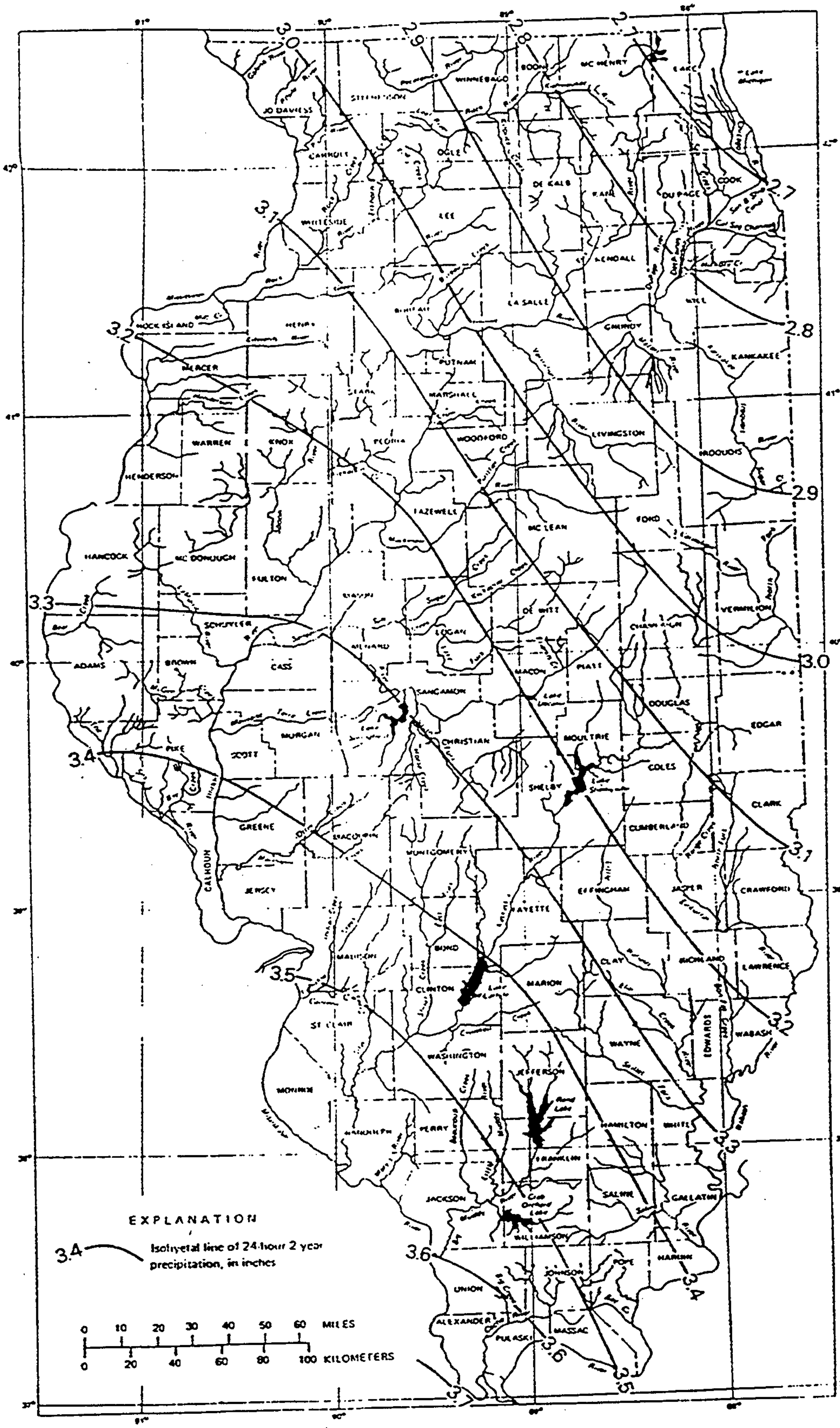


Figure 3.--Distribution of rainfall intensity (24-hour 2-year) I, in inches, in Illinois (modified from Hershfield, 1961).

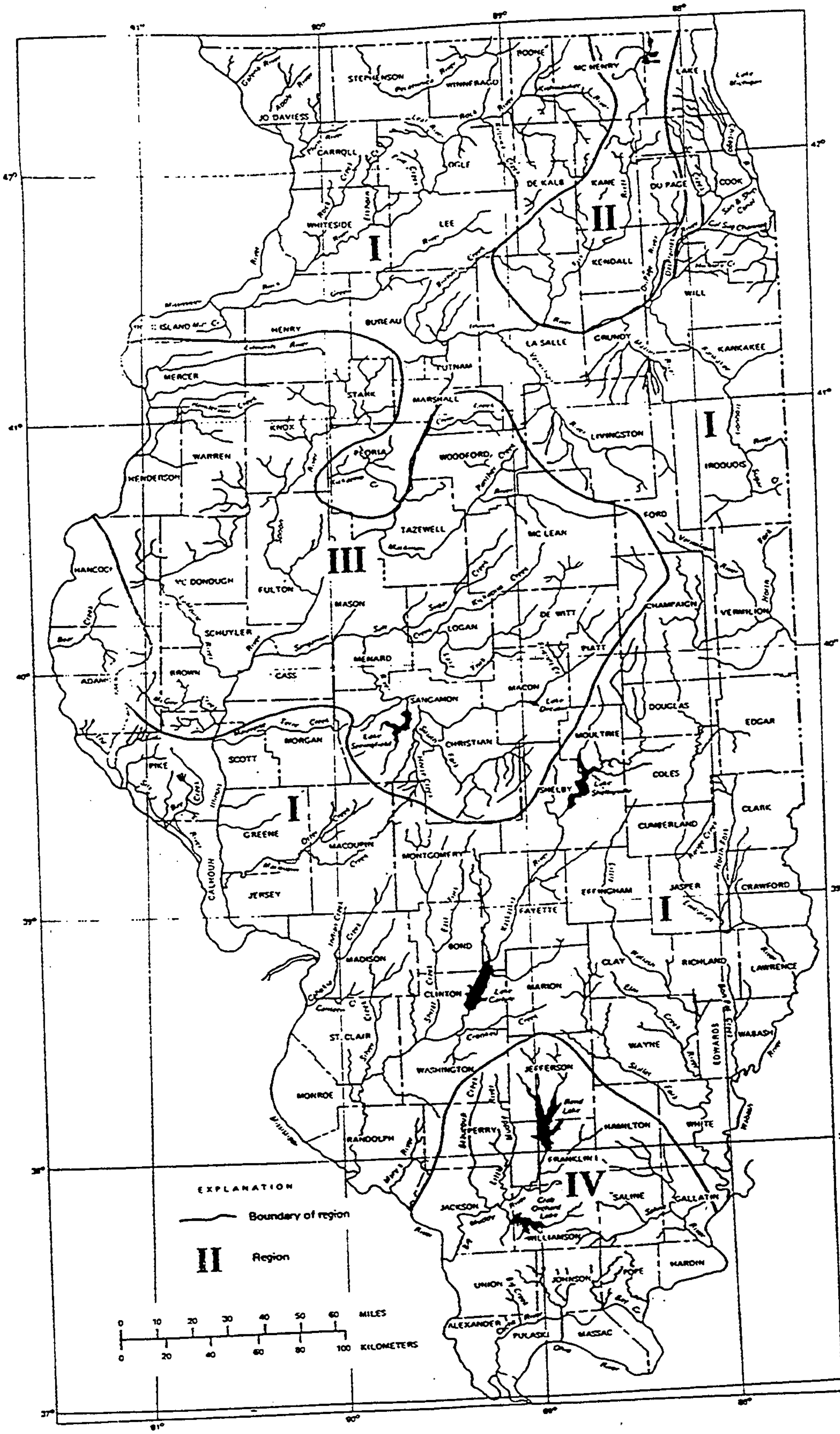


Figure 4.--Geographic regions, R, in Illinois.

Table 3.--Regional factors, Rf

[T-year, recurrence interval in years]

Flood, T-year	Region			
	I	II	III	IV
2	1.057	0.578	0.805	0.983
5	1.053	.576	.822	.894
10	1.053	.574	.837	.859
25	1.051	.570	.853	.826
50	1.050	.567	.862	.806
100	1.048	.563	.870	.790
500	1.044	.555	.886	.759

Table 4.--Accuracy of estimating equations, $Q_T = a\ A^b\ S^c\ (I-2.5)^d\ Rf$

[T-year, recurrence interval in years]

Flood T-year	Standard error of prediction	
	Percent	Equivalent years of record
2	34.9	3.36
5	33.0	4.54
10	34.9	5.36
25	38.0	6.18
50	40.6	6.58
100	43.4	6.84
500	50.3	7.05

Appendix IV: Determination of Flood Frequency at Cahokia Creek Site

There is no stream flow gaging station at the Cahokia Creek site. Thus, the flood magnitudes of different occurrence frequencies can be estimated only indirectly. There are three ways to estimate the floods. One way is to use a synthetic method. This approach is rather involved and is beyond the scope of this study. The second way is to use a regional formula/procedure, and for this site is the USGS technique for Illinois rural streams (Curtis, 1987). This method is outlined in Appendix III. The third way is to transpose the flood frrequency of a nearby gaging station along the same river. This is, relatively, the most reliable method provided the stream flow data are available at nearby stations. This is the case for the Cahokia site.

The USGS gaging station No. 05587900 at Highway 143 bridge at Edwardsville drains 212 square miles. The streamflow record available covers the period from August 1969 to the end of 1995. The annual maximum series of the peak discharge data from this record is analyzed by using Gumbel extremes Type I distribution and log-Peason III (LP3) distribution. The results are summarized in Table A4-1. The Gumbel distribution yields a much better fit to the data--coefficient of variation (COV) around 0.12 for Gumbel vs. 0.35 for LP3 for large return periods and comparable COV (around 0.08) for both distributions for small return periods.

The USGS provides a weighted adjustment for estimated peak discharge from regional regression equations if streamflow record is available. Among the four selected sites, Cahokia Creek is the only site that has a nearby gauging station at Edwardsville. Using 17 years of streamflow record, USGS determined the peak discharges at Edwardsville based on frequency analysis, regional regression equations and weighted from the following formula:

$$\mathrm{Log}(Q_t)_{wt} = \frac{X(\log(Q_{t-\mathrm{station}}))+Y(\mathrm{Log}(Q_{t-\mathrm{regional}}))}{X+Y}$$

in which X is the number of years of record at the station used for frequency analysis and Y is the equivalent number of years of record. The estimated discharges and Y as determined by USGS (Curtis, 1987) are listed in column 2 to 5 in Table A4-2. By using the 26-year frequency analysis values instead of those from the 17 years, the weighted results are given in Columns 8 and 9 in Table A4-2 for Gumbel and LP3 distributions, repectively. It can be observed that LP3 estimates are far below the corresponding floods estimated from other methods. Figure 5-1 shows that for Edwardsville at large return periods LP3 does not follow the trend of the data. Therefore, the direct 26-year annual maximum series frequency analysis result of the Gumbel distribution at Edwardsville is accepted as the data to be transposed to the study site.

This site is located 5.5 miles upstream of the gauging station and at this site Cahokia Creek drains 151 square miles. Therefore, the flood at the gauging station should be adjusted and reduced to reflect the smaller drainage area.

Usually the flood peak ratio is proportional to the drained area ratio to the power of 0.8 to 0.9. A statistical analysis performed at the University of Illinois at Urbana - Champaign for 81 Illinois watersheds gives a value of 0.88 (Yen, 1984). Curtis (1987) recommended an exponent of 0.79 for his flood equations for Illinois. Using this value the peak discharge at the site is computed as

$$(Q_t)_{site} = \left[\frac{A_{site}}{A_{gauge}}\right]^{0.79} (Q_t)_{gauge}$$

The result is given in Table A4-1 in the last column and Table 4-2 in the main body of the report.

Table A4-1. Flood Magnitude of Cahokia Creek Determined from Frequency Analysis of 1970-1995 Annual Maximum Series at USGS Guaging station at Edwardsville

Return Period (yr)	Peak Discharge (cfs) @ Edwardsville		Peak Discharge (cfs) @ Site
	Gumbel	LP3	Gumbel
2	4841	5436	3703
5	6534	6852	4998
10	7655	7262	5855
25	9071	7482	6938
50	10122	7537	7742
100	11165	7552	8540

TABLE A4 - 2. Flood Magnitude of Cahokia Creek at Edwardsvill Gauging Station Estimated by Using USGS Regional Weighted Method (Eq. 8 of Curtis, 1987)

Return Period (yr)	Peak Discharge (cfs) estimated by Curtis (1987)			Equivalent No.Years Of Record	Peak Discharge (cfs) From Freq. Analysis of 26-yr data			
					Distribution		Weighted	
	Freq. Analysis of 17-yr. data	Computed from Regression Eqs.	Weighted		Gumbel	LP3	Gumbel	LP3
2	4780	5660	4910	3.4	4841	5436	4929	5461
5	6700	9570	7110	3.4	6534	6852	6829	7122
10	7780	12300	8470	3.9	7655	7262	8143	7779
25	8930	15800	10100	4.5	9071	7482	9845	8354
50	9680	18400	11200	4.9	10122	7537	11128	8683
100	10300	21000	12200	5.2	11165	7552	12405	8955