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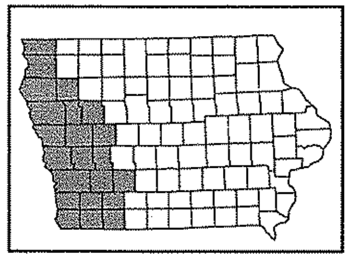
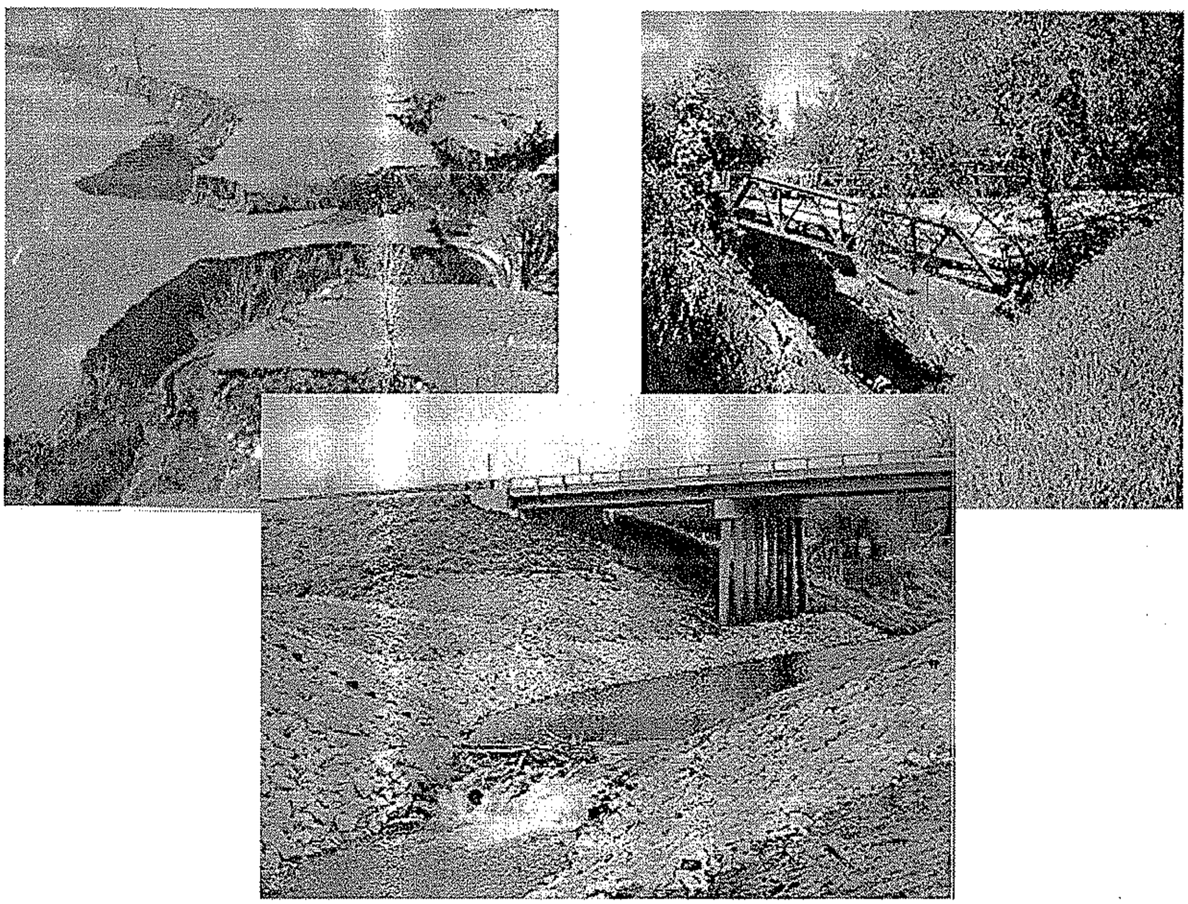


Loess Hills Development
& Conservation Authority
Oakland, Iowa

Design Manual

Streambed Degradation and Streambank Widening in Western Iowa



September 1998

Design Manual
Streambed Degradation and Streambank Widening in Western Iowa

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Chapter 1
Introduction

Authority

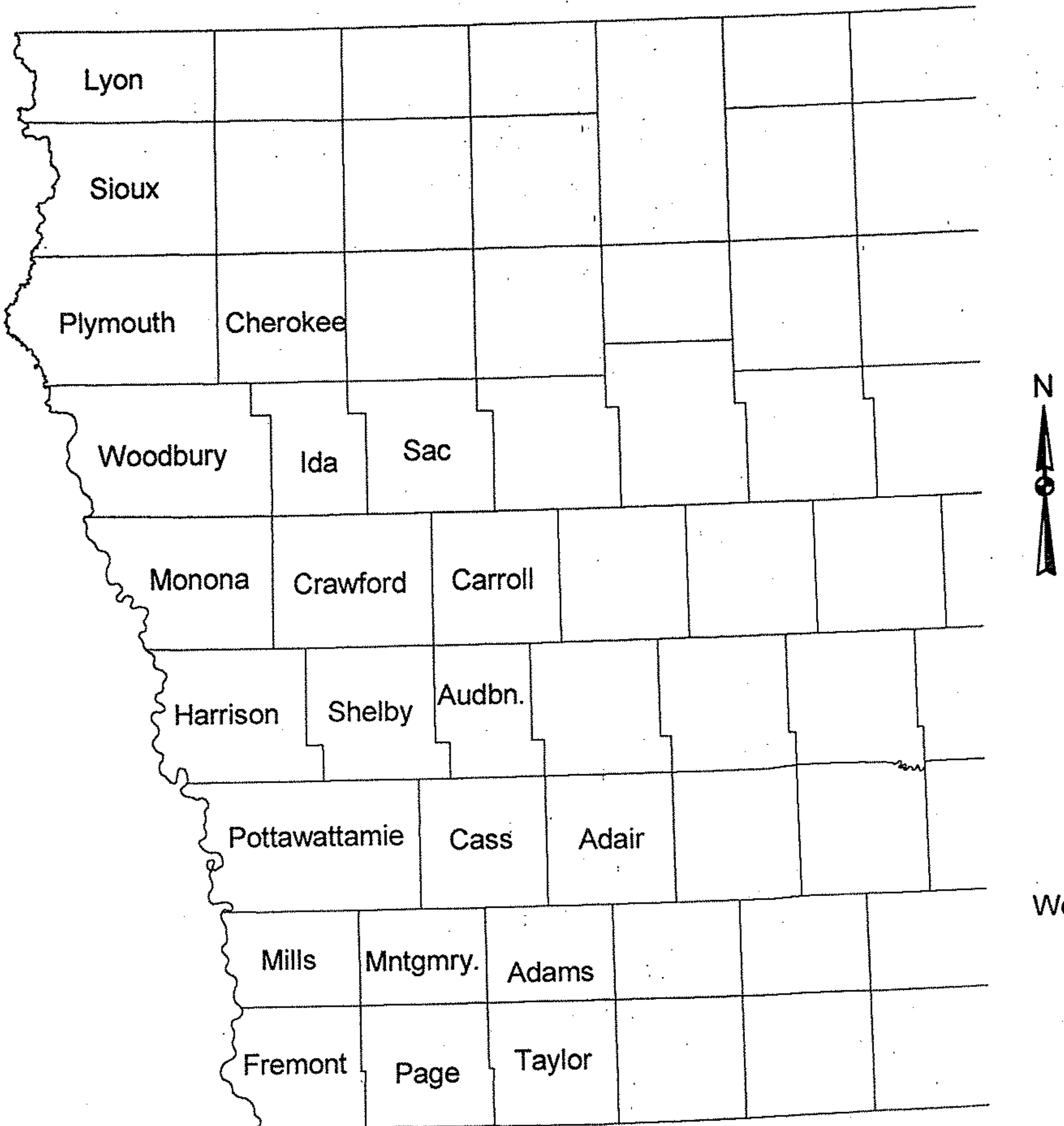
This design manual was developed under the Corps of Engineers' Planning Assistance to States Program, authorized by Section 22 of the Water Resources Development Act of 1974, as amended. The "Section 22" program authorizes the Corps of Engineers (COE) to cooperate with States, local entities, and Indian Tribes in the preparation of comprehensive plans for the development, use, and conservation of water and related land resources. This manual was prepared with the assistance of Golden Hills Resource Conservation and Development, Inc. (Golden Hills RC&D), Iowa State University (ISU), and the Natural Resources Conservation Service (NRCS). The following 22 western Iowa counties also provided valuable information for the development of this manual: Adair, Adams, Audubon, Carroll, Cass, Cherokee, Crawford, Fremont, Harrison, Ida, Lyon, Mills, Monona, Montgomery, Page, Plymouth, Pottawattamie, Sac, Shelby, Sioux, Taylor, and Woodbury.

The Iowa Department of Transportation (IDOT) provided funding for the Golden Hills RC&D to undertake and complete research and local management requirements for this manual. And as a cost-sharing participant in this research project, the opinions, findings, and conclusions expressed in this publication are those of the COE and not necessarily those of IDOT or the U.S. Department of Transportation, Federal Highway Administration.

Purpose and Scope

The purpose of this design manual is to evaluate various potential measures to address streambed degradation and streambank widening occurring along the primary rivers and their tributaries in western Iowa. Although streambed degradation has already taken place throughout much of the 22-county study area, the impacts to infrastructure, the high costs of repairs, and the reduction of Federal matching dollars to address degradation problems have brought this subject to the forefront of county concerns. Realizing that this problem is widespread and will continue to impact future infrastructure planning, the 22 counties of western Iowa, in cooperation with other Federal and State agencies, requested the development of a design manual to provide State and county officials the tools required to plan for the implementation of grade stabilization structures, common remedial measures, and the assessment of existing grade stabilization structures located within the study area. The study area is presented in Figure 1.

Western Iowa Counties



Western Iowa Stream Stabilization Study
Counties Included in Study

Figure 1

According to previous studies conducted by Golden Hills RC&D and ISU, damages to highway bridges represent the highest costs associated with streambed degradation and streambank widening, followed by damages to railroad bridges and right-of-way, loss of agricultural land, and utilities. Severe channel erosion occurs along more than 1,000 miles in 155 stream and river basins in western Iowa. The average time-neutral costs per county is $8.3 million. The estimated time-neutral costs associated with streambed degradation and streambank widening is $174.9 million and the total time- value costs are estimated to be $1.1 billion. The estimation of time-value costs is recognized as being more accurate of total damages because compound interest could have been earned during earlier years. The costs were determined from an evaluation of the economic losses, repairs, and social-economic changes in traffic rerouting for roadway and railway bridges, agricultural lands, and utilities.

Design Manual Organization

This design manual is organized into nine chapters, which provide general information on the 22-county area studied, the economic impact associated with streambed degradation,and standards or criteria for classifying the stage of degradation occurring in any particular stream. This manual also presents detailed criteria for determining future degradation and streambank widening as well as systematic site planning of structure implementation, remedial measures, and structure monitoring.

Chapter 1 (this chapter) provides background and general information concerning development of the design manual.

Chapter 2 describes the basins located within the 22-county study area, the general causes of streambed degradation and streambank widening, and a stream classification system for degrading streams and provides a review of bridges potentially susceptible to damages from streambed degradation and streambank widening.

Chapter 3 presents information regarding the economic impact associated with streambed degradation and streambank widening and the general design of grade stabilization structures and discusses several of the remedial measures commonly used in western Iowa today.

Chapter 4 discusses nonstructural measures such as board dams and debris-catcher dams.

Chapter 5 provides reference information on streambank stabilization projects.

Chapter 6 presents a systematic approach to site evaluation and grade stabilization planning. Tools for determining future streambed degradation and streambank widening are also provided.

Chapter 7 discusses structure performance through monitoring and evaluation.

Chapter 8 focuses on construction permit requirements at the State and Federal level.

Chapter 9 provides additional resources to be to contacted with questions regarding streambed degradation and streambank widening.

Chapter 2
Description of Study Area

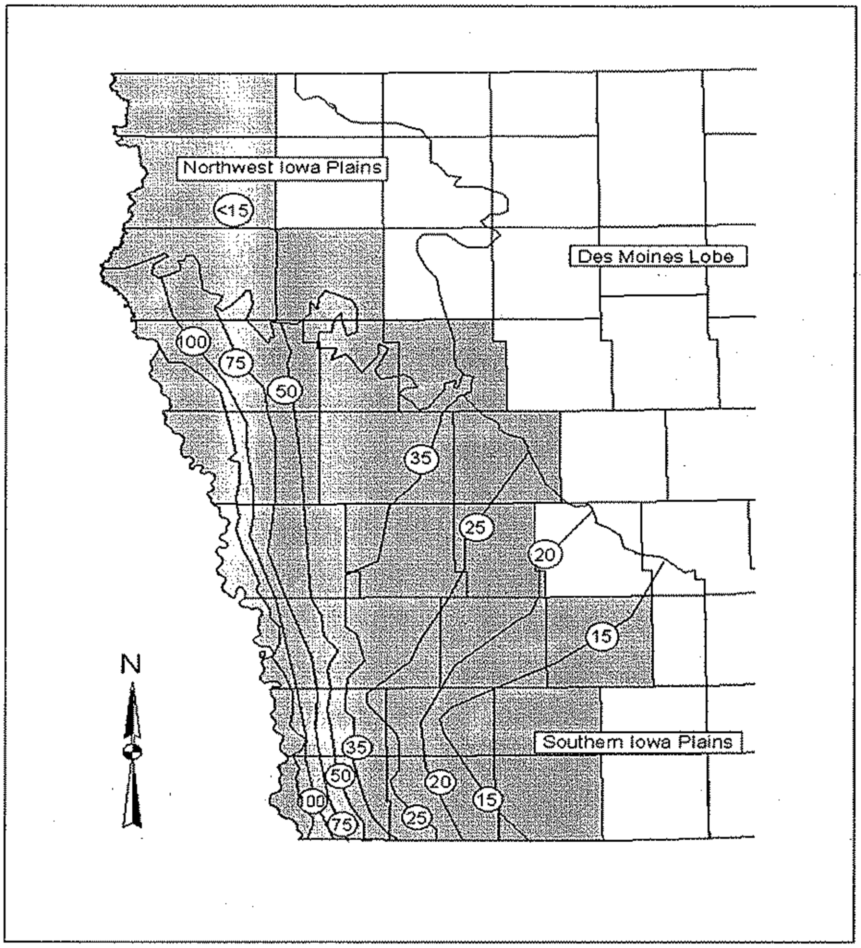
The 22-county western Iowa study area shown in Figure 1 is mantled with a thick deposit of Wisconsin-age loess, or wind-blown silt, believed to have originated from the Missouri River flood plain. Deposition of the loess occurred roughly 30,000 to 14,000 years ago. Figure 2 shows the loess depth distribution, which ranges from well over 100 feet near the Missouri River flood plain to 15 feet along the east edge of the study area.

Geologic Description

Loess is composed of silt and clay-sized particles and is highly susceptible to water erosion. The deep loess of western Iowa typically has in-place dry densities ranging from 69 lb/ft^3 to 84 lb/ft^3. Loess can maintain steep cliffs due to its low density and moderate shear strength, but when saturated collapses under its own weight. Collapse of the loess is likely caused by the loss of capillary adherence forces and expansion of the montmorillonite clay fraction of the soil as the moisture content increases. The high erodibility of loess is evident in the gullied appearance of the bluffs near the Missouri River flood plain and in the deep gullies of the upstream tributaries.

The alluvium in the streams is derived from loess and tends to have a slightly higher clay content and plasticity than the upland soils. Depth of alluvium would be expected to be greater in the lower reaches of a stream, but Antosch and Joens (1979) reported an irregular pattern of thickening and thinning from the mouths to the headwaters of streams in the study area. They also report the depth below the alluvium to the underlying glacial till is greater than 15 feet in the streams studied. This fact is important when considering the effect of a more resistant material on the degradation process.

The alluvial soils have been classified as the DeForest Formation and are believed to represent cut and fill deposits dating from the end of the Wisconsin glaciation period through the end of the nineteenth century.



Deep Loess Deposits of Western Iowa
Figure 2

Description of Study Area Drainage Basins

The primary drainages located within the 22-county study area are the Big Sioux River, Perry Creek, Floyd River, Monona-Harrison Drain, Little Sioux River, Boyer River, Nishnabotna River, and Nodaway River. Smaller drainages located within the study area drain into the Missouri River along the west and the Raccoon River along the east. Descriptions of the primary study area drainage basins follow. Table 1, which follows the basin descriptions, provides additional summary statistics for each basin.

Big Sioux River Basin. The Big Sioux River basin drains an extensive portion of South Dakota and northwest Iowa. The basin drains approximately 8,424 square miles at Akron, Iowa. Major western Iowa tributaries to the Big Sioux River are Rock River, Sixmile Creek, Indian Creek, and Broken Kettle Creek.

Perry Creek Basin. Perry Creek is located in Plymouth and Woodbury counties. The drainage area at Sioux City is 65 square miles. West Branch Creek is the only major tributary to Perry Creek.

Floyd River Basin. This basin covers portions of five counties in northwest Iowa. A gaging station near James, Iowa, which is located aproximately 9 miles from the mouth, indicates that 886 square miles are drained. Major tributaries contributing to this basin are West Branch, Deep Creek, Willow Creek, and Mink Creek.

Monona-Harrison Drain. This drainage covers portions of four counties in west-central Iowa. A gaging station located near Turin, Iowa, indicates the basin drains 900 square miles. Major tributaries to the Monona-Harrison Drain are Big Whiskey Creek, Elliot Creek, Wolf Creek, West Fork Little Sioux River, and Garretson Ditch.

Little Sioux River Basin. The Little Sioux River basin drains approximately 4,426 square miles at Turin, Iowa, which is located approximately 15 miles upstream from the mouth. The basin is located in west-central Iowa. Major tributaries to the Little Sioux River include Mill Creek, Grey Creek, Willow Creek, Maple Creek, and Rock Creek.

Boyer River Basin. This basin covers parts of nine counties in west-central Iowa. At Logan, Iowa, which is approximately 10.5 miles upstream from the mouth, the gaging station indicates a drainage area of 871 square miles. Major tributaries to the Boyer River are East Boyer River, Willow Creek, Mill Creek, Picayune Creek, Paradise Creek, and Otter Creek.

Nishnabotna River Basin. The Nishnabotna River basin provides an estimated 2,806 square miles of drainage at Hamburg, Iowa. The river is split between the East and West Nishnabotna Rivers, which cover parts of 10 southwest Iowa counties. Major tributaries located within this basin include Silver Creek, Long Branch, Indian Creek, Graybill Creek, Walnut Creek, Troublesome Creek, and Turkey Creek.

Nodaway River Basin. This basin covers parts of six counties in southwest Iowa. A gaging station at Clarinda, Iowa, reports a drainage area of 762 square miles. Major tributaries to the Nodaway River are Sevenmile Creek, Ninemile Creek, Kemp Creek, Shanghai Creek, and East and West Forks Nodaway River.

Table 1
Summary Statistics for Drainage Basins

Basin	Drainage Area (square miles)	Mean Annual Discharge[1] (cfs)[2]	Exceeded Discharge[3] (cfs) 10%	50%	90%
Big Sioux River	8,424	1,126	2,560	332	68
Perry Creek	65	38.6	29	5.1	0.8
Floyd River	886	958	502	68	11
Monona-Harrison Drain	900	497	1,160	141	39
Little Sioux River	4,426	1,067	2,680	383	0.4
Boyer River	871	349	757	155	30
Nishnabotna River	2,806	1,199	2,700	550	113
Nodaway River	762	378	810	100	19

[1] Arithmetic mean of the daily mean discharges for the period of record.
[2] cfs = cubic feet per second
[3] The discharge which is exceeded 10%, 50%, or 90% of the time during the period of record.

Stream Classification

The U.S. Geological Survey (USGS) developed a system of classifying streams according to channel evolution for dominant channel processes that could cause streambed degradation and streambank widening. Six stages of classification were established within this system to identify premodified channels, constructed or modified channels, degrading channels, streambank widening, aggrading channels, and restabilized channels.

The Golden Hills RC&D conducted an aerial reconnaissance of western Iowa streams during 1993 and 1994 to provide a regional assessment of existing stream conditions with regard to channel and streambed erosion. A complete description of the aerial reconnaissance is presented in the final report entitled "Stream Stabilization in Western Iowa," by Golden Hills RC&D, Oakland, Iowa, December 1994.

Description of Channel Evolution Stages

A description of the six stages of channel evolution is presented below, and a graphical depiction of each stage is shown in Figure 3.

Stage 1. Stage 1 channels tend to be very stable, having a dense vegetative covering along the side banks down to the low-flow channel. Bank failure does not generally occur during this stage.

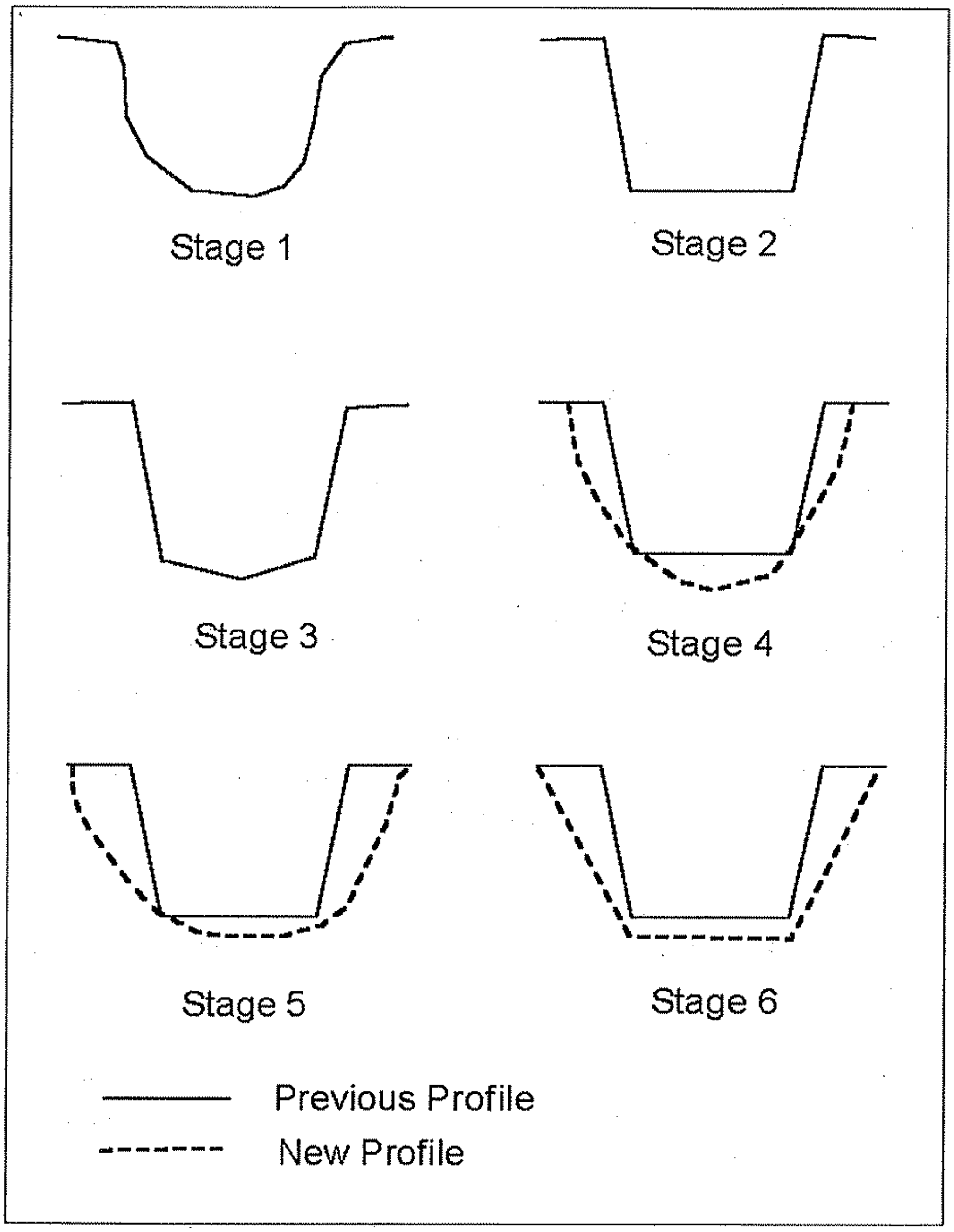
Stage 2. This category is associated with channels that have been recently modified by construction. If the modification results in a trapezoidal-shaped channel, the banks tend to remain stable. In some instances, degradation or aggradation may take place, depending on characteristics such as channel slope, cross-sectional area, and angle of sideslopes.

Stage 3. Due to downstream increases in channel slope and streamflow velocities, bank heights increase and sideslopes become steeper because of lowering of the channel invert. The channel invert and toe of sideslopes are undercut, and streambank failures occur. Full-scale degradation takes place during this stage, while the streambanks remain fairly stable.

Stage 4. Streambed degradation occurs at a lesser rate, while channel widening is predominant during this stage. The channel widens by mass wasting of the streambank material, causing a scalloped appearance.

Stage 5. During this stage, aggradation occurs along the streambed, which begins to reduce the height and sideslope angle to the top of bank. As the overall height is reduced, bank failure decreases and revegetation increases.

Stage 6. Mass failure of streambanks is greatly diminished, and revegetation extends in a dense cover up the sideslopes. The channel invert becomes increasingly stable, and bank widening is eliminated.



Stream Classification
Figure 3

The final results of the Golden Hills RC&D classification analysis indicated that of the 990 miles of streams evaluated, approximately 10.3 percent of the total streams appeared to fall within the Stage 3 classification. Approximately 56 percent of the streams were classified as Stage 4, while 25.6 percent were classified as Stage 5 streams. Table 2 provides the classification breakdown on all streams classified by the Golden Hills RC&D during 1994.

Table 2
Results of Golden Hills RC&D 1994 Stream Classification

USGS Classification	Miles	Percent of Total
Stage 1	42.2	4.3
Stage 2	0.8	0.1
Stage 3	101.8	10.3
Stage 4	553.0	55.9
Stage 5	253.9	25.6
Stage 6	37.4	3.8

From Table 2, it appears that funding to combat degradation in streams classified as Stage 3, Stage 4, or Stage 5 would provide the most benefit. Funding for streams having other classifications would not provide substantial benefits during the initial stages of degradation; the streams would probably require significant funding if the channel width was excessive, as is typical of channels in the later stages of degradation. Prevention of future degradation during Stages 3 and 4 could save thousands of acres of farmland and millions of dollars in future infrastructure costs.

Primary and Secondary Roadway Bridge Assessment

Previous studies conducted by the Golden Hills RC&D and ISU have indicated that a substantial amount of streambed degradation and streambank widening has been occurring throughout western Iowa. Numerous State and county bridges have been severely damaged by degrading streams, and many more structures will be affected in the future. The following discussion focuses on State and county bridges within the 22-county study area that may be susceptible to damages resulting from streambed degradation and streambank widening.

Bridges Susceptible to Damages from Degrading Streams

Streambed degradation and streambank widening affect bridges by damaging piers and abutments. Bridges may be permanently closed to (or limit) heavy load traffic or be required to have modifications to existing spans. Sometimes bridges may be so severely damaged that they may be required to be replaced.

Structure Inventory and Appraisal

Bridge information for all 22 counties was obtained from the (IDOT). The information was provided as a computer database in which all codes within the "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges" were contained. This guide was developed by the Federal Highway Administration (FHWA) as a tool to be used by each State to collect bridge data to be used to develop Defense Bridge and Federal Emergency Management (FEMA) reports.

Each State and each county conduct detailed bridge inspections once every 2 years by trained maintenance personnel who provide the rating for each coding contained on the structure evaluation form. The guide contains 116 fields to provide information ranging from bridge location to the minimum navigational vertical clearance for each bridge site.

Codes Used for Determining Bridges Susceptible to Damages from Degradation

Key fields which appear to focus on channel instability were queried within the structural inventory to determine which bridges may be susceptible to damages resulting from streambed degradation and streambank widening. Several fields were queried so that bridges with deficiencies in substructure elements alone would not be selected. Each of the fields queried contained a numerical rating provided by the bridge inspector to identify the current status of the bridge for that specific item. In some instances the scour critical bridges field was not queried because no bridges were currently identified within that field. The three fields queried are described below

Substructure. This field refers to the physical condition of piers, abutments, piles, fenders, footings, or other components. All substructure elements should be inspected for visible signs of distress including evidence of cracking, settlement, misalignment, scour, and corrosion.

The ratings queried on this field: 0 Failed Condition. Out of service; beyond corrective action.
1 Imminent Failure. Major deterioration or section loss.
2 Critical Condition. Scour may have removed support.
3 Serious Condition. Loss of section and deterioration.
4 Poor Condition. Advanced section loss, and deterioration.

Channel and Channel Protection. This field refers to the physical condition associated with the flow of water through the bridge; e.g., the stream stability and the condition of the channel, riprap, slope protection, or stream control devices including spur dikes. Inspectors should be particularly concerned with visible signs of undermining of slope protection or footings, erosion of streambanks, and realignment of the stream which may result in immediate or potential problems.

The ratings queried on this field: 0 Bridge out of service due to channel failure.
1 Bridge temporarily closed due to channel failure.
2 Bridge near state of collapse due to waterway change.
3 Bridge affected by aggradation or degradation.
4 Streambank protection is severely undermined.
5 Streambank protection is being eroded.
6 Streambank is slumping; streambed movement is evident.
7 Streambank protection requires minor repairs.

Scour Critical Bridges. The scour critical bridge field identifies the current status of the bridge regarding its vulnerability to scour. Scour evaluations should be made by hydraulic/foundation engineers. A scour critical bridge is one with abutment or pier foundations that are rated as unstable due to observed scour at the bridge site or a scour potential as determined from a scour evaluation study.

The ratings queried on this field: 0 Bridge has failed and is closed to traffic.
1 Failure of piers/abutments is imminent.
2 Extensive scour has occurred at bridge foundations.
3 Bridge foundations are determined to be unstable.

After identifying which fields and numerical values to review in the structure inventory database, each of the bridges contained in the 22 counties was individually queried to determine the number of and location of bridges that are or may be susceptible to damage from streambed degradation and streambank widening. Table 3 presents the number of bridges, by county, that are susceptible to damage, and Plates 1 through 22 identify the location of each of these susceptible bridges as well as all other functioning bridges for each of the 22 counties in the study area.

State and county officials may use the data presented in Table 3 and Plates 1 through 22 to plan preventive maintenance or traffic routes prior to the closing of additional bridges. This information may also be compared to the stream classification maps developed by Golden Hills RC&D inits December 1994 report entitled "Stream Stabilization in Western Iowa," to determine whether the bridges identified in the structure inventory database coincide with the streams identified in the Golden Hills RC&D report as having active streambed degradation or streambank widening. Since the data used for querying each bridge site were provided during 1996, there may be additional bridges that are currently showing signs of damage caused by streambed degradation or streambank widening that are not presented in this data set.

Table 3
Bridges Susceptible to Damage from Streambed Degradation and Streambank Widening

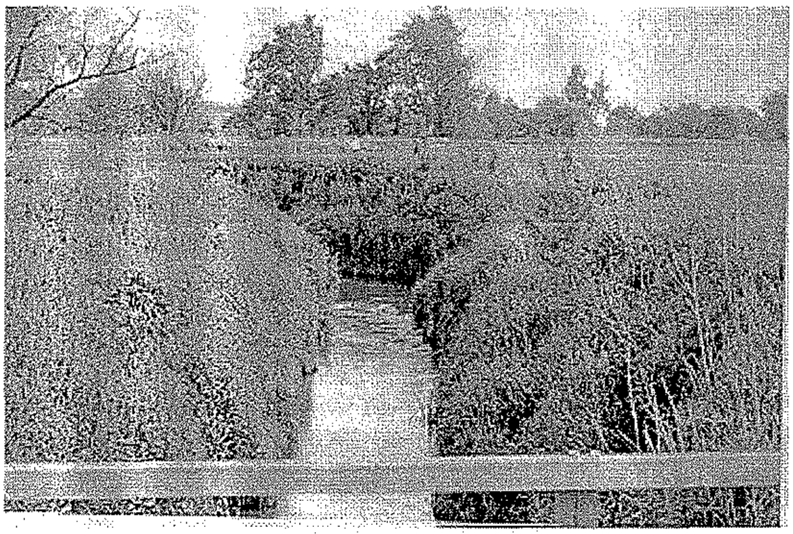
County	Total Number of State and County Bridges	Total Number of State and County Bridges Susceptible to Damages from Degradation
Adair	318	11
Adams	212	7
Audubon	176	5
Carroll	296	7
Cass	289	5
Cherokee	227	12
Crawford	345	12
Fremont	171	6
Harrison	202	6
Ida	192	22
Lyon	252	6
Mills	189	19
Monona	165	7
Montgomery	213	7
Page	239	29
Plymouth	531	108
Pottawattamie	472	17
Sac	236	15
Shelby	275	48
Sioux	392	1
Taylor	264	39
Woodbury	403	59

Chapter 3
General Discussion and Design Techniques

Streambed Degradation and Streambank Widening

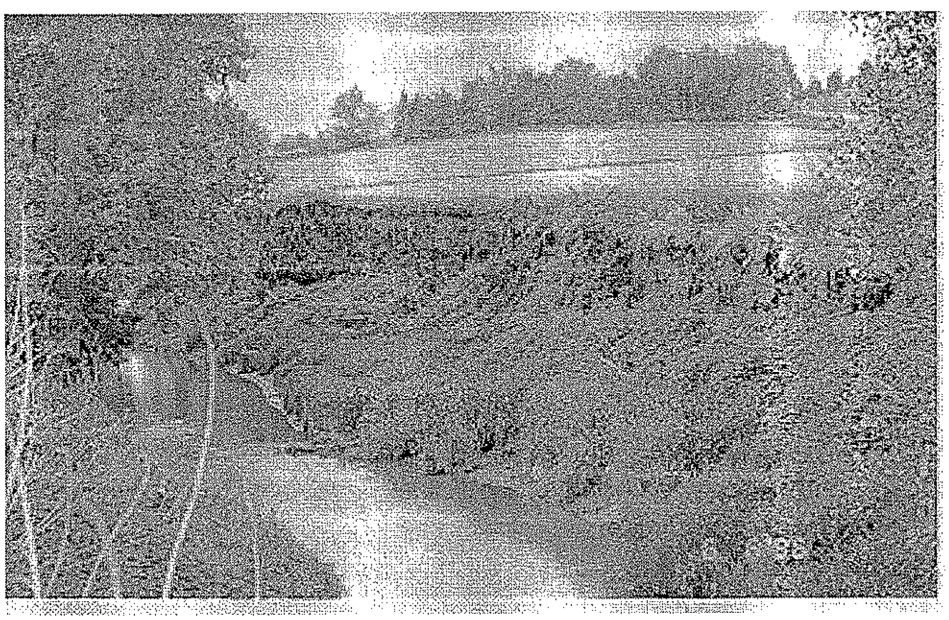
Grade control structures have been successfully used in the deep loess region of western Iowa to contain the migration of streambed degradation. Beginning in the 1880's, rivers and streams throughout the 22-county study area were straightened by individual landowners and drainage districts. The channel straightening was primarily undertaken to promote quicker drainage of existing farmland, to assist in establishing a transportation system that minimized the number of bridge crossings for any given stream, and to generate additional farmland which normally may have been unfarmable due to meanders, wetlands, and inaccessibility.

As depicted in Figures 4 through 6, the straightened streams began to degrade through the development of nick points and head cuts. As the streambed became more incised, the streambanks became overly steep, to a point where the natural angle of repose would be exceeded, causing widespread streambank failure. This process repeated itselfover and over until a deep canyon emerged from the devastated landscape.



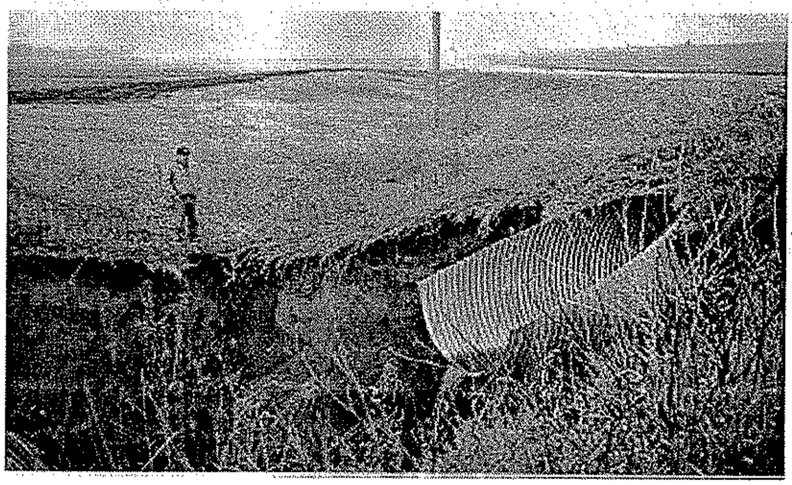
Stream Prior to Degradation
Figure 4

Typical Headcut
Figure 5

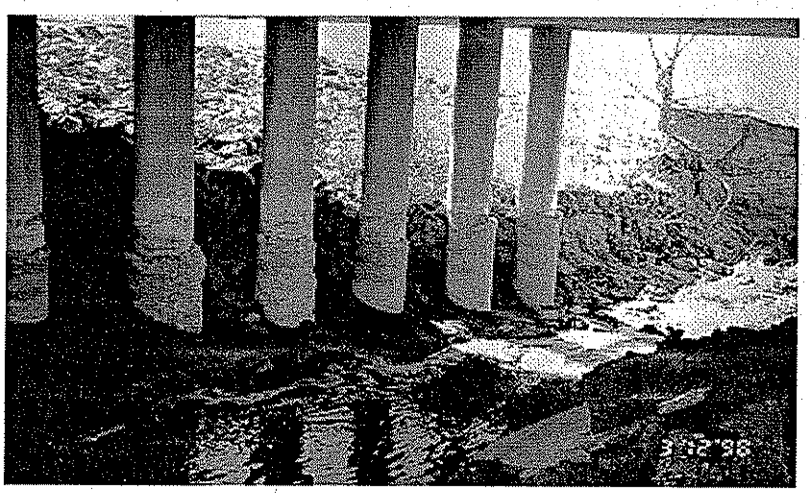


Result of Streambed Degradation and Streambank Widening
Figure 6

The action of streambed degradation and streambank widening has had a devastating effect on infrastructure and land erosion in western Iowa. In a 1994 report entitled "Estimates of Future Impacts of Degrading Streams in the Deep Loess Soil Region of Western Iowa on Private and Public Infrastructure costs" by Professor Baumel of the Department of Economics, ISU, the estimated future streambed degradation and streambank widening costs for western Iowa will be $177.3 million (1992 dollars). Based on the estimation of time value of money, future degradation costs associated with damages to infrastructure and land erosion could reach $1.1 billion. Figures 7 through 10 show the impact of streambed degradation and streambank widening on infrastructure.



Drainage Culvert Damages Due to Streambed Degradation
Figure 7



Bridge Pier Damage Due to Streambed Degradation
Figure 8

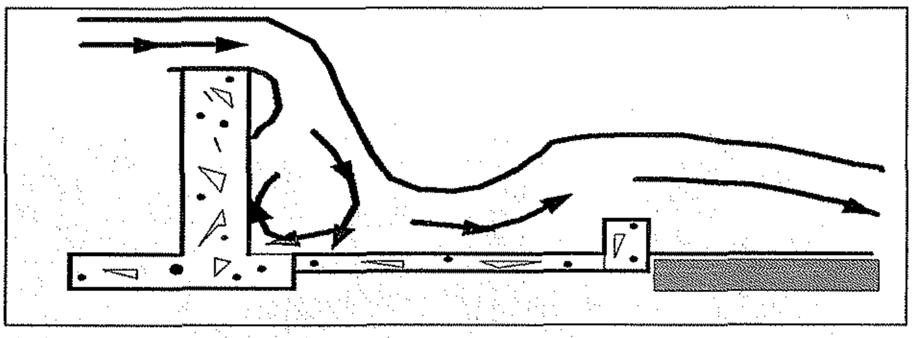
Bridge and Drainage Structure Damage
Figure 9

Bridge Damage Due to Streambed Degradation
Figure 10

Grade Stabilization Structures

Grade stabilization structures can either be implemented to stabilize streambeds, thereby reducing the amount of valuable farmland eroded annually, or be placed downstream from a bridge or utility line to protect the infrastructure from collapse. The 2-year peak flood discharge is typically chosen for the design discharge since it closely resembles the bankfull discharge for most streams. The 2-year flood discharge also forms velocities that may induce streambed degradation when flowing through a steep reach.

Although the four grade stabilization structures presented in this manual contain differing features, many of the features function similarly. The vertical drop structure shown in Figure 11 has structural features that generally are typical of all grade stabilization structures; these features are discussed below.



Typical Grade Stabilization Structure
Figure 11

Structure Features

Weir. Each grade stabilization structure requires a weir that provides the backwater effect on flows in the river or stream. Weirs are typically constructed of riprap, sheetpile, or concrete. As will be discussed in Chapter 4, weirs may also be constructed of logs, natural rock, and debris. The height of the weir determines the amount of grade stabilization that may be obtained from a structure. If the weir height is too great, flows will rise above either streambank, flooding property adjacent to the stream. If the height of the weir is too low, the structure will be ineffective in stabilizing the streambed, and degradation will continue.

Hydraulic Jumps. The fall of water over a weir, as presented in Figure 11, will typically result in a hydraulic jump as the flow transitions from subcritical at the weir to supercritical below the weir, and back to subcritical farther downstream from the weir as the energy of the flow is dissipated. The supercritical flow is an unstable flow usually associated with high velocities and scouring. If supercritical flow is allowed to continue downstream, it may induce additional erosion of the streambed and streambanks or perhaps cause the grade stabilization structure to fail.

Stilling Basin. A stilling basin is a short length of concrete, grouted riprap, concrete blocks, or large derrick stone where the hydraulic jump occurs. Several types of stilling basins are typically used in the design of stabilization structures. Two of the most common stilling basins are mentioned below.
(1) Saint Anthony Falls Basin. This basin was developed at the Saint Anthony Falls Hydraulic Laboratory, University of Minnesota, for use with small drainage structures, small spillways, and outlet works. Basins of this type usually incorporate a concrete apron, chute blocks, an end sill, and a cutoff wall in the design.
(2) U.S. Bureau of Reclamation (BOR) Basins. The BOR has developed generalized designs for several different types of stilling basins. The basins were developed for controlling jumps on both flat aprons and sloping aprons.

Tailwater. The length of the hydraulic jump depends on the downstream tailwater elevation. If the tailwater fluctuates, the length of jump will also fluctuate. It should be noted that the tailwater depth must be as deep or deeper than the jump depth in the stilling basin or the jump will try to occur farther downstream, which would result in additional erosion to the streambanks.

End Sill. To maintain a somewhat constant length of jump, end sills can be constructed into the lower end of the stilling basin. The end sills will artificially increase the tailwater within the stilling basin and ensure that energy dissipation does not take place farther downstream where the channel and sideslopes may be unprotected.

Energy Dissipating Blocks. When the vertical drop is large and the downstream channel slope is steep, channel blocks, chute blocks, or baffle piers may be required to dissipate the energy developed through the drop. The blocks are usually constructed of concrete, sheetpile, or large derrick stone. The blocks disrupt the flow occurring after the hydraulic jump, preventing erosion of the streambed and streambank from occurring farther downstream.

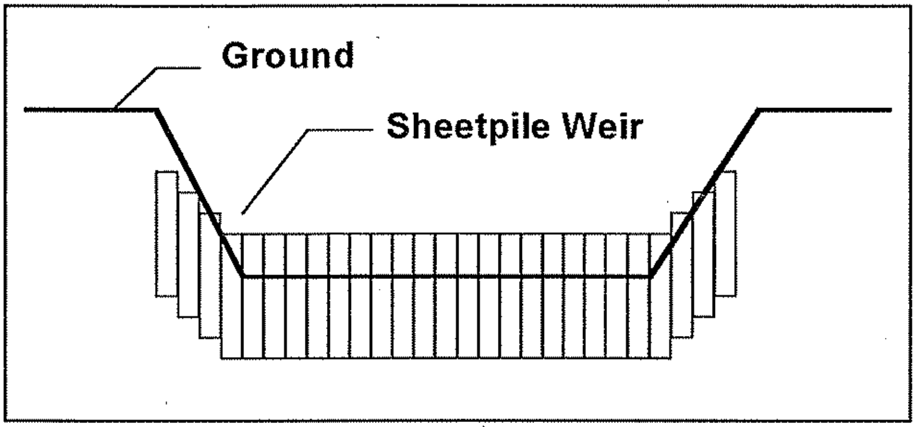
Riprap Slope Protection. Riprap can be placed in layers for slope protection. The riprap maximum, minimum, and 50-percent sizes are determined by hydraulic analyses.

Bedding Layers. The gradation of the bedding material should provide for the retention of bedding particles by the overlying riprap and for the retention of the material underlying the bedding layer. The application of bedding layers will sometimes require the inclusion of an intermediate filter layer between the bedding and riprap to ensure the bedding material remains in place.

Structure Types

This design manual focuses on four specific grade control structure types: sheetpile, H-pile, rock sill, and concrete block. Reinforced box culverts were initially evaluated early in the studybut were eliminated from evaluation after it was determined that a significant amount of design information already existed regarding this type of structure. The four structures evaluated during this study are discussed below.

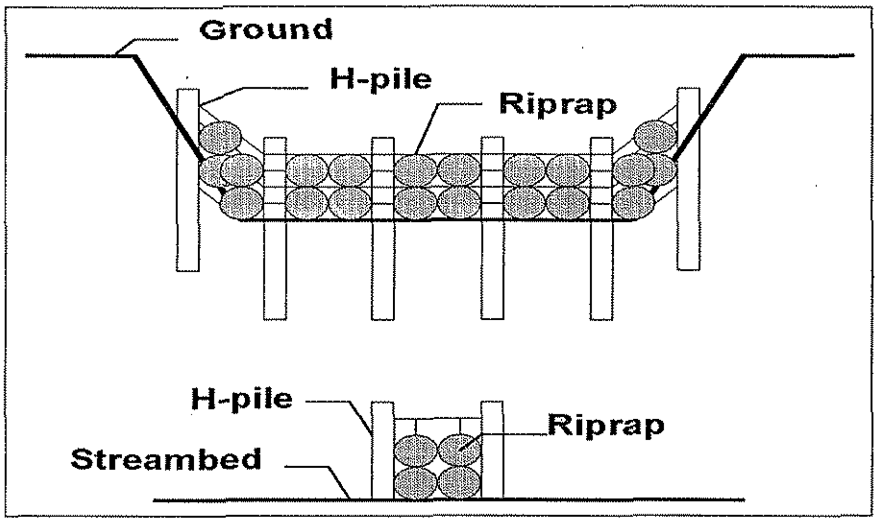
Sheetpile Structure. The sheetpile structure is a common grade stabilization structure typically used within deep loess regions of the United States. The sheetpile is interlocked, providing a good barrier to downstream flows. As riverflows are slowed upstream fromthe sheetpile, sediment begins to drop out of the water, thereby increasing the elevation of the streambed. A simplistic illustration depicting a sheetpile grade stabilization structure is shown in Figure 12. The structure uses the sheetpile to form a weir across the channel bottom width. The sheetpile is keyed into the streambanks to prevent flanking of the structure and possible failure during flood events. Riprap may be required along the streambanks and below the structure to prevent streambank erosion and undermining of the sheetpile. The height of the sheetpile is dependent upon the amount of grade being stabilized, streambank elevations, possible fish migration, ande economic costs.



Sheetpile Grade Stabilization Structure
Figure 12

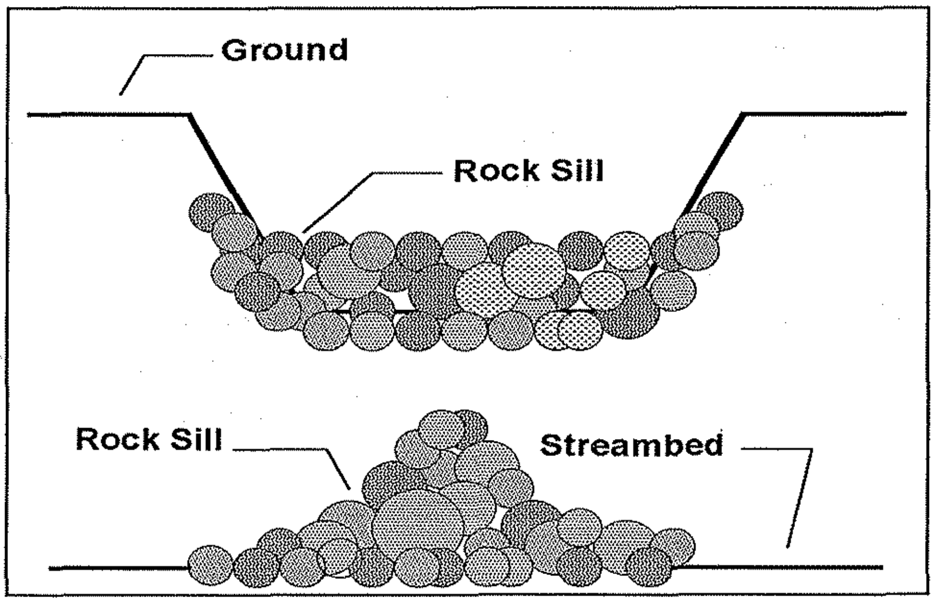
H-pile Structure. For this type of grade stabilization structure, H-piles form the skeletal frame from which steel-stranded cable or hog panels are fastened. Riprap is placed within the crib formed by the cable or hog panels, creating a weir from which a backwater effect is developed. Figure 13 depicts the general features of the H-pile structure.

Weir height is restricted to the strength of material used to form the crib. Occasionally, a filtration fabric is attached to the crib to capture sediment normally lost through the void spaces associated with riprap. As with the sheetpile structure, riprap may also be required along the streambanks and downstream from the structure to avoid erosion and possible failure of the weir.



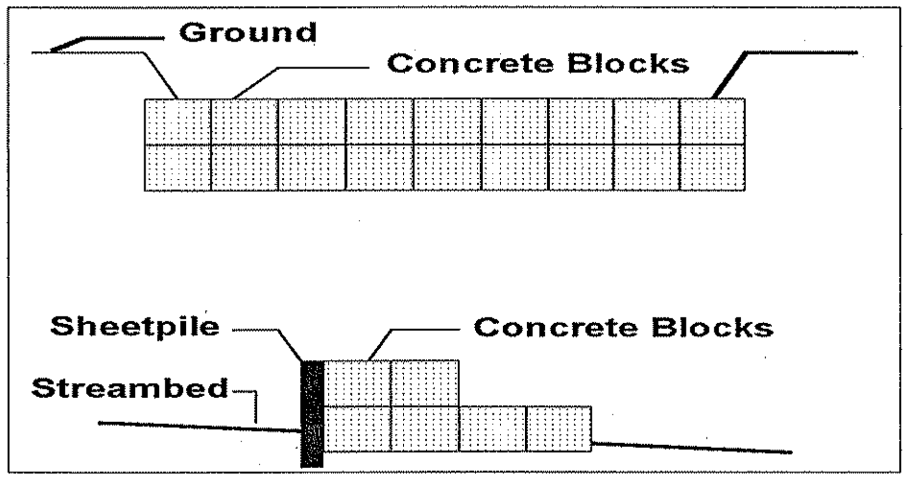
H-pile Grade Stabilization Structure
Figure 13

<u>**Rock Sill Structure.**</u> The rock sill shown in Figure 14 provides adequate grade stabilization, has low construction costs, and is fairly easy to implement. Drawbacks to rock sills include moderate to high maintenance costs to maintain the riprap, and the fact that the sill that is occasionally displaced by high discharges, easily affected by ice and debris, and limited in overall height.



Rock Sill Grade Stabilization Structure
Figure 14

Concrete Block. Concrete block structures have been implemented recently in several locations in western Iowa for grade stabilization where large riprap, derrick stone size, is difficult to obtain. These structures have been used in streams where velocities could easily displace small riprap. The concrete blocks are usually cubic in shape, with typical dimensions of 3 feet x 3 feet x 3 feet. As shown in Figure 15, typical concrete block structures incorporate a sheetpile weir upstream from the blocks. The blocks may be stacked on top of one another to establish a large weir height. The blocks may also be tied together to make the structure more resistant to high-velocity flows.



Concrete Block Grade Stabilization Structure
Figure 15

Typical Grade Stabilization Unit Costs

Various types of grade stabilization projects have been implemented in each of the 22 counties located in the study area. Although many of these projects resemble one another, most of the projects have not been standardized. Many of the structures are subjected to varying levels of design criteria from one county to another. The availability of materials such as riprap, sheetpile, H-pile, engineering fabric, gabions, and concrete often dictate the final costs associated with construction of a project. County engineers lack standardized unit cost tables to compare the cost-effectiveness of one project to another.

The location and availability of quarry-run riprap have a significant effect on the cost of certain grade stabilization structures throughout the study area. As quarries use up their reserve of derrick stone, other options such as reinforced concrete blocks may become more feasible to use for construction. The price of construction material could be impacted by the change in the number of suppliers within a region. Factors such as these will slightly affect the overall price structure for materials, but for a comparison basis, a database of typical unit costs will assist the design engineer in making cost-effective choices in selecting the most feasible structure type.

Unit Costs Database

The COE has developed a database referred to as the Micro Computer Aided Cost Estimating System (MCACES) for maintaining standardized unit costs for materials within different regions of the United States. These regional databases provide engineers a tool to develop construction cost comparisons between different types of structures; this helps the engineers determine the most economically feasible project. Table 4 provides the unit costs associated with construction materials typical for grade stabilization projects in the midwestern United States.

Unit Costs Database Updates

The unit costs database was developed using December 1997 construction costs. These costs may change over a period of time. The costs should be adequate for comparing various structures to one another to determine the most cost-effective structure to construct. One method of updating the database would be to use the Engineering News Record (ENR) cost indexes for construction economics. The ENR cost indexes are updated monthly and are available as a magazine. They can also be found on the internet at the following address: *www.enr.com.*

The ENR cost indexes provide regional updates for construction, building, labor, and material costs. The indexes can be used to update the unit costs database presented in Table 4 by multiplying the unit cost by the ENR regional index.

Unit Cost Assumptions

Several assumptions were necessary for determining the unit costs for construction material. The assumptions were developed according to similar construction projects implemented within the Midwest. A description of each of the major assumptions used in developing the MCACES unit costs is provided in Table 5.

Table 4
Typical Grade Stabilization Unit Costs

Construction Material	Measurable Unit	Cost ($) Low - High
Riprap	Ton	20-40
Bedding	Ton	15-35
Sheetpile	Square Foot	15-23
H-Pile	Pound	0.30-2.00
Channel Excavation		
backhoe	Cubic Yard	1.50-2.50
dragline	Cubic Yard	2.50-5.00
Earth Hauling	Cubic Yard	0.50-5.00
Gabions	Cubic Yard	100-115
Concrete Grout	Cubic Yard	100-300
Engineering Fabric		
geotextile	Square Yard	1.50-3.00
geonet	Square Yard	1.60-2.50
Hog Panels	Square Foot	2.00-3.00
Clearing and Grubbing	Acre	250-10,000
Seeding and Mulching		
field seeding	Acre	500-1,000
hydroseeding	Acre	1,250-2,500
erosion control netting	Square Yard	0.75-1.25
Cost Contingencies		
low risk	% of total	10-15
normal risk	% of total	20-25
high risk	% of total	30-35

Table 5
Assumptions Used in Unit Costs Development

Item	Unit Cost Assumption
Riprap	Plant Cost at $8/ton Placement at $8/ton Haul Distance from Plant to Site is 50 Miles at $0.12/ton/mile
Bedding	Plant Cost is $3-8/ton Placement is $3-8/ton Haul Distance and Cost Similar to Riprap
Sheetpile	Depths of 15-25 Feet Rating of 2-38 psf Costs Include 20% Overhead/Profit
H-pile	Costs Include Labor/Equipment/Material Installed Costs Include 20% Overhead/Profit
Channel Excavation	Costs Without Material Hauling
Earth Hauling	Use of 12-cubic-yard Trucks 50 Minute Hours Average Travel Speed of 22 mph Costs Include 20% Overhead/Profit
Gabions	Costs Do Not Include Excavation Costs Include Galvanized Baskets Costs Include In-place Stone Costs Include 20% Overhead/Profit
Concrete Grout	Costs Include Labor/Equipment/Long Haul Distance Costs Include Extreme Case Concrete Pumps Costs Include 20% Overhead/Profit
Hog Panels	Costs Include all Labor/Equipment for Installation Costs Include Galvanized Panels, Cable, and Clamps Costs Include 20% Overhead/Profit
Clearing and Grubbing	Costs Include all Labor/Equipment Costs Include 20% Overhead/Profit
Seeding and Mulching	Costs Include Labor/Equipment/Material Installed Costs Include 20% Overhead/Profit

Chapter 4
Nonstructural Measures and Techniques

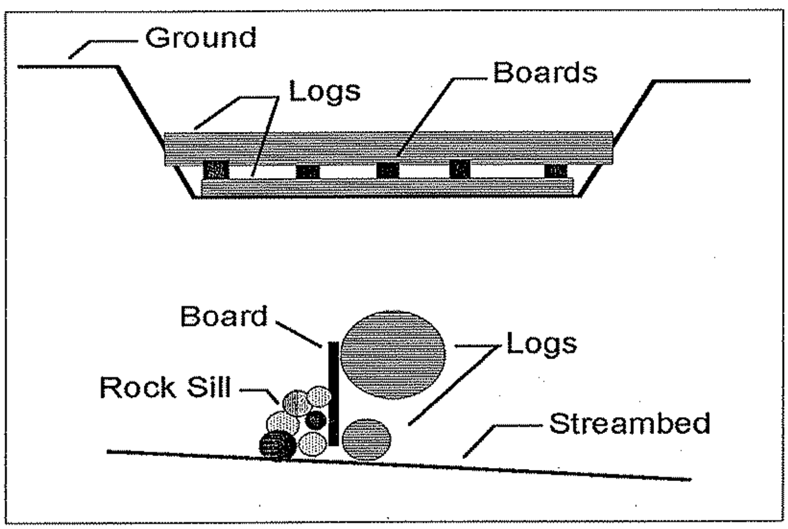
Soft Structures

Soft structures are not comprised of the kinds of construction materials commonly associated with grade control structures. In place of concrete basins, sheetpile weirs, and graded riprap, logs, boards, and channel debris are used. The materials used for the construction of soft structures are usually available in the area of construction. Soft structures should not be implemented if the existing vertical drop is in excess of 2 feet. As with the hard structures, care must be taken to provide proper energy dissipation downstream from the structures. Soft structures can be constructed at a low cost, but maintenance and repair costs may be substantially higher than for hard structures.

The U.S. Fish and Wildlife Service, the U.S. Forest Service, and several State Fish and Game Departments have successfully implemented soft structures on small streams and drainageways. Descriptions of several soft structures follow.

Board Dam

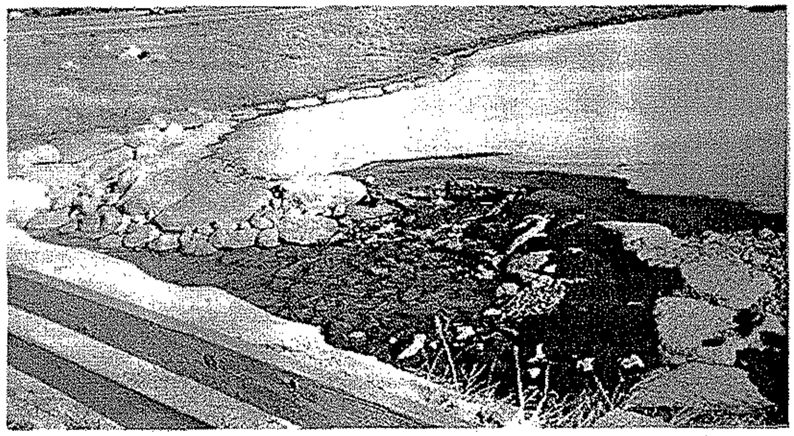
This structure, shown in Figure 16, consists of a single log that extends from bank to bank, slightly higher than the channel invert. A smaller diameter log is placed at the channel invert and attached to the main damming log by boards nailed to the logs. A gravel and rock sill is placed along the upstream side of the logs.



Board Dam
Figure 16

Natural Rock Dam

This type of structure is normally constructed in locations where natural rocks are readily available. Large rocks should be placed in the channel with an upstream arch for stability. Graded riprap may then be placed upstream from the main rocks to provide a seal for the structure. The photograph shown in Figure 17 portrays one of a series of small rock dams constructed to stabilize the channel shown, to provide an aesthetically pleasing view to pedestrian traffic, and to provide fish migration through the ripples and pools created by the rocks.



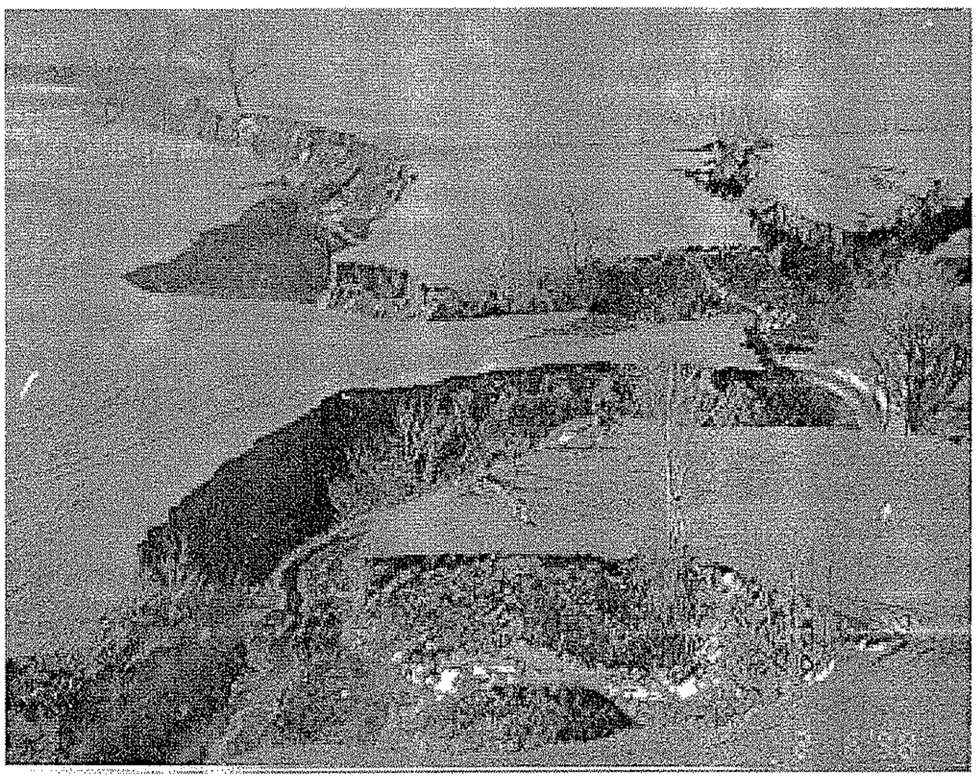
Natural Rock Dam
Figure 17

Beaver Dam

Although not constructed by man, beaver dams are very efficient at maintaining a stable streambed while allowing large flows to move downstream without causing considerable damage. If located in an area of natural vegetation, beavers will not cause extensive damage to agricultural crops. A typical beaver dam is shown in Figure 18.

Beaver Dam
Figure 18

During several low-level flights over a number of streams located in the deep loess region of western Iowa, it was observed that many of the streams appeared to remain in a stable state if strips of natural vegetation existed along either side of the channel. In many instances, the land leading up to the edge of the streambanks is used for farming. Streambanks lined with corn offer little resistance to streambank erosion. Several programs are offered through the NRCS and the Iowa Department of Natural Resources (DNR) to help provide financial and technical assistance to landowners for development of buffer strips along rivers and streams. These buffer strips reduce erosion through the establishment of a deep root network, help provide sources of construction material for beavers, filter agricultural runoff prior to reaching waterways, and prevent livestock from initiating erosion along streambanks. With sound conservation practices, many damaged river reaches may be protected and valuable farmland saved. These practices ensure that river reaches devastated by streambed degradation and streambank erosion, as shown in Figure 19, do not become widespread in western Iowa.



Typical Stream Damages Without Conservation Practices
Figure 19

Combination Hard and Soft Structures

These types of structures rely on a combination of hard and soft features to perform their function. As with the soft structures, these structures should never be used when the existing vertical drop is in excess of 2 feet. For greater drop heights, a series of structures may be incorporated. Several kinds of combination hard and soft structures are described below.

K-Dam

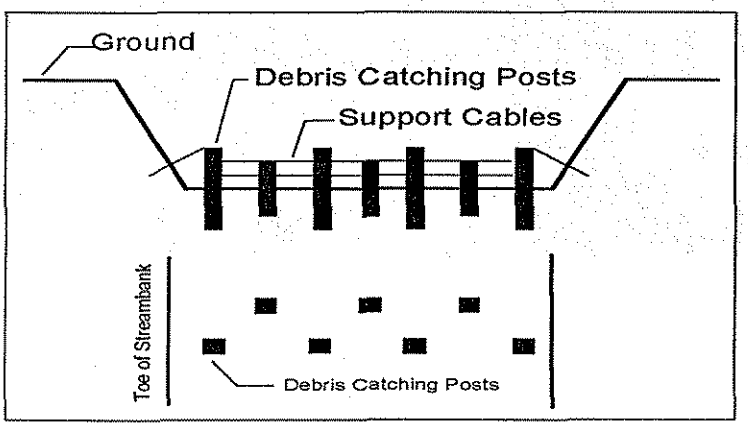
This structure, consisting of rocks and logs, is used to span a small river or stream. Logs, referred to as mudsills, provide the foundation of the K-dam. The mudsills are placed parallel to the flow, along the invert of the channel. The abutments of the dam are placed 6 feet into the streambank. A large damming log is placed above the mudsills, perpendicular to the flow of the channel. A woven wire is then attached to the mudsills, damming log, and abutments and then backfilled with riprap.

Log Dam

Where velocities and scouring are not excessive, a low-head dam may be constructed by spanning the stream with logs. The logs must tie into the channel banks for stability. Graded riprap is then placed upstream from the logs to act as a seal for the structure.

Debris-Catcher Dam

This structure is constructed by securing steel posts into the streambed and attaching woven wire to the upstream side of the posts and along the streambed upstream from the posts. Large rocks should be placed over the woven wire upstream from the steel posts for stability. Debris will tend to catch or build up along the steel posts and woven wire as shown in Figure 20, creating a small damming effect within the channel.



Debris-Catcher Dam
Figure 20

Chapter 5
Streambank Stabilization Measures and Reference Documents

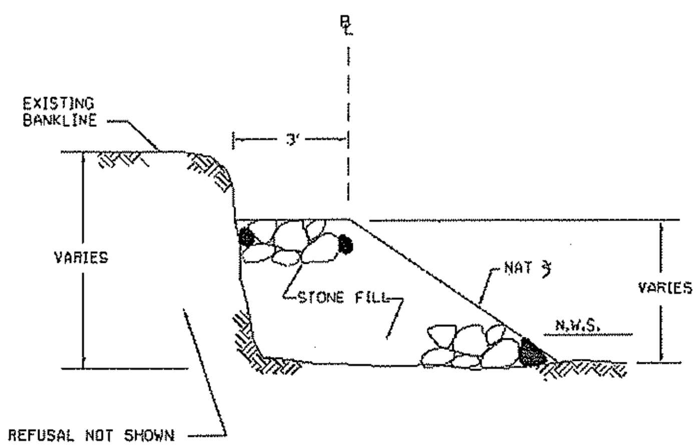
Since this design manual was developed to emphasize streambed degradation and streambank widening design criteria, and an enormous amount of information is available on streambank stabilization, no design criteria for stabilization projects is presented. Descriptions of several structural and nonstructural streambank stabilization measures are presented; these are followed by a list of references to be used as sources of design procedures for streambank stabilization projects.

Structural Measures

Many of the so-called structural streambank stabilization projects employ common engineering materials such as stone fill, riprap, concrete mattresses, and gabions to perform their intended function. Several of these structural measures are described below.

Stone Toe Section

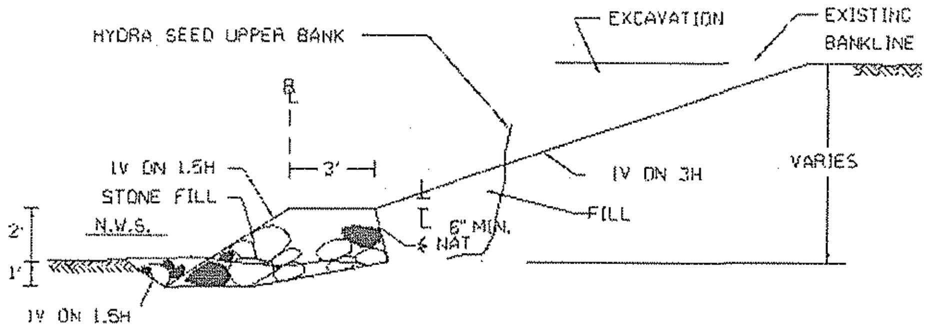
Stabilization of the streambank may be achieved by reinforcing the toe with stone fill as shown in Figure 21. The stone is placed against the toe of the streambank and allowed to slope toward the waterway at its natural angle of repose. The stone fill is placed up to a width of approximately 3 feet.



Typical Stone Toe Section
Figure 21

Stone Toe with Upper Bank Reshaping

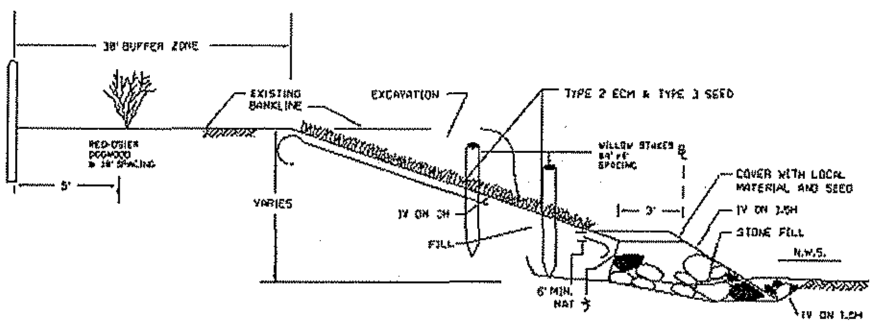
The stone toe with upper bank reshaping measure is a stone toe section that is excavated and filled to reduce the gradient of the upper bank. The upper bank is typically seeded to minimize erosion. A typical structure is shown in Figure 22.



Typical Stone Toe with Shaped Upper Bank
Figure 22

Stone Toe with Willow Stakes

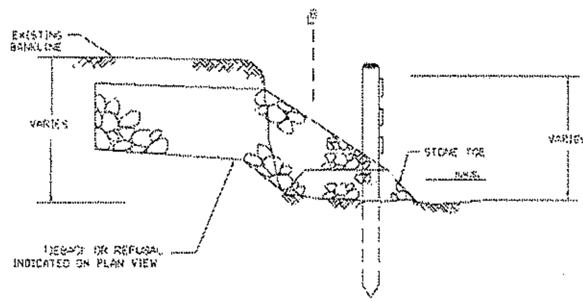
The stone toe with willow stakes measure, shown in Figure 23, is a stone toe section that is excavated and filled to reduce the gradient of the upper streambank. The stone toe section is capped with a minimum of 6 inches of soil. Willow stakes are driven into the ground near the stone toe for additional slope stability.



Typical Stone Toe with Willow Stakes
Figure 23

Single Row Fence with Stone Toe

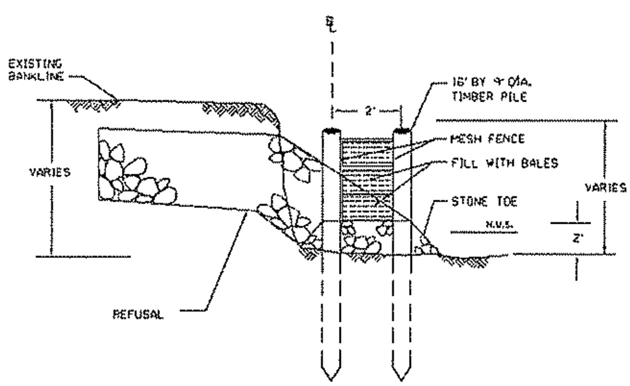
The single row wood and wire fence measure, shown in Figure 24, has a tie-back or refusal into the bank. Fences are used primarily in small, low-gradient streams. They are constructed parallel to the bankline to promote sedimentation. The toe is stabilized with stone fill material.



Single Row Fence with Stone Toe
Figure 24

Double Row Fence with Hay Bale Fill

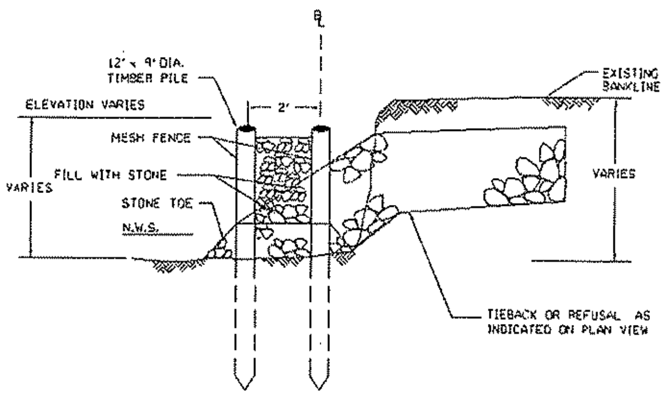
The double row wood and wire fence measure shown in Figure 25, is similar to the single row fence except that an additional row of posts is driven 2 feet from the first row. The area between the posts is wrapped with wire mesh and then filled with hay bales to promote sedimentation.



Double Row Fence with Hay Bale Fill
Figure 25

Double Row Fence with Stone Fill

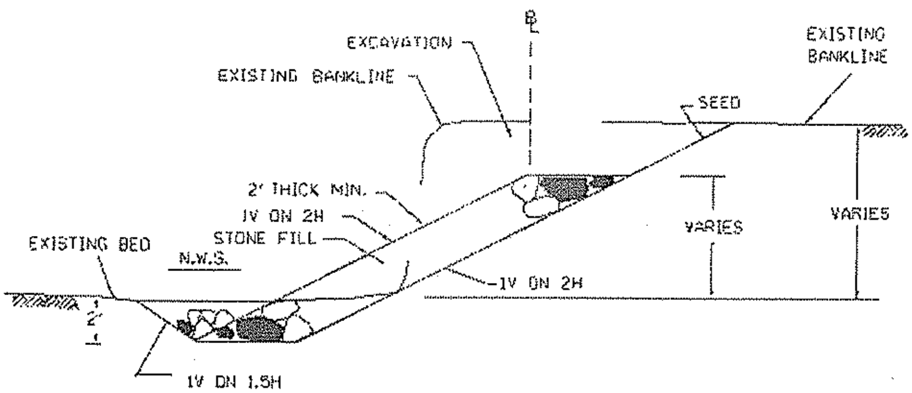
The double row wood and wire fence measure shown in Figure 26, utilizes stone fill material between the two fences to promote sedimentation. This structure is used with higher velocity channels.



Double Row Fence with Stone Fill
Figure 26

Revetment with Reshaped Streambank

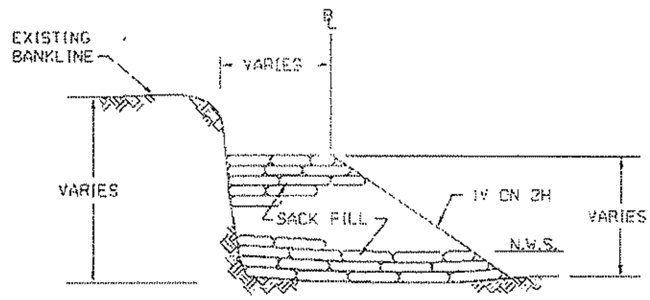
The revetment measure (commonly referred to as riprap revetment) shown in Figure 27,is constructed by decreasing the gradient of the bank and covering the bank with stone. Stone fill revetments are the most commonly used erosion control structure.



Revetment with Reshaped Streambank
Figure 27

Concrete Sack Revetment

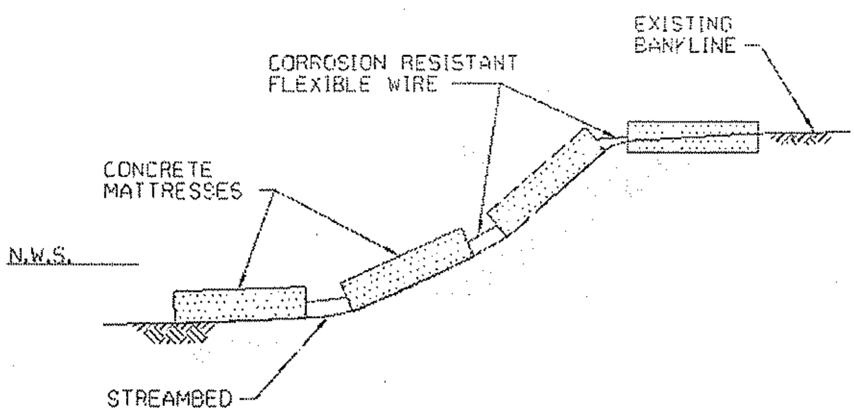
For this measure, concrete-filled sacks are stacked one on top of another to provide reinforcement to the toe. The concrete sack revetment does not require bank reshaping. A typical structure is shown in Figure 28.



Concrete Sack Revetment
Figure 28

Concrete Mattresses

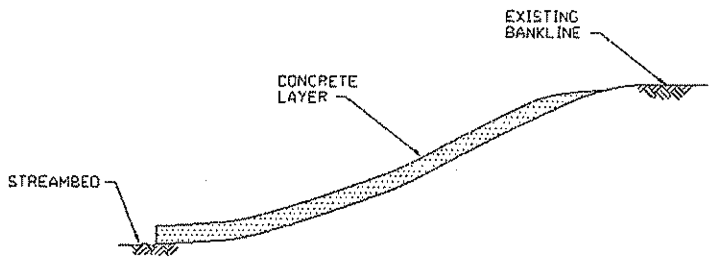
Concrete mattress revetment, shown in Figure 29, does not require extensive streambank reshaping. Concrete mattresses are positioned on the streambank and attached to one another by wire cable.



Concrete Matresses
Figure 29

Concrete Paving

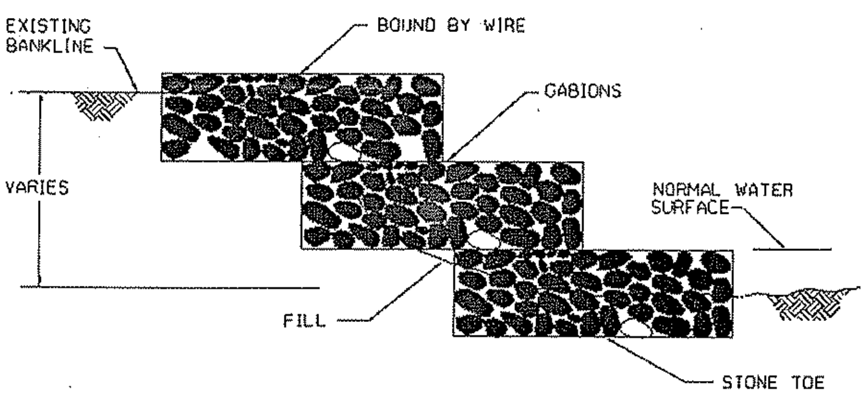
Concrete paving revetment does not require extensive streambank reshaping. Concrete pavement is poured directly on the streambank to prevent additional streambank erosion from occurring. A typical structure is shown in Figure 30.



Concrete Paving
Figure 30

Gabion Revetment

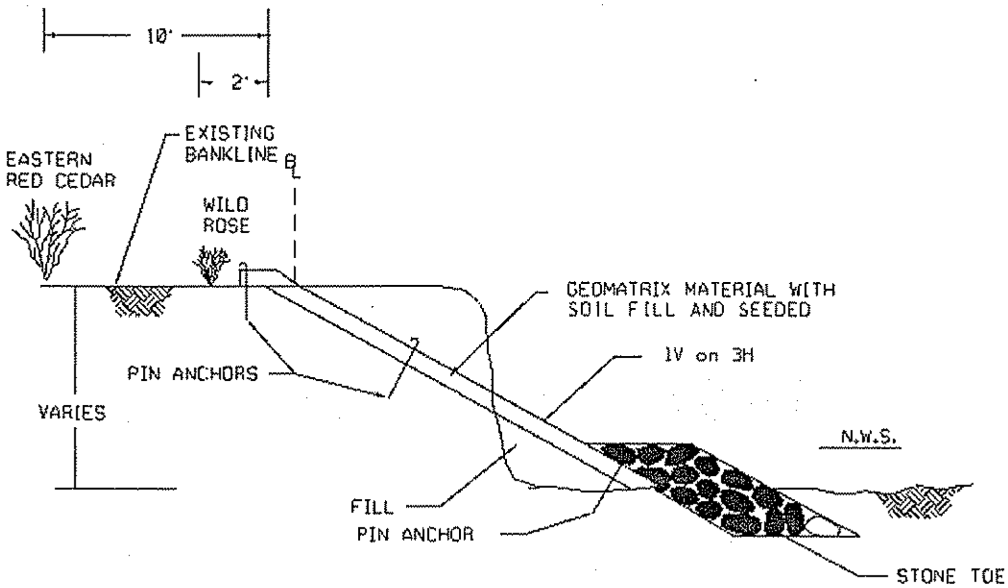
Gabion revetment, shown in Figure 31, is constructed by excavating the streambank just enough to allow placement of the gabions. The gabions are typically placed one atop another.



Gabion Revetment
Figure 31

Cellular Geomatrix Revetment

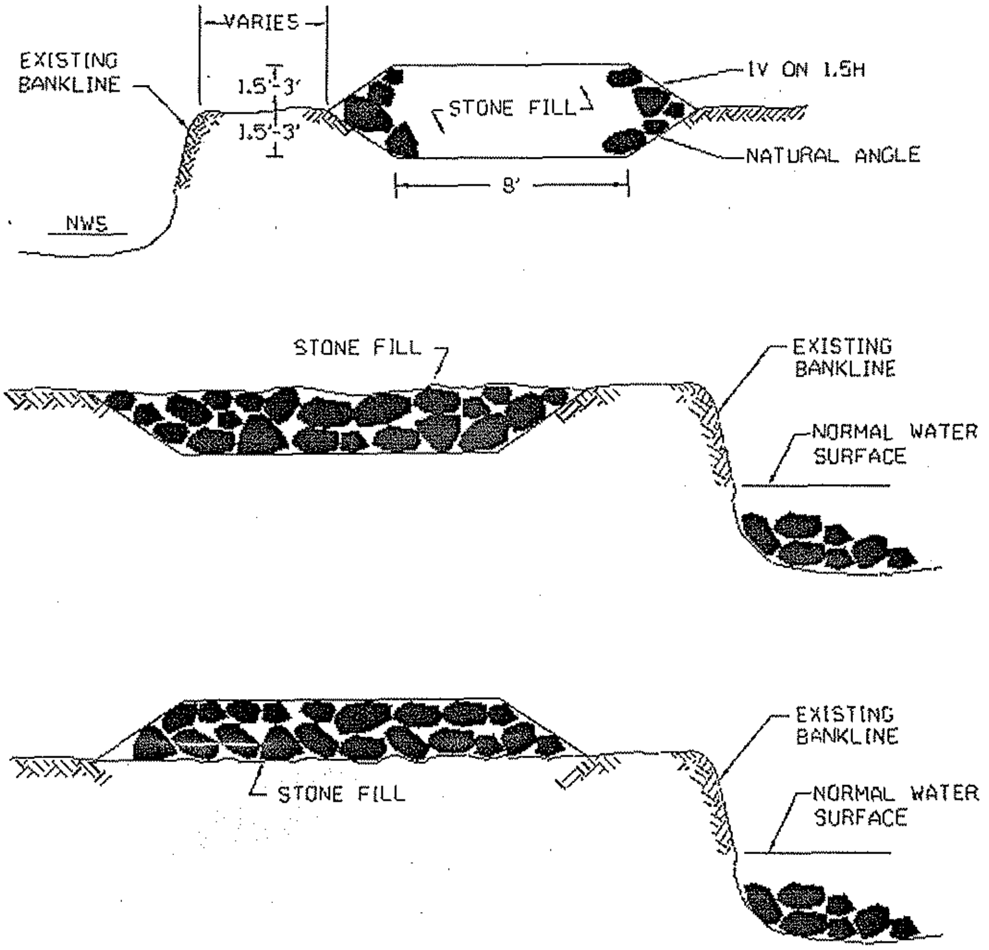
Geomatrix revetment, shown in Figure 32, is constructed by shaping the bank and placing a geoweb material on top of it. The geoweb is then filled with soil and seeded. A stone toe is used to support the geoweb. Various trees and shrubs are also planted for stability.



Cellular Geomatrix Revetment
Figure 32

Windrow Revetment/Refusal

Windrow revetment consists of placing stone fill parallel to the channel bankline and allowing current erosion forces to cause the stone material to migrate to the toe of the streambank. A typical windrow structure is shown in Figure 33.



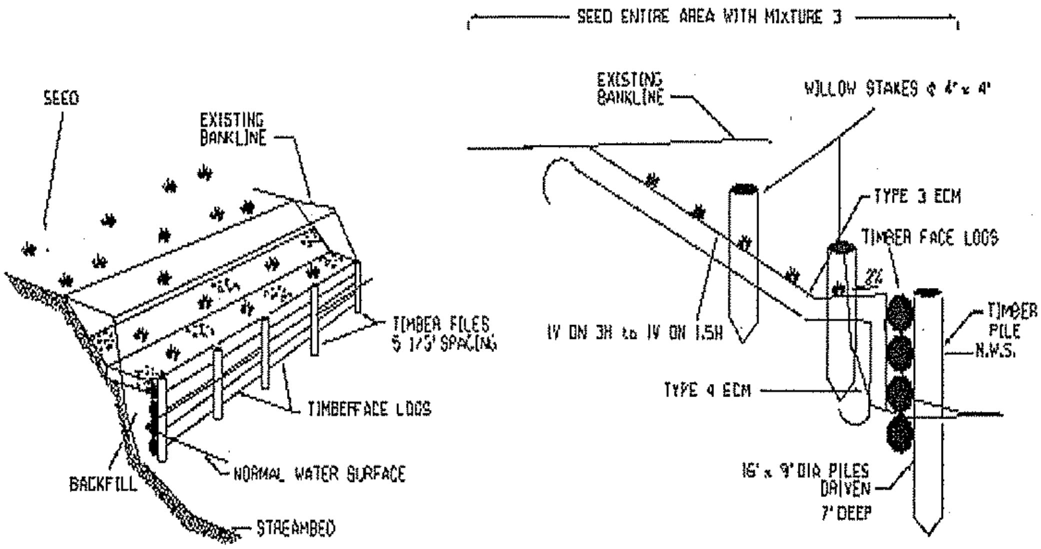
Windrow Revetment/Refusal
Figure 33

Nonstructural Measures

Nonstructural measures rely on vegetation or brush to provide streambank stabilization. While the costs are substantially lower than structural measures, nonstructural streambank stabilization projects may not provide a permanent solution to the erosion problem.

Timber Pile Stabilization

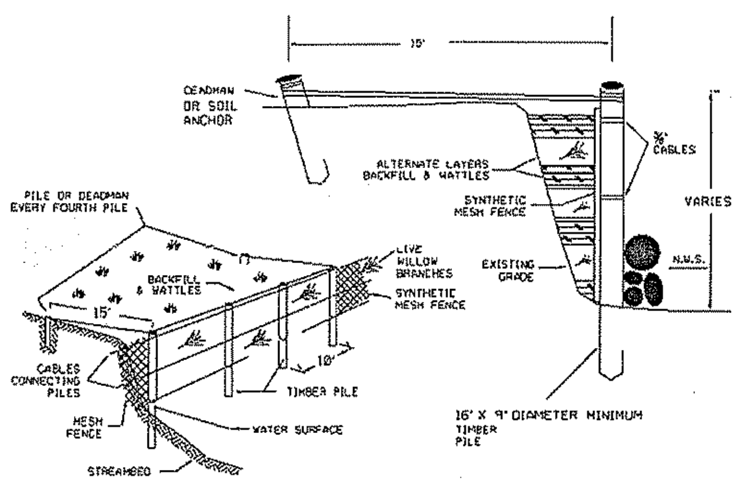
The streambank stabilization measure using timber piles is constructed by reshaping the bank and driving round timber piles into the streambed. Timber logs are placed horizontally to support the streambank. Willow stakes are driven into the streambank for additional stability. A typical structure is shown in Figure 34.



Timber Pile Stabilization
Figure 34

Mesh Fence Stabilization

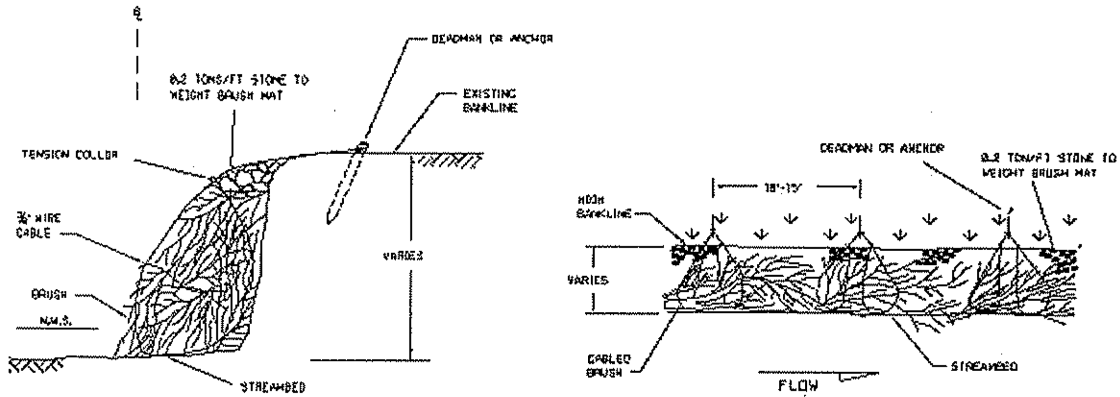
The streambank stabilization measure shown in Figure 35 is constructed by driving round timber piles into the streambed, spaced 10 feet apart on center. The system is anchored using a pile or deadman on every fourth pile. The wire mesh is reinforced by cables, connected to the pile, and is used to support alternating layers of backfill and wattles. Wattles consist of layers of branches and twigs. Reshaping of the bank is not necessary when using this method of stabilization.



Mesh Fence Stabilization
Figure 35

Cable Brush Revetment

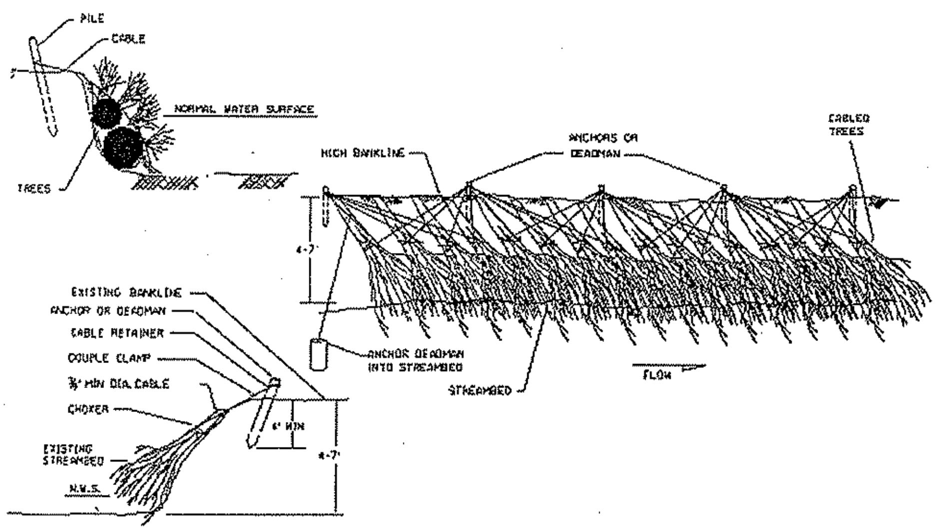
No reshaping of the bank is required for cable brush revetment. The brush is secured to the bank using cable wire with tension collars. The cables are anchored with deadmen, and 0.2 ton of rock per foot is placed on top of the brush as a brush hat. A typical structure is shown in Figure 36.



Cable Brush Revetment
Figure 36

Tree Revetment

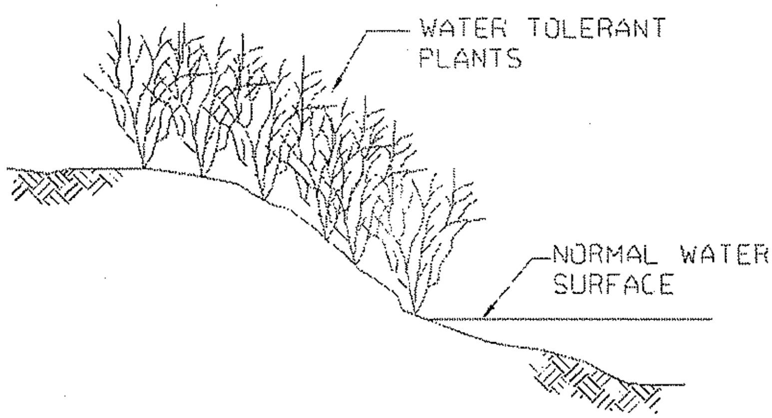
No reshaping of the streambank is required for the tree revetment shown in Figure 37. The trees are secured to the bank and to the streambed using cable wire with tension collars. The cables are anchored with deadmen spaced 5 to 7 feet apart.



Tree Revetment
Figure 37

Water-Tolerant Vegetation

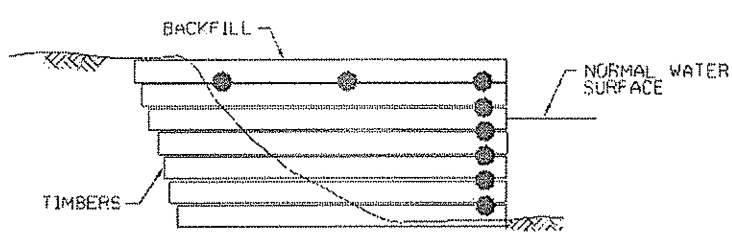
Planting water tolerant vegetation is a simple method of controlling streambank erosion. A typical structure is shown in Figure 38.



Water Tolerant Vegetation
Figure 38

Timber Cribbing

Timber cribbing revetment, shown in Figure 39, consists of constructing a crib, which consists of a rectangular timber structure, perpendicular to the flow of the channel to capture sediment, and regain a portion of the streambank that had been previously eroded.



Timber Cribbing
Figure 39

Reference Documents

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Chapter 6

Grade Stabilization Planning and Design Measures

This chapter presents (1) planning and design measures for determining future degradation estimates, for determining future streambank widening estimates, for site evaluation, and structure selection; (2) grade stabilization design criteria; and (3) riprap evaluation. Four grade stabilization structure types (sheetpile, H-pile, rock sill, and concrete block) are discussed in detail.

Estimation of Future Streambed Degradation

Prediction of future streambed degradation is useful for determining structure placement. If it is determined that an appreciable amount of degradation will occur, then a decision can be made as to the most beneficial placement of the control structure. A significant amount of research has been done on describing and predicting streambed degradation in rivers and streams. Prediction models have been developed based on characteristics of flow regime and on the behavior of water in open-channel flow conditions. Other models characterize the response of a stream's longitudinal profile to the nature of the material through which the stream is flowing. Several empirical models use the decelerating nature of the degradation process over time to estimate the stable bed elevation.

The objective of this section is to determine the most accurate model to use for estimating streambed degradation in western Iowa. Ease of application, amount of data required, and accuracy of the calculated degradation depth are the main criteria for model use. Streams are described according to channel and drainage basin characteristics that influence the degradation process.

Degradation of the western Iowa Missouri River tributary streams has been occurring since the early part of this century and was thought to be caused by the lowering of the Missouri River streambed. Dr. Robert Lohnes of ISU documented vertical channel stability for the Missouri River between Sioux City, Iowa and Omaha, Nebraska, between 1879 and 1952. Since that time, streambed degradation has occurred from Gavins Point Dam, near Yankton, South Dakota, to the mouth of the Platte River, approximately 25 miles downstream from Omaha. Clear-water (sediment-free) discharge, flow regulation from Missouri River mainstem dams in the 1950's and 1960's, and channel realignment are believed to be the causes of this channel lowering. Bed erosion ranged from about 6 feet near Sioux City to roughly zero at Omaha. However, streambed lowering of the Missouri River is not the cause of the western Iowa stream degradation.

Most western Iowa streams of significant runoff area were straightened between the late 1800's and mid-1900's to alleviate frequent valley flooding and create additional farmable land. Original land survey notes of western Iowa (circa 1852) frequently mention swamps and marshes with rather sluggish stream systems. Straightening frequently resulted in a drastically different stream alignment. Piest noted that the Tarkio River confluence with the Missouri River is now 16 miles farther upstream on the Missouri River than its original location and that the mouth of Little Tarkio Creek has been moved 9 miles upstream on the Tarkio River. Daniels noted that channelization of Willow Creek resulted in a 23.1 percent reduction in length and an increase in average stream gradient from 5.16 to 7.66feet per mile and from 7.50 to 8.48 feet per mile in the two sections of the creek straightened between 1916 and 1923.

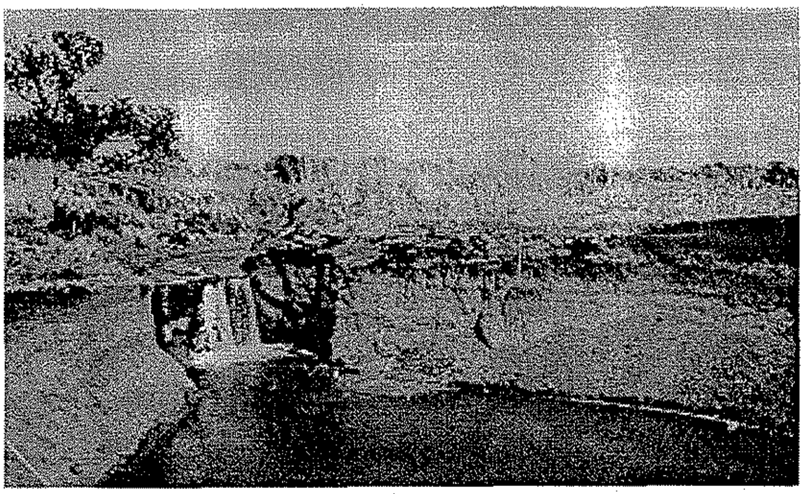
When streams are straightened, channel slope can be dramatically increased and frictional resistance decreased from the original meandering channel. These factors lead to an increase in the velocity and scour ability of the flowing water. With an increase in slope and discharge, the drainage system responds by degrading and widening in order to reach a new quasi-equilibrium state. Daniels reported that Willow Creek at the Monona-Harrison County line had increased from an original 1920 top width and depth of 30 feet and 11 feet, respectively, to a top width of 115 feet and a depth of 42 feet. Floodwaters were now confined to the channel instead of flowing out onto the floodplain which increased the shear or tractive force applied on the streambed.

Review of field data indicates the primary cause of streambed degradation in western Iowa is due to stream straightening. This is supported by residents of Willow Creek who have indicated the channel started to deepen and widen soon after construction. Field and Reed discuss the result of straightening Indian Creek in Pottawattamie County, Iowa. A section of their work reads: "After due consultation with eminent engineers, it was determined to make a straight ditch to allow the water to escape and prevent flooding. This was done, but the creek rose to the occasion and commenced eating off the rear of abutting lots. Old wooden bridges which spanned the creek were replaced by arches of stone resting on piling, but another shower and those bridges became a memory." Many similar quotations abound in other historical documents. Accumulation of this type of data demonstrates Misouri River tributary stream straightening as the predominant cause of streambed degradation.

Mechanism of Streambed Degradation

The majority of streambed degradation takes place by the upstream movement of a headcut or knickpoint. A knickpoint represents a discontinuity in the longitudinal profile of a stream. Upstream advancement of a knickpoint does not occur at a constant rate but rather depends on the discharge, location in the stream, and upstream conditions. This sporadic movement has been verified by both field and flume studies, which indicate that rapid movement coincides with periods of high flow.

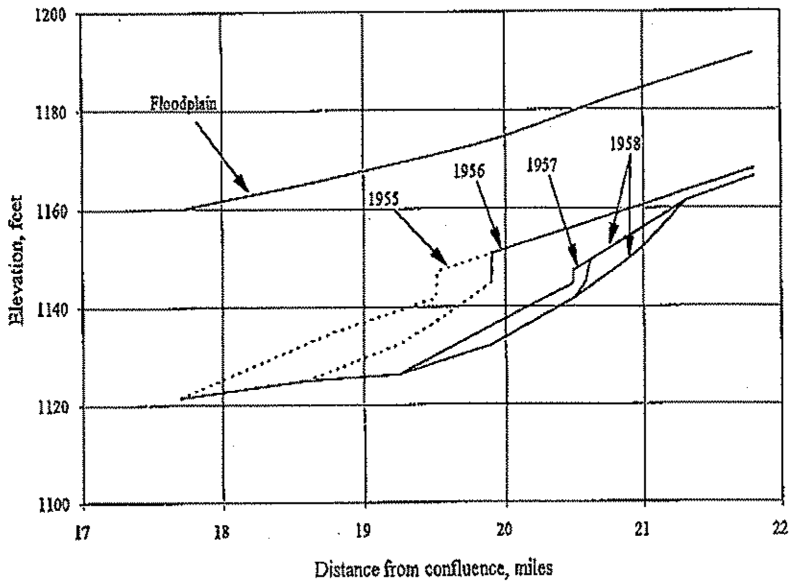
When a knickpoint passes a given location, there will be a substantial increase in both channel width and depth, as shown in Figure 40. The passage of a knickpoint exposes a horizontal seepage line above the lowered water surface, and mass movement of the saturated material can occur. Movement of this lower material undermines the bank above and initiates mass wasting of the streambank.



Knickpoint Migrating Upstream

Channel After Knickpoint has Migrated Upstream
Figure 40

A knickpoint starts as a steep-faced overfall but will eventually progress into a series of small riffles or bed disturbances as it moves upstream. Depending on the position in the stream system and the type of knickpoint, channel erosion may occur after passage of the overfall. Figure 41 identifies a knickpoint on Willow Creek upstream from the confluence with the Boyer River and the amount of degradation that occurred after the knickpoint's passage. A considerable amount of erosion continued to occur after 1955 but has gradually decreased around 1958.



Knickpoint Movement along Willow Creek Upstream from the Boyer River Confluence (Daniels, 1960)
Figure 41

Streambed Degradation Estimation Models

Estimation of the stable equilibrium profile of a stream is a formidable task. Information on original stream plans and profiles and constructed drainage ditch plans is rare. This has made analysis of the pre-disturbed stream system even more difficult. Seven streambed degradation estimation models are presented in the following paragraphs.

Various degradation estimation models, based on many different stream characteristics and hydraulic parameters, have been developed. Much time and effort have been spent on determining which of these may be applicable to western Iowa streams. Hack observed that a stable stream profile could be determined simply from an analysis of the existing profile. Lohnes and Massoudi used the hydraulic behavior of open-channel flow to determine the equilibrium conditions of a stream. An analysis of stream degradation over time has yielded several empirical models based on power, hyperbolic, and exponential functions. All of these models have been applied to western Iowa streams with a varying amount of success.

Geomorphic Model. Hack studied approximately fifteen streams in seven regions of Virginia and Maryland with drainage areas ranging from 0.12 to 375 square miles. Each stream was analyzed at different locations along their length, and measurements were made to determine stream length, drainage area, channel slope, channel cross section, and bed material size. The streams ranged from those with extremely steep slopes of over 500 feet per mile to coastal streams with very gentle slopes. Bed material size ranged from a few inches in the coastal streams to boulders several yards in diameter in the steeper mountain streams.

Hack discovered that for a given drainage area, the channel slope is directly proportional to a power function of the size of bed material:

$$S=25\cdot\left(\frac{M^{0.6}}{L}\right)$$

where S is the slope in feet per mile (1 m/km = 0.5 ft/mi), M is the median particle size of the bed material in millimeters, and L is distance from the headwater in miles (1 km = 1.62 mi). Hack also showed that for a given size of bed material, the channel slope is inversely proportional to a power function of the drainage area:

$$S=18\cdot\left(\frac{M}{D_A}\right)^{0.6}$$

where D_A is the drainage area in square miles (1 mi^2 = 2.62 km^2). An analysis of bed samples for streams in western Iowa (obtained by Gregg Hadish of Golden Hills RC&D) does not reveal a systematic change with distance from the headwater. The relation of bed material size to distance from the headwater for the western Iowa streams is exactly the opposite of that observed in typical fluvial systems -- the median bed material size increases with distance from the headwater. In general, a decrease in particle size occurs with distance from the headwater, which reflects the decrease in channel slope and flow velocity. Hack described the profile of a stream with the following equation:

$$Z = k\ log\ L + C$$

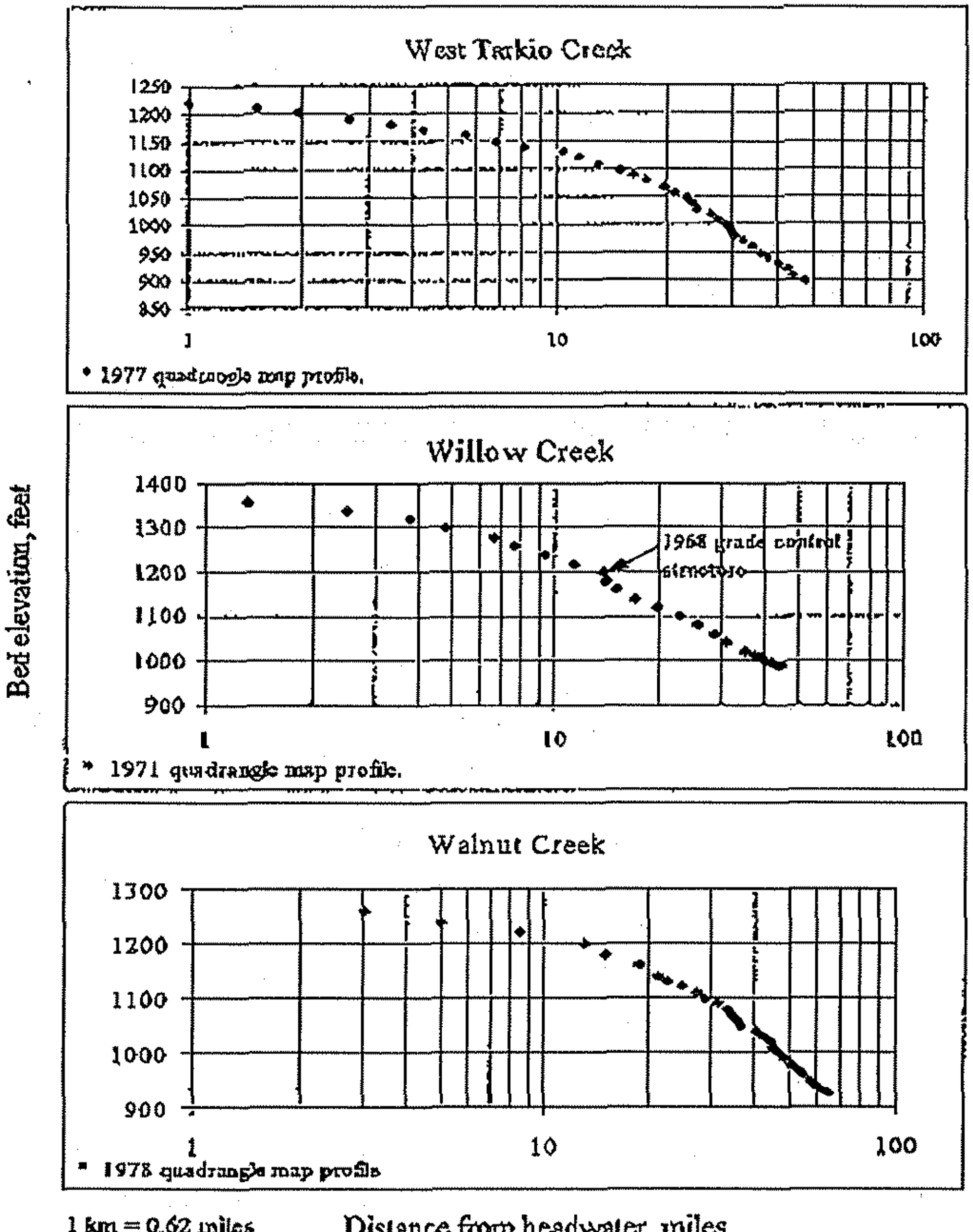
where Z is the fall from the drainage divide, L is the distance from the drainage divide, and k and C are constants. The equation when plotted on a semi-logarithmic graph is a straight line in regions of constant geology. When the geology of the streambed changes, an abrupt change in the channel slope occurs that reflects the change in erosion resistance of the material. A more resistant material will be able to carry a given flow at a higher slope angle than a less resistant material.

From the semi-logarithmic profile plot, the stable reach of the stream can be defined by identification of the downstream section where the slope is constant and the bed elevations have not changed significantly over time. Extrapolation of the straight line from this stable section upstream identifies the upstream reach that is actively downcutting. The projected stable profile lies below the existing profile, and the elevation difference between the stable profile and the actual profile is the estimated amount of future degradation.

Daniels first applied the geomorphic model described by Hack to Willow Creek. Daniels examined the 1958 profile of Willow Creek using semi-logarithmic graph and noted the lower reach existed at a lower slope angle than the upstream creek reach. He assumed the lower reach was in equilibrium, fit a straight line to this section, and projected it upstream using the semi-logarithmic paper. By comparing the 1958 profile with the projected line, Daniels estimated up to 59 feet of future degradation 10 miles from the headwater of Willow Creek. Lohnes compared the 1966 Willow Creek profile with the stable profile calculated by Daniels and verified Daniels' degradation estimation. It was determined that the elevations of the estimated stable profile did not differ significantly from the measured 1966 profile. After 1968, the profile of Willow Creek from river mile 15.0 to river mile 32.5 were influenced by several grade stabilization structures. Surveys performed in 1980 and 1993 could not be used to confirm the stable profile in this reach of Willow Creek.

The geomorphic model appears to fairly accuraely estimate the amount of downcutting on a short reach of Willow Creek, but little success has been experienced on other western Iowa streams and a longer reach of Willow Creek. As shown in Figure 42, the profiles of western Iowa streams plot concave downward, not as straight lines, on semi-logarithmic graphs. Without a straight semi-logarithmic profile, it is not possible to estimate the amount of future degradation with the geomorphic model.

The variation of the western Iowa stream profiles from those studied by Hack may be explained by several factors. All the streams in Hack's study had width to depth ratios that increased in the downstream direction, but this trend is not apparent on western Iowa streams. Western Iowa streams also do not follow the relationships shown in Hack's study for channel slope versus bed material size to drainage area and accumulation of drainage area with distance from the headwater. The combination of these factors makes western Iowa streams differ significantly from those of Virginia and Maryland, thus making the geomorphic model unsuitable for application in western Iowa.



1 km = 0.62 miles
1 m = 3.28 feet

Geomorphic Modelling Curves
Figure 42

Stratigraphic Model. The stratigraphic model is based on the premise that streams will degrade vertically until they encounter a more erosion resistant bed material. Glacial till exhibits a higher erosion resistance than the overlying loess and would slow or halt the erosion when the channel degraded to its elevation. Contact with bedrock, which displays a much higher erosion resistance than the overlying loess, would act as a limiting condition for the stream by setting a new, higher base level. Most bedrock deposits are at a low enough elevation that contact in the upper reaches of the stream is not likely, but visual inspection of a bedrock topography map establishes this possibility in stream reaches near the Missouri River flood plain.

The channel may also cease degradation by "channel armoring." Channel armoring is the buildup of coarse-grained material deposited by the stream or by the removal of fine-grained bed material. This material is heavier and more resistant to transport and protects the channel bottom from additional erosion. Livesey has shown that as little as 10 percent coarse-grained material in a standard sieved sample may be sufficient to provide the bed armor.

A severe limitation of this method is the lack of reliable data on the erosion resistance and shear strength of subsurface strata. The little knowledge that is available has proven to be inconclusive in determining the ultimate erosion depth of the streams and suggests that this model may be inappropriate for estimating future degradation..

Tractive Force Model. Massoudi developed the tractive force model based on hydraulic principles of stream erosion and morphometric observations on Willow Creek. In this model, Massoudi states that a stream will respond to changes in discharge and velocity by adjusting its channel geometry and slope angle. In this way, the stream minimizes its energy gradient and returns to a quasi-equilibrium condition.

The tractive force model is founded on five main assumptions. The first is that the shear stress exerted by the water on the channel bed is given by the following expression:

$$\tau = \gamma_w YS$$

where τ is the shear stress (lb/ft^2 or Newton/m^2), γ_w is the unit weight of water (lb/ft^3 or Newton/m^3), Y is the depth of water above the channel bed (ft or m), and S is the slope of the energy grade line (ft/ft or m/m). For simplicity, it is typically assumed that the slope of the energy grade line is the same as that of the channel bed. With this relation, the shear stress exerted on the channel bed can be determined for any flow condition.

The second assumption is that the width to depth ratio is constant at any cross section for any depth of degradation and systematically changes downstream. This change is expressed by the following equation:

$$W/D = 0.077L + 5.23$$

where W is the channel width (feet), D is the channel depth (feet), and L is the distance from the stream headwater (miles). A trapezoidal channel cross section with 1V on 1H sideslopes is the third assumption. Bottom width, as with the width to depth ratio, is assumed to change systematically with the following relation:

$$B = 1.67L + 12.79$$

where B is the channel bottom width (feet) and L is the distance from the stream headwater (miles). It should be noted that both equations for width to depth ratio and bottom width were determined only from Willow Creek. A constant Manning's roughness coefficient of 0.035 is the fourth major assumption. The fifth and final assumption is that the critical erosion resistance can be determined from the re-stabilized reach of the stream.

Critical erosion resistance for a stream is determined by plotting various dated stream profiles and noting sections where the bed elevation has not changed significantly over time. The slope of this region is determined and the depth of flow calculated using the original channel geometry. The depth and slope are then used to calculate the erosion resistance for this section. This is assumed to be the critical erosion resistance for the entire stream and is used for the determination of the future depth of degradation.

The flow rate at any cross section is determined by using a modified form of Manning's Equation:

$$Q = \frac{1.49}{n} A R^{\frac{2}{3}} S^{\frac{1}{2}}$$

where Q is the flow rate (cfs), n is Manning's roughness coefficient, A is the cross-sectional area (ft^2), R is the hydraulic radius of the cross section and is equal to the wetted area divided by the wetted perimeter (ft), and S is the slope of the channel at that point (ft/ft). The flow calculated with Manning's Equation is then used to calculate the recurrence interval, RI, of the bankfull discharge for the following equation:

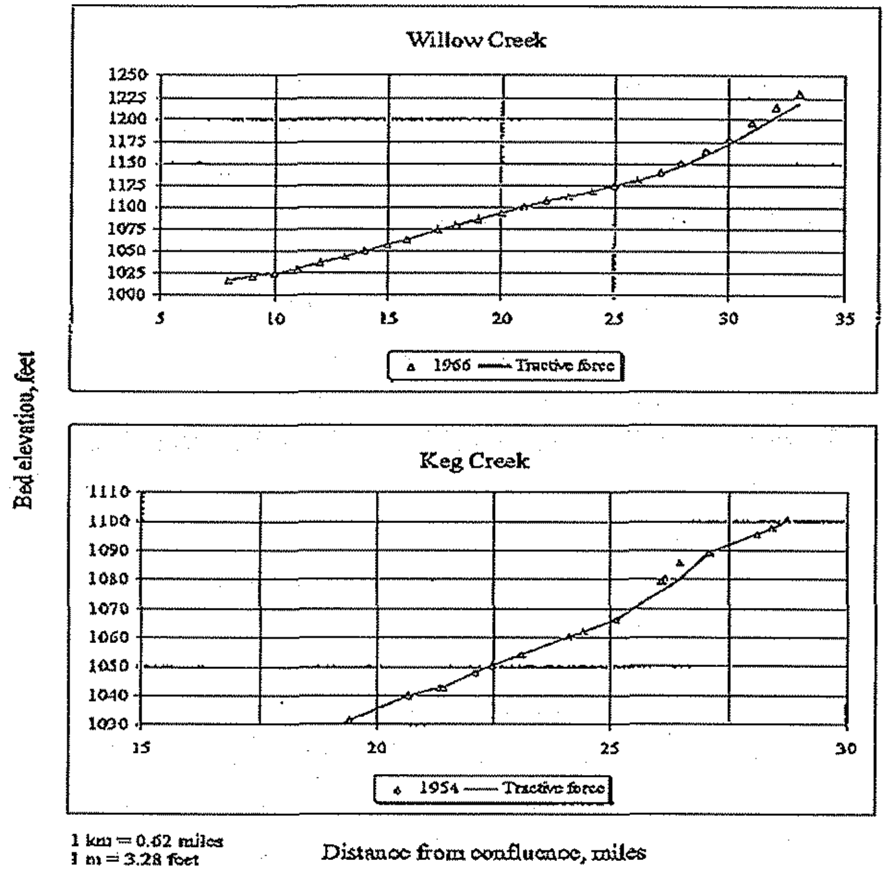
$$Q = 422.58\ (LF)\ (RI)^{0.301}\ (D_A)^{0.504}$$

where Q is the flow (cfs), LF is the land use factor for the area, and D_A is the drainage area (mi^2). When Massoudi performed this procedure for Willow Creek, he obtained a recurrence interval of approximately 2 years. It is generally thought that a flow of this magnitude corresponds to bankfull flow and is the most geomorphically important flow to stream channel form.

Application of the tractive force model is an iterative process in which the discharge, depth of flow, stream slope, and bed elevation are all calculated and adjusted until the shear stress acting on the channel bed is below the critical erosion resistance of the stream. This process is repeated over several small profile sections until the entire stable profile of the stream is generated. Levich has written a computer program that performs these calculations automatically. This program was used on several western Iowa streams based on the parameters developed from Willow Creek. Results of using this model on Willow Creek and Keg Creek can be seen in Figure 43, where surveyed bed elevations are compared to the tractive force profile..

Several problems with this model are evident and create limitations on its applicability. The equations for width to depth ratio and bottom width are based on the characteristics of Willow Creek and may not be applicable to other streams. The bankfull discharge used in the determination of the depth of flow is that of the original premodified channel. In order to calculate this flow, the cross-sectional geometry of the premodified channel must be known, but this information is not available for many streams. A significant amount of survey information is also necessary to determine the channel geometry and longitudinal profile of the existing stream. The iterative computations make this model time consuming and uneconomical to use for frequent widespread application.

Assuming a bankful discharge corresponds to a recurrence interval of two years may not be accurate. Previous research has shown a recurrence interval of 2 years does not necessarily provide a good estimate of bankful discharge. In his study of instantaneous bankful discharge on 36 rivers, Williams found only one-third of the rivers to have a recurrence interval near 2 years. The recurrence interval for all the rivers Williams studied ranged from 1.01 to 32 years. With this wide variation among the rivers, which was contributed to slope and other channel characteristics, Williams stated that a recurrence interval of 2 years may in fact not represent the bankful discharge for a given stream system, and therefore suggest limited success in applying this model to strewams having configurations that are different than Willow Creek.



Tractive Force Application
Figure 43

Velocity Adjustment Model. Using hydraulic principles, Lohnes showed how open channels change their geometry to accommodate their flow. After the streams were straightened, the channel gradient was extremely high, resulting in an increased flow velocity. The primary stream characteristics that adjust in an unstable condition are width, depth, and channel slope. As these parameters change, the velocity will decrease as a decay function with time, ultimately reaching an equilibrium value. This method utilizes the continuity equation:

$Q = vA$

where Q is the flow in the channel, v is the flow velocity, and A is the channel cross-sectional area, and Manning's equation:

$$v=\frac{1.49}{n}R^{\frac{2}{3}}S^{\frac{1}{2}}$$

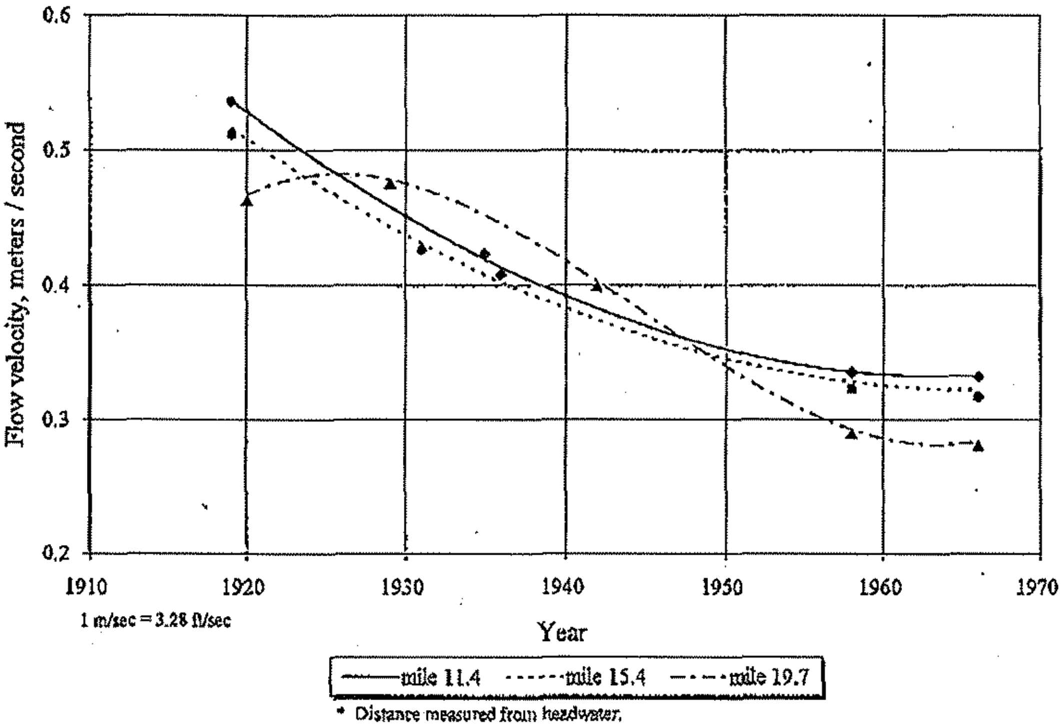
where v is the flow velocity (ft/s), R is the hydraulic radius of the channel, n is Manning's roughness coefficient, and S is the channel slope (ft/ft). By assuming

$$R=\frac{A}{2D+B} \quad \text{and} \quad A=D\cdot B$$

where D is the channel depth and B is the channel width, Manning's equation can be reduced to the following expression:

$$v=\frac{1.49}{n}\left[\frac{D*B}{2D+B}\right]^{\frac{2}{3}}S^{\frac{1}{2}}$$

Using this equation and the continuity equation, the depth and velocity of flow in a channel can be calculated for any discharge. As the channel adjusts to the new flow conditions, velocity decreases nonlinearly with time. Figure 44 shows the decay of velocity over time on three cross sections of Willow Creek. A nonlinear function such as an exponential or a hyperbolic equation could be fit to the data to estimate the ultimate stable channel velocity. After determining this minimum velocity, the value could be substituted back into Manning's equation, which could be solved for the channel geometry parameters. In this way, the depth of flow could be calculated and the stable channel elevation determined. While the results of this model can be used to determine stable channel reaches, the requirement of detailed channel surveys make its application limited.



Velocity Adjustment Application
Figure 44

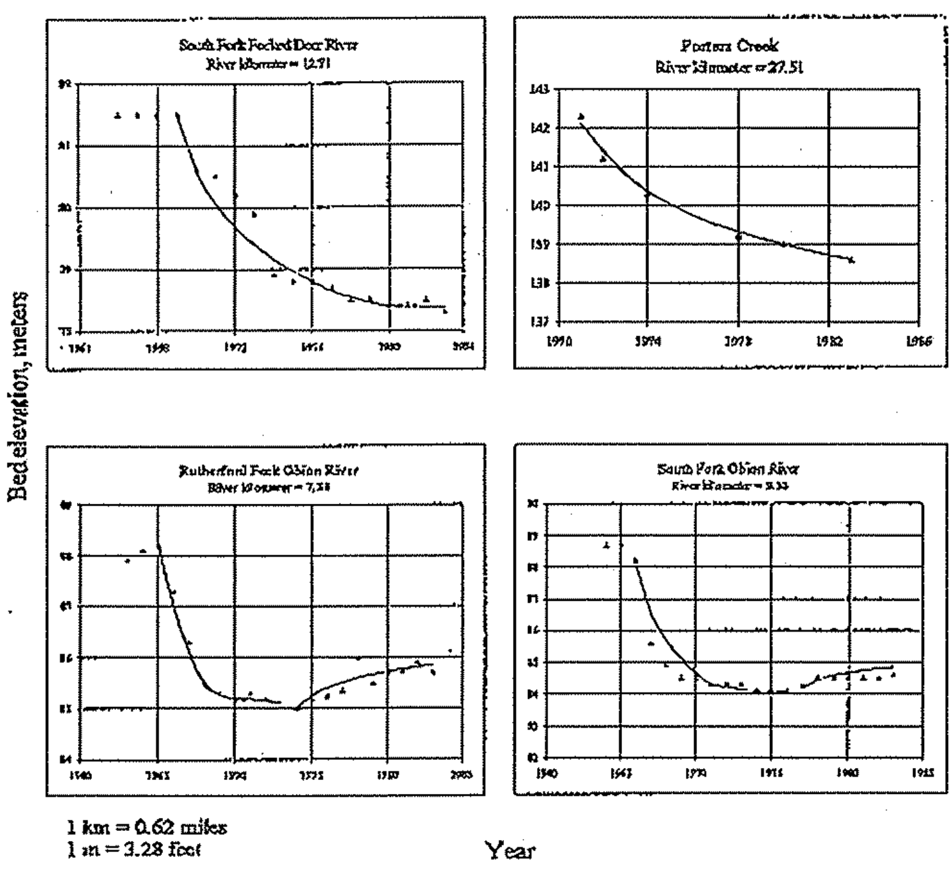
__Power Function.__ Simon used the power function to characterize the decelerating rate of degradation over time for several rivers and streams in western Tennessee. The major rivers of that region flow in channels composed of medium-sand and silt-clay banks, while the small tributary streams flow through extensive deposits of Wisconsin loess. In Tennessee, as in Iowa, frequent flooding caused the river and stream systems to be channelized and straightened, thus decreasing the stream length and increasing the channel gradient.

The form of the power function for either Imperial or metric units is:

$$Z = Z_o(t)^k$$

where Z is the elevation of the bed at time t_o, Z_o is a coefficient determined by regression representing the premodified bed elevation, t is the time in years since the onset of degradation with $t_o = 1.0$, and k is a coefficient determined by regression representing the nonlinear rate of bed level change. This function was found to accurately fit the empirical data for the bed level adjustments over time for the western Tennessee fluvial systems. However, as shown in Figure 45, the power function does not provide an asymptote value representing the stable bed elevation and thus should not be used in degradation estimation for western Iowa streams.

The power function is a curve fitting method and is limited in its application as a degradation estimation model because it does not provide stability to streambed elevations With this model, degradation would continue indefinitely, although at an increasingly small rate. A time limit could be placed on the function at which degradation could be considered to be complete, but this would be an arbitrary decision and may not reflect the actual conditions of the stream system.



Power Function Application
Figure 45

Hyperbolic Model. Williams and Wolman used a hyperbolic function to describe bed level adjustment over time in reaches downstream from dams. Twenty-one dams constructed on rivers in the Midwest and semiarid Southwest regions of the United States having bed material size ranging from silt to gravel were studied, and detailed examination was made at 114 cross sections.

The rate of degradation was noted to be fastest immediately after the onset of erosion and then decreased with time, becoming asymptotic toward a lower, stable bed elevation. Much of the elevation versus time data in their study show significant scatter. This may be the result of variation in flow release from the dams, but no information on this was given by Williams and Wolman. After analyzing the degradation trends, they determined the best fit to the data was obtained using a hyperbolic function with the following form:

$$z=\frac{t}{a+bt}$$

where z is the degradation depth at any time, t is time in years ($t_o = 0$), and a and b are both constants. This equation can be linearized by rearranging the terms resulting in the following equation:

$$\frac{t}{z}=a+bt$$

When this equation is plotted on an arithmetic scale, the coefficient "a" will be the intercept and "b" will be the slope of the line. Both constants can be determined by linear regression. The reciprocal of "a" represents the initial rate of degradation, k, represents units of length per year, and the reciprocal of "b" is the asymptote on a plot of bed elevation versus time and represents the estimated ultimate depth of degradation, Δz. The relation as stated does not directly result in the stable bed elevation, but rather it provides the ultimate depth of degradation. In order to obtain the final bed elevation, the ultimate depth of degradation, 1/b, must be subtracted from the initial bed elevation. The elevation of the ditch immediately after construction is taken as the initial bed elevation.

The time required for the channel to reach stability using the hyperbolic method is calculated using only the initial rate and depth of degradation coefficients determined from the linear regression. Degradation time is determined by first noting the ultimate degradation depth is equal to 1/b:

$$\Delta z = \frac{1}{b}$$

Second, any time, t_p, less than the ultimate degradation time will produce a depth of degradation equal to a percentage, p, of the ultimate depth. Substitution of t_p and p produces the following expression:

$$p \cdot \Delta z = \frac{t_p}{a + bt}$$

Division of equations results in an expression for the percent, p, of degradation depth:

$$p = \frac{bt_p}{a + bt}$$

Solving this equation for time, t_p , yields:

$$t_p = \left(\frac{p}{1-p} \right) \frac{a}{b}$$

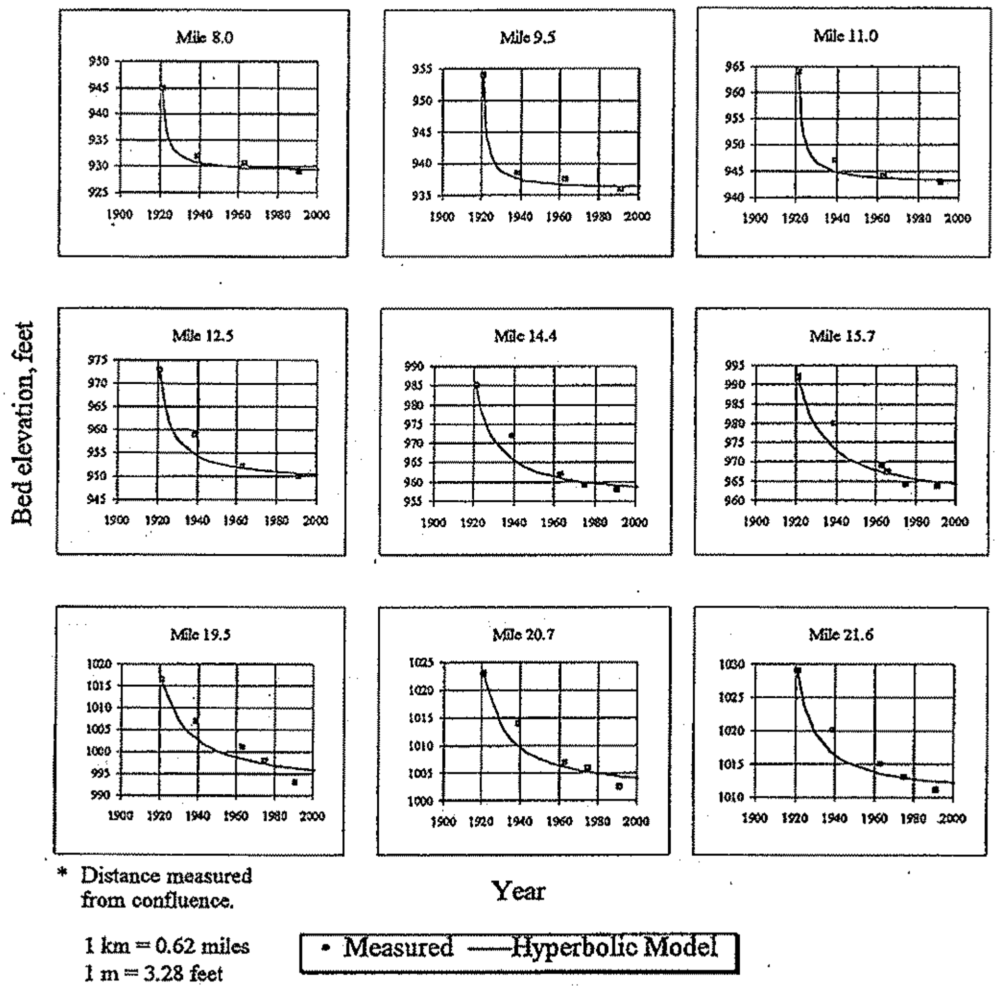
This equation has a slightly different form than the one developed by Williams and Wolman which makes the mathematical computation simpler. This equation is preferable to back-calculating the degradation time because rounding of the degradation depth can cause significant variations in the computed degradation times. The variable "p" is taken as 0.98 since the time to reach the ultimate degradation depth becomes infinite as the function approaches the asymptote.

The hyperbolic model, as with the power function model, is a curve fitting method that has no physical basis. The accuracy of the best fit line for the linearized hyperbolic function is susceptible to several sources of error. Small depths of degradation, a limited number of data points (especially in the first few years of degradation), and irregularities in the degradation curve all tend to produce a low correlation. The constructed ditch elevation is required to perform the analysis, which limits its application to cross sections where these data are available.

Application of the Hyperbolic Model to West Tarkio Creek. For application of the hyperbolic model, the bed elevation data must be modified to degradation depth by subtracting the dated bed elevation from the initial elevation. Once this has been done, the data are linearized by dividing the time since straightening by the degradation depth at that time. When these data are plotted against time since straightening, the result is a straight line having the following form:

$$\frac{t}{z} = a + bt$$

Substitution of the generated coefficients allows for the degradation depth at any time to be calculated. Figure 46 shows the calculated elevation versus time curves with the observed data.



* Distance measured from confluence.

1 km = 0.62 miles
1 m = 3.28 feet

Hyberbolic Fit of West Tarkio Creek Data
Figure 46

Exponential Model. Graf noted that an exponential decay function used by physicists and chemists to describe the relaxation times of radioactive materials and chemical mixtures would be useful for the description of geomorphic adjustment in fluvial systems. When a stream system is disturbed, initial adjustment will be rapid, but the rate of change will slow over time. Graf successfully used this function to calculate changes in gully length over time and indicated that adjustments in the gully system take place at a decreasing rate and approach a steady state condition. Lohnes first used the exponential decay function to describe streambed degradation over time and showed the physical basis of the model with the following relation:

$$\frac{dZ}{dt} = -k'Z$$

where $\frac{dZ}{dt}$ is the rate of vertical degradation, Z is the elevation of the stream cross section, and k′ is a rate constant. By separating the variables and applying the boundary conditions, the exponential decay function reduces to

$$\ln\left(\frac{z}{z_0}\right) = -k't$$

where Z is the elevation at time t and Z_0 is the initial bed elevation. Lohnes applied this relationship to several cross sections on Willow Creek and the Tarkio River with good results, shown in Figure 47. This form of the exponential function describes the decreasing trend of the degradation process but does not show the channel approaching an ultimate equilibrium elevation. According to this function, the degradation would continue indefinitely at an ever decreasing rate.

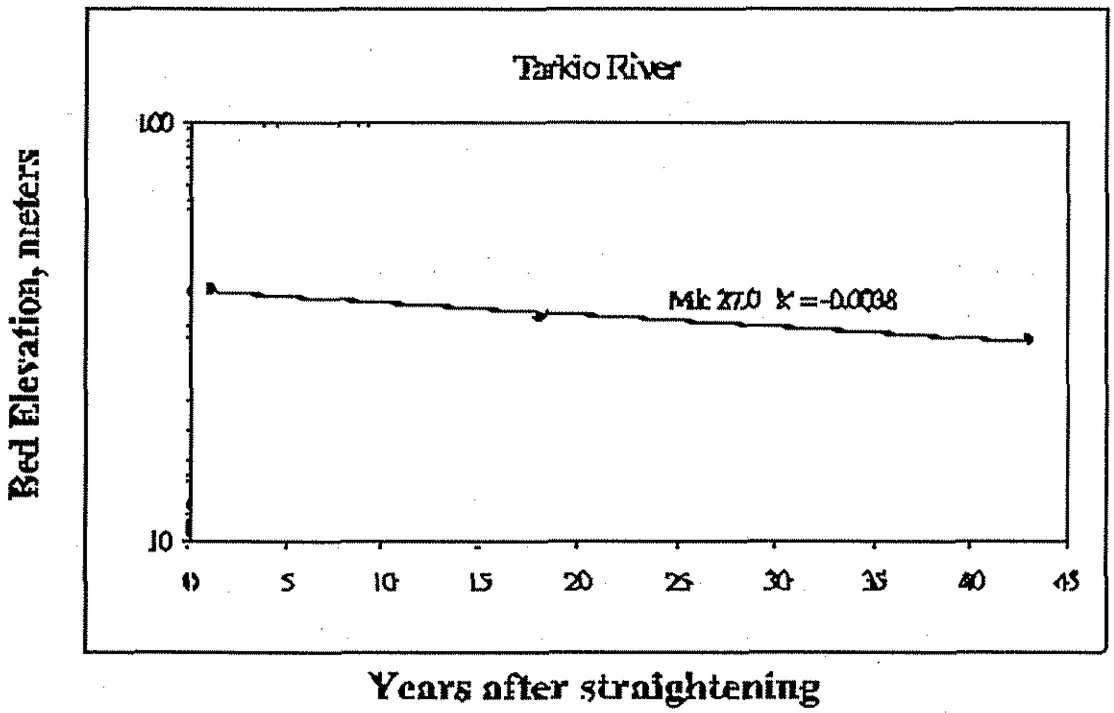
Simon described bed level changes over time using the exponential model and adding an asymptote term to the function to represent the ultimate stable elevation of the channel. The dimensionless form of the equation is:

$$\frac{z}{z_0} = \frac{z_{ult}}{z_0} + \frac{\Delta z}{z_0} e^{(-k't)}$$

where Z_o is the bed elevation at t = 0, Z is the elevation at any time t, Z_{ult} is the ultimate degraded elevation, ΔZ is the total change in elevation, and k′ is the rate constant. For ease of discussion, the dimensionless equation is written as:

$$\frac{Z}{Z_o} = c + de^{(-k't)}$$

where $c = \frac{Z_{ult}}{Z_o}$ and $d = \frac{\Delta Z}{Z_o}$. The coefficients c, d, and k′ are determined by regression. The ultimate stable elevation for a section can be obtained by multiplying the coefficient "c" by the initial bed elevation. The depth of degradation is determined by multiplying the coefficient "d" by the initial bed elevation.



* Distance measured from headwater.

1 km = 0.62 miles
1 m = 3.28 feet

Exponential Function Application to Cross Sections
Figure 47

The rate constant k′ represents the rate of degradation per unit of degraded depth. In order to obtain the true rate of degradation, i, the regression constant k′ must be multiplied by the total depth of calculated degradation. This results in a number representing an average rate of degradation with units of length per time.

The equation for calculating the time to stabilization for the exponential model is developed as follows. Any percentage, p, of the ultimate degradation depth is expressed by the following equation:

$$\frac{Z}{Z_{o_p}} = c + (1-p)*d$$

where c and d are the coefficients. When solved for time, an expression for the time, t_p, to any percent of the ultimate degradation is produced:

$$t_p = \frac{\ln(1-p)}{k'}$$

where p is the percentage of degradation depth in decimal form and k′ is the rate constant. As with the hyperbolic model, the variable "p" is assumed to be 0.98.

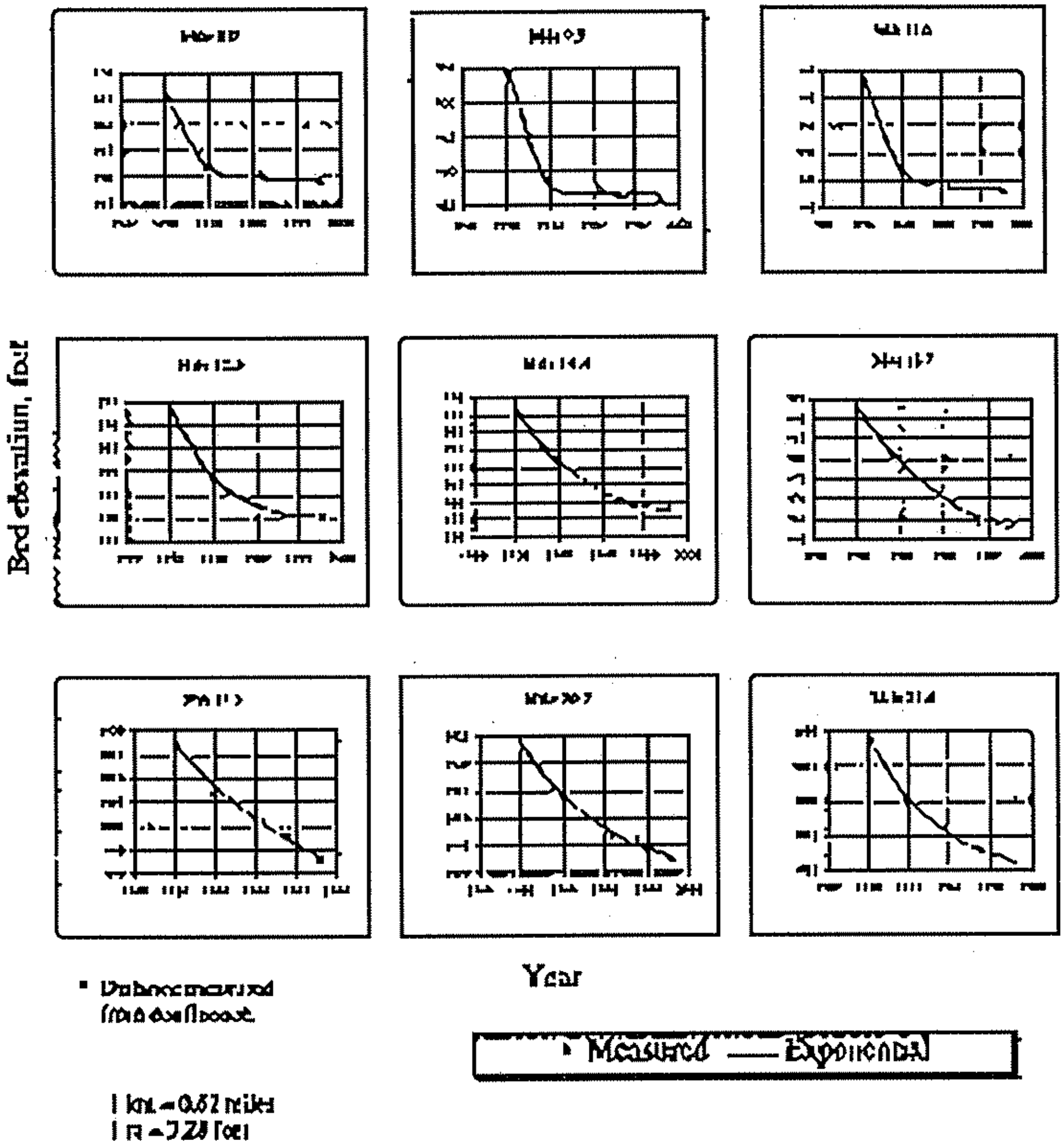
The exponential model is similar to the hyperbolic model in the nonlinear decay of the function with time. The limitations are also similar to those of the hyperbolic model. Without the initial bed elevation for a cross section, the stable bed elevation can not be accurately estimated.

Application of the Exponential Model to West Tarkio Creek. Simon applied the exponential model on bed elevation versus time data to nine cross sections on West Tarkio Creek. He provided the ultimate relative degradation elevation coefficient, c, on each section but did not supply the remaining two coefficients, the amount of relative total degradation, d, and the degradation rate, k′. The detailed method used by Simon for determining the exponential coefficients also was not reported. In order to ensure the exponential method was applied properly, Simon's data had to be re-created for the same cross sections. Stenback suggested using the Solver application in Microsoft® Excel 5.0. This program is an optimization tool that allows a linear or nonlinear function to be solved for any number of variables simultaneously. For this application, Microsoft® Solver was used to minimize the sum of the square errors between the actual and estimated relative degradation depth. The "c" coefficients calculated in this study were nearly the same as those generated by Simon, as can be seen in Table 6. The small variation is attributed to error in reading the bed elevation data on the original elevation versus time graphs in Simon's 1995 paper. The fit of the calculated elevation versus time data to the observed data for the nine sections is very good, as shown in Figure 48.

Table 6
Exponential Coefficients for West Tarkio Creek

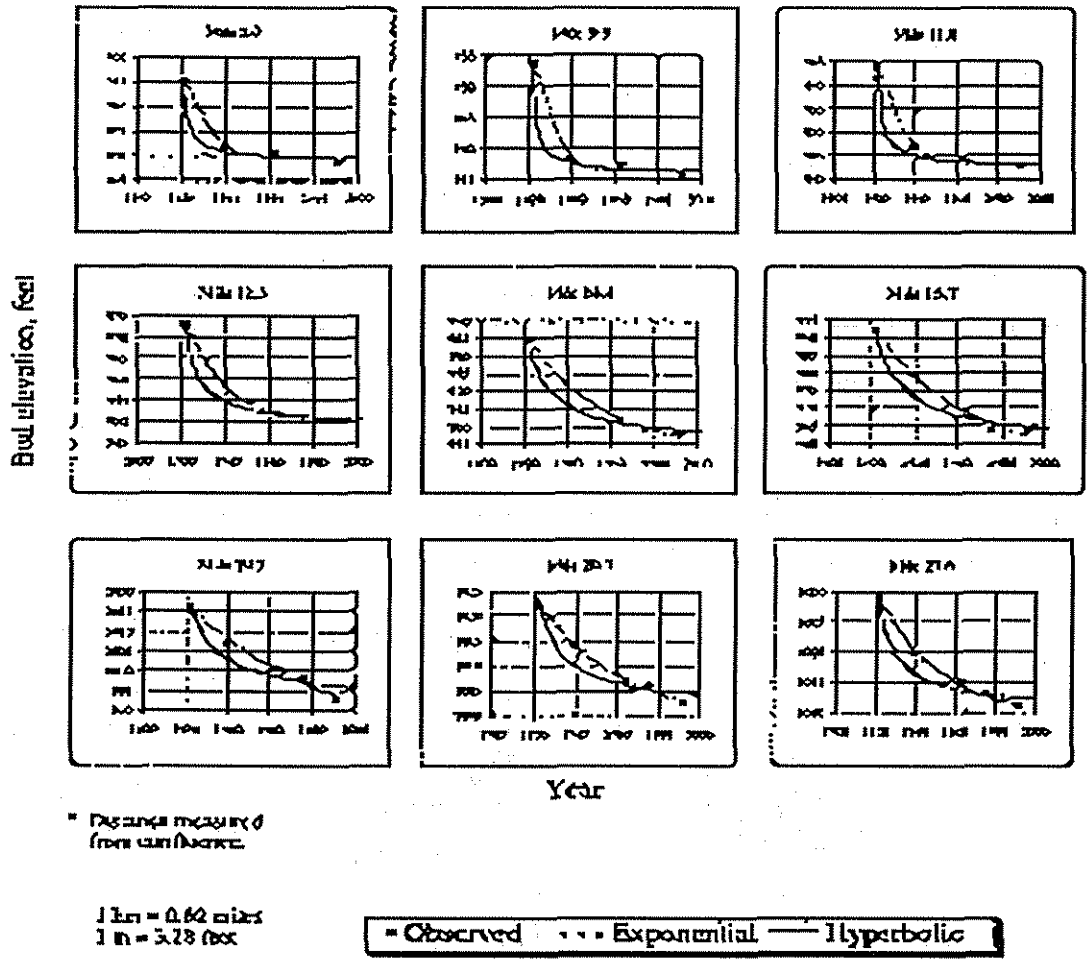
Distance from confluence, miles	c		d	k′	Elev, ft msl	
	Simon (1995)	This study			Initial	Final
8.0	0.9836	0.9837	0.0163	-0.0994	945.0	929.6
9.5	0.9808	0.9818	0.0182	-0.1219	954.0	936.7
11.0	0.9787	0.9785	0.0215	-0.0939	964.0	943.2
12.5	0.9756	0.9755	0.0245	-0.0497	973.0	949.2
14.4	0.9703	0.9687	0.0315	-0.0320	985.0	954.1
15.7	0.9652	0.9639	0.0364	-0.0251	992.0	956.2
19.5	0.9624	0.9628	0.0367	-0.0130	1016.5	978.7
20.7	0.9760	0.9762	0.0237	-0.0247	1023.0	998.7
21.6	0.9705	0.9806	0.0193	-0.0307	1029.0	1009.0

The exponential and hyperbolic calculated elevation versus time curves for West Tarkio Creek, shown in Figure 49, indicate the exponential model better represented the degradation process than the hyperbolic model. The hyperbolic model appears to be insensitive to the intermediate degradation depths and more dependent on recent bed elevations. The upward concavity of the hyperbolic elevation versus time curves is greater than the survey data, which pulls the asymptote of the curves up and results in an estimation of less future degradation than shown by the observed data.



Exponential Model Application to West Tarkio Creek

Figure 48

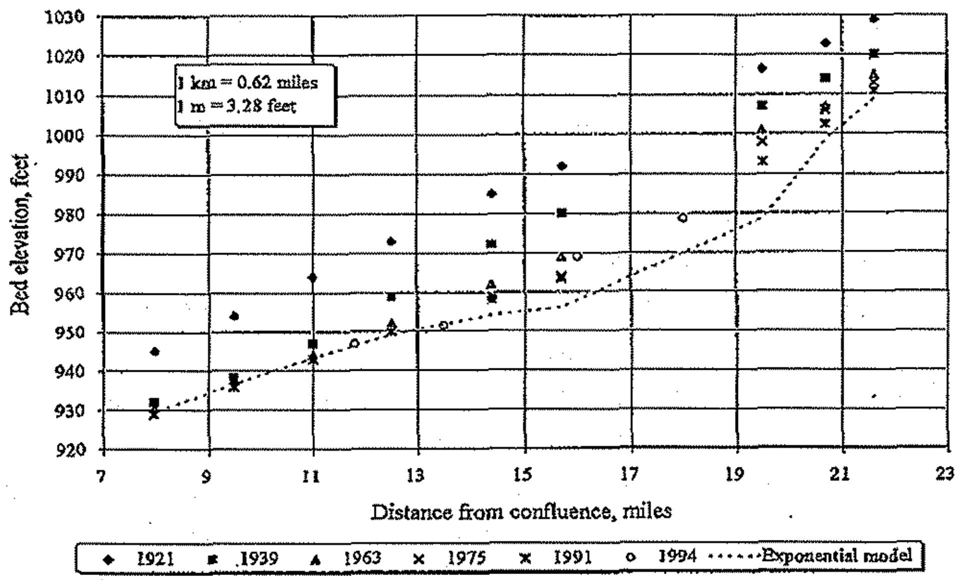


Comparison of Exponential and Hyperbolic Models on West Tarkio Creek
Figure 49

The exponential elevation versus time plots in Figure 49 reveal several important points. The estimated exponential elevation versus time curves accurately fit the experimental data, and the concavity is indicative of the stage of degradation. The more concave the curve, the closer the section is to equilibrium. Sections between river miles 8.0 and 12.5 on West Tarkio creek seem to have attained stability, as indicated by the asymptotic appearance of the curves. The curves for miles 14.4 to 19.5 are less concave and indicate the continuation of degradation. Mile 21.6 and, to a lesser extent, mile 20.7 have more pronounced curvature than the sections immediately downstream which suggests the onset of vertical stability. The coefficient "c", which equals $\frac{Z_{ult}}{Z_o}$, is transformed into the stable equilibrium elevation, Z_{ult}, by multiplying it by the initial bed elevation, Z_o. When this is done for all the sections, the estimated stable profile of the stream is generated, as shown in Figure 50. Up to mile 12.5, both the estimated profile and the stable profile are in agreement. Upstream from this section, the exponential model estimated stable bed elevations that are lower than the 1991 survey data. An over estimation of degradation at miles 14.4 and 15.7 is evident since both of these cross sections were determined to be stable. Interpretation of the results for sections between miles 15.7 and 19.5 is difficult because of the lack of current survey information. Survey data from 1994 exists at miles 16.0 and 18.0, but the mile 16.0 elevation is questionable due to the large elevation difference with the 1991 elevation at mile 15.7. Knickpoint activity between miles 15.7 and 16.0 had occurred in the recent past, thus it is not likely another overfall existed between 1991 and 1994 which would account for the elevation disparity. The 1994 elevation at mile 18.0 plots above the stable elevation calculated by the model. The classification from the stream upstream of mile 15.7 as Stage 4 supports the exponential model's estimation of additional future degradation.

The drop in the estimated profile between miles 14.4 and 20.0 can be explained by several factors. First, this is the location of recent knickpoint migration. The passage of an overfall causes a high rate of vertical adjustment and a sudden drop in the bed elevation. This quick decrease in elevation does not fit the overall decelerating nature of the degradation process and causes several of the elevation points on the elevation versus time plots in Figure 48 to drop below the exponential curves. The data for miles 15.7, 19.5, and 20.7 illustrate this phenomenon. Second, the calculated exponential curve is generated using a least squares regression technique. In this way, the curve is adjusted to minimize the error between the estimated and observed elevations. The elevation versus time data for miles 14.4 and 15.7 show this effect for the 1975 and 1991 data points and results in a calculated curve that falls below the 1991 stable elevation. Finally, the point at which the profile begins to rise coincides with the location of the suspected glacial till outcrop. The higher erosion resistance of the till would lead to less degradation and a steeper channel slope.

Piest provided several additional explanations based on experimental evidence for the large amount of degradation occurring in the mid-section of the profile. The first is that this is the logical location for maximum downcutting since the natural evolutionary sequence dictates that the middle reaches of a stream would be the most deeply incised. Second, some erosion resistance values measured by the NRCS for lower strata of West Tarkio Creek were shown to be lower than overlying sediments, allowing faster erosion to occur. These values are presented in Table 7.



Estimated Exponential Derived Stable Profile for West Tarkio Creek
Figure 50

Table 7
Erosion Resistance Factors for West Tarkio Creek Substrata[1]

Distance from confluence, miles									
12.1		13.6		15.1		18.6		20.6	
Elev.	K	Elev.	K	Elev.	K	Elev.	K	Elev.	K
> 952	0.28	> 967	0.32	> 973	0.43	> 995	0.43	> 1008	0.37
952	0.50	967	0.19	973	0.43	995	0.32	1008	0.28
945	0.50	958	0.38	962	0.12	989	0.38	1002	0.28
941	0.17	954	0.17	< 962	0.34	981	0.40	1001	0.28
< 941	0.40	< 954	0.34	-	-	< 981	0.17	< 1001	0.40

[1]From Piest et al. (1977)

Wischmeier developed this parameter to describe the average soil loss per unit of storm intensity and kinetic energy. The factor K is used in the universal soil loss equation shown below:

$$A = (0.224) \cdot R \cdot K \cdot L \cdot S \cdot C \cdot P$$

where A is the soil loss $(kg/m^2 s^{-1})$, R is the rainfall erosivity factor, L is the slope length factor, S is the slope gradient factor, C is the cropping management factor, and P is the erosion control practice factor. No units were provided for K, but it is assumed the numbers represent relative resistance values where a higher factor represents a more resistant material. The final reason is that the gradient of West Tarkio Creek is much steeper than the comparable reach of the Tarkio River. Slopes of the stable reaches of theTarkio River and West Tarkio Creek are similar, but the gradient of the rapidly eroding section is nearly twice as steep. This steeper slope produces a higher flow velocity and a greater shear force on the channel bed, causing more degradation than experienced by the rest of the stream.

Time to Streambed Stabilization. The time to stabilization for West Tarkio Creek and the exponential estimated time are presented in Table 8. The calculated time is the time required to reach 98 percent of the ultimate degradation depth. The data in Table 8 reveal that the estimated stabilization times fall within five out of the eight observed stabilization time ranges. This does not provide a good measure of model accuracy since the observed stabilization time can only be defined as a time range.

Table 8

Observed and Estimated Stabilization Times (Years) for West Tarkio Creek

Creek	Distance from confluence, miles	Observed	Estimated
West Tarkio	8.0	18-42	39
	9.5	18-42	32
	11.0	42-70	42
	12.5	42-70	79

An overestimation of the stabilization time occurs in the upper reach of both streams. The concavity of the estimated elevation versus time curves in the upstream sections is less than that in the downstream sections. As the degradation rate decreases, less degradation occurs, but over a much longer time span. When the 98 percent degradation depth for the time computation is plotted on the elevation versus time graphs, the depth plots on the low concavity calculated curves at a more recent date, representing a longer time to stabilization. It is not recommended that degradation time be used as a basis for stability determination.

Limited Survey Data Degradation Estimation. The accuracy of the estimated degradation depth was tested by successively removing the more recent data from the estimation analysis. Estimated exponential profiles generated using a varying number of data points on West Tarkio Creek did not vary by more than 1.0 foot, with the exception of two points. The deviations at miles 15.7 and 19.5

on West Tarkio Creek are caused by elevations that do not fit the overall elevation versus time degradation trends of the cross sections.

The minimum number of data points used in the analysis depends on their position in the degradation process and the time period between them. At least three elevations should be used in the analysis to define the decelerating degradation trend, with early points being most important. These points should cover the largest time span possible, with twenty years being a minimum for early elevations. More recent elevation data should span a larger time period to increase the probability of having a greater elevation difference.

Initial Elevation. In many situations, the initial straightened bed elevation, Z_o, is not available. For analyzing cross sections without the initial elevation, several criteria must be first met. At least three bed elevations must be used which define the decelerating degradation trend, i.e. they do not lie in a straight line on an elevation versus time plot. Data used should show the largest elevation difference possible.

The analysis was performed on all the West Tarkio Creek cross-sections, with the results shown in Table 9. Several cross sections display a large elevation difference when compared to the calculation using Z_o. These sections have a linear degradation trend which causes the calculated elevation versus time curve to plot far below what would logically be expected. The estimated stable elevations at miles 11.0 and 12.5 display less than 0.5 foot of difference due to the good decay over time trend of the cross sections.

Table 9

Exponential Estimation Without Using Initial Streambed Elevation on West Tarkio Creek

Distance from mouth, miles	Estimated stable elevation, feet		Elevation difference	
	With Z_o	Without Z_o	meters	feet
8.0	929.6	922.2	2.3	7.4
9.5	936.7	921.0	4.8	15.7
11.0	943.2	942.7	0.2	0.5
12.5	949.2	949.5	-0.1	-0.3
14.4	954.1	956.2	-0.6	-2.1
15.7	956.2	960.1	-1.2	-3.9
19.5	978.7	115.6	263.1	863.1
20.7	998.7	994.7	1.2	4.0
21.6	1009.0	1003.4	1.7	5.6

Estimation of Future Streambank Widening

Estimation of future streambank widening is useful for determining the need for grade stabilization or streambank stabilization projects. If it is determined that an appreciable amount of streambank widening will occur, then a decision can be made as to the most effective use of remedial measures. If the stream is determined to be actively degrading, a grade stabilization structure may be most effective, while in streams where the streambed has become stable, a streambank stabilization structure may be most cost-effective. Estimation of future widening will assist in determining the possible overall length of future bridges, approach span requirements for existing bridges, and the potential loss of land due to streambank widening. Dr. Lohnes, ISU, has developed a prediction model for estimating the amount of future streambank widening for deep loess streams.

Landvoid Model

The Landvoid model was developed to estimate the amount of streambank widening and land voiding that occurs due to streambed degradation. The model relies on an iterative process of comparing a critical height of embankment stability versus the total depth of degradation (i.e., the sum of the existing channel depth and the expected channel depth due to degradation) to determine future streambank widening. It is assumed the streambed will continue to degrade and widen as long as the total depth of degradation is greater than the height of a stable embankment. The embankment slope is made progressively flatter, and thus more stable, until the embankment's stability height is greater than or equal to the total degradation depth. Once this occurs, it is assumed there will be no further degradation, and the additional channel width created by the degradation is then computed. The area of land voided due to streambed degradation can then be computed by multiplying the additional channel width by the length of reach being considered.

<u>**Landvoid Model Description.**</u> The primary focus of this model is the computation of the critical height at which a vertical or sloped streambank will remain stable without collapsing or sloughing. The determination of this height has its basis in the science of soil mechanics and the theory of stability of slopes on soils with cohesion and internal friction.

The failure of a streambank in cohesive soil is usually preceded by tension cracks forming in the soil a short distance behind the crest of the bank. At some point after the formation of the cracks, the soil mass beneath the bank fails by sliding along a curved surface. For the purpose of this model, it is expected that stream degradation causes undercutting of the toe of the streambank. Thus, it is assumed the streambank will slide in a *toe failure* mode, that is, the curved failure surface intersects the streambank at the toe of the embankment.

For a slope that fails along a toe circle, the critical height, Hc, of the slope is determined by the equation:

$Hc = Ns\,(c/Y)$, where

Ns = stability factor
c = soil cohesion (lbs/ft^2)
Y = saturated unit weight of soil (lbs/ft^3)

The value of the stability factor, Ns, depends on both the streambank angle, $\angle_s$, and the soil's internal friction angle, $\angle_f$. The program uses the Culmann method to determine the value of Ns which is given by the equation:

$$Ns = (4 \sin \angle_s \cos\angle_f) / [1 - \cos(\angle_s - \angle_f)]$$

Thus, the actual equation used by the program to determine the critical height of a slope is:

$$Hc = (4c \sin\angle_s \cos\angle_f) / [Y(1 - \cos(\angle_s - \angle_f))]$$

It is important to remember that slope stability analysis is based on several simplifying assumptions. Among them is the assumption that the entire soil mass of the embankment is completely homogeneous, while, in fact, discontinuities within the actual soil may invalidate the results of the analysis. Also, it assumes that values of the soil's cohesion and internal friction angle can be reliably determined, while, in fact, there is a fairly large uncertainty in this respect. In addition, it should be noted that the Culmann method gives good results for vertical and near vertical slopes but gives larger and much less conservative errors for flatter slopes approaching the value of the internal friction angle. For these reasons, care should be taken in evaluating the results of the program, since analysis methods are only approximations at best.

The Landvoid model uses an iterative analysis process to determine the amount of stream widening and land voiding that occurs due to streambed degradation. The first step in using the model is to convert all angle measurements from degrees to radians. Then, the total degradation depth, D_{depth}, can be calculated by summing the existing channel depth, C_{depth} and the expected depth of additional degradation, D_{pot}, so that:

$$D_{depth} = C_{depth} + D_{pot}$$

Next, the critical height of a vertical ($\angle_s$=90 deg) cut in the given soil type that will stand without collapse can be determined by:

$$Hcv = (4c \sin 90 \cos\angle_f) / [Y(1 - \cos(90-\angle_f))]$$

The logic for this step follows that if the stream were to cut a purely vertical channel in the soil, at what critical height would the vertical cut remain stable. The expected depth of degradation versus this critical height should then be compared:

Does $D_{pot} > Hcv$

Obviously, if the expected degradation depth is greater than the critical height of stability then the vertical cut would be expected to fail, widening the channel and thus creating a new, less-than-vertical sideslope.

The next step of this analysis is to approximate the new sideslope angle, $\angle_{ss}$, to which the channel has just degraded. But first, the program performs an intermediate step to determine the magnitude of the height difference, D_{diff}, between the expected degradation depth and the critical vertical height,

$$D_{diff} = D_{pot} - Hcv$$

If the height difference is greater than 10 feet, it is factored into the approximation of the new sideslope; otherwise, it is ignored. The new slope is determined by summing the separate portions of the channel (i.e., the existing portion, expected degradation or critical height portion, etc.) times their respective depths and dividing by the total degradation depth, thus giving a single, approximately equivalent sideslope. This process is described by the equation:

If $D_{diff} > 10'$ $\qquad \angle_{ss}= [75^0(Hcv) + 90^0(D_{diff}) +\angle_{int}(C_{depth})] /D_{depth}$

If $D_{diff} <= 10'$ $\qquad \angle_{ss}= [75^0(Hcv) +\angle_{int}(C_{depth})] /D_{depth}$

where $\angle_{int}$ = initial streambank slope, in radians.

It should be noted that in the above equations it is assumed, since the stream is continuing to degrade, that the once stable vertical cut will itself have failed and slid to a 75 degree angle over its critical height. Also, it should be noted that when height difference, D_{diff}, is included, it is assumed to remain vertical at 90 degrees so that the new approximated sideslope angle will be steeper and thus more conservative for the next iteration. A new critical height, Hcn, is computed for the embankment based on the new slope angle, $\angle_{ss}$, computed above:

$$Hcn = (4c \sin \angle_{ss} \cos\angle_f) / [Y(1 - \cos(\angle_{ss} - \angle_f))]$$

The next iteration begins by comparing the *total* degradation depth versus this new critical height for the sloped embankment:

Does $D_{depth} > Hcn$

Again, logically, if the total degradation depth is greater than the critical stability height, it is assumed the embankments will continue to slide, further widening the channel and creating flatter sideslopes. The analysis continues by approximating the new, flatter side slope angle, $\angle_{ss2}$, to which the channel has degraded. The new slope is estimated to be the average angle between the previous slope $\angle_{ss}$ and the internal friction angle, $\angle_f$ (note that at the internal friction angle the slope would be considered inherently stable):

$$\angle_{ss2} = (\angle_{ss} + \angle_f) / 2$$

Once again, a new critical height, Hcn2, is computed for the embankment based on the flatter slope angle, $\angle_{ss2}$:

$$Hcn2 = (4c \sin \angle_{ss2} \cos\angle_f) / [Y(1 - \cos(\angle_{ss2} - \angle_f))]$$

A final comparison is made between total degradation depth and the new critical height:

Does $D_{depth} > Hcn2$

If the total degradation depth is still greater than the critical stability height, it is assumed the embankment will continue to slide to a final, stable slope, $\angle_{stable}$, estimated to be halfway between slope angle, $\angle_{ss2}$, and the internal friction angle, $\angle_f$, thereby ending with:

$$\angle_{stable} = (\angle_{ss2} + \angle_f) / 2$$

Returning to the initial comparison between expected degradation depth, D_{pot}, and the critical vertical height, Hcv, if the expected degradation depth is less than or equal to the critical height, then the vertical cut would be considered to remain standing. However, since this comparison deals only with the *expected* degradation and does not include the effect of the existing channel depth and slope, the analysis does not automatically assume there is no additional degradation. Instead, the model performs an additional "verification" step, in which a new single, equivalent sideslope angle, $\angle_{ess}$, is approximated and a critical height, Hca, for the embankment as a whole is computed. The approximation of angle $\angle_{ess}$ takes into consideration both the expected and existing degradation, thus:

$$\angle_{ess} = [90\,(D_{pot}) + \angle_{int}\,(C_{depth})] \,/\, D_{depth}$$

and critical height, Hca, is computed using the same equation as before:

$$Hca = (4c \sin \angle_{ess} \cos\angle_f) \,/\, [Y(1 - \cos(\angle_{ess} - \angle_f))]$$

For verification, the analysis compares the total degradation depth versus the critical height:

Does $D_{depth} > Hca$

If the total degradation depth is less than or equal to the critical height, then the verification is complete and it is assumed the slope of the total expected and existing channel will be stable and will not widen beyond the existing (i.e., additional stream widening = 0). On the other hand, if D_{depth} is greater than Hca, then it is apparent the effect of the existing channel on stability is significant and the analysis continues to the same iteration process of computing sideslope angles $\angle_{ss2}$ and $\angle_{stable}$ as described previously.

If, during any of the various comparison steps, it is found that the total degradation depth is less than or equal to the critical stability height (i.e., critical height has become greater than total degradation) then it is assumed that the embankment slope has reached a stable angle and there will be no further widening of the channel. At this point the amount of additional stream widening should be computed by using the sideslope angle ($\angle_{ss}$, $\angle_{ss2}$, or $\angle_{stable}$) that produced the stable slope (i.e., whichever angle produced the critical height value that exceeded the total degradation depth). The amount of additional widening, WA, is computed in the following equation by taking the channel top width created by the stable angle and subtracting from it the existing channel top width:

$$WA = (D_{depth} / \tan \angle_{xx}) - (H / \tan \angle_{int})$$

where $\angle_{xx} = \angle_{ss}$ or $\angle_{ss2}$ or $\angle_{stable}$

Finally, the area of land voided due to degradation, in square feet, is computed by multiplying the additional width, WA, times the length, L, of the reach under investigation:

Area = WA x L

The amount of voided area can be converted from square feet to acres using the conversion:

Acres = Area / 43560

<u>Landvoid Model Concerns.</u> Several concerns with the operation of this model should be noted prior to estimating future streambank widening.

1. The model requires specific soil parameters to calculate streambank widening. These parameters for soil cohesion and saturated unit weight will need to be determined for sites being evaluated.

2. The model user may be required to determine estimated future channel degradation in order to insert values for expected depth of degradation, D_{pot}.

3. The model estimates future streambank widening for one-half of the channel only. The user will be required to double the area calculated for streambank widening and land voiding to determine the impacts to the entire channel.

4. The angles $\angle_{ss2}$ and $\angle_{stable}$ begin to flatten out pretty quickly by making large jumps from the previous angle. The angles may overestimate the critical height, and the flatter they get, the larger the error. As the angles become flatter, the estimated amount of stream widening and land voiding becomes larger.

Example of Streambank Widening Estimation

The following example allows the user to follow the logic established to determine the amount of future streambank widening and land voiding for a typical stream in western Iowa.

For this example, assume: channel length, L = 2000 feet
existing channel depth, C_{depth} = 24.5 feet
expected degradation, D_{pot} = 29.5 feet
total depth, D_{depth} = 24.5 + 29.5 = 54.0 feet
soil cohesion, c = 221 lbs/ft^2
saturated unit weight, Y = 118.5 lbs/ft^3
internal friction angle, $\angle_f = 27.0^0 = 0.471$ radians

critical height, Hcv $=(4c \sin 90^0 \cos\angle_f) / [Y(1 - \cos(90^0 - \angle_f))] = 12.17$ feet
and if D_{pot} = 29.5 feet > Hc = 12.17 feet, which is true,
then, $D_{diff} = D_{pot}$ - Hcv = 29.5 - 12.17 = 17.33 feet

and if D_{diff} = 17.33 feet > 10 feet, which is true,
then, $\angle_{ss} = [75(Hcv) + 90(D_{diff}) + \angle_{int}(C_{depth})] / D_{depth} = 1.432$ radian
then, Hcn $= (4c \sin \angle_{ss} \cos\angle_f) / [Y(1 - \cos(\angle_{ss} - \angle_f))] = 15.41$ feet

and if D_{depth} = 54.0 feet > Hcn = 15.41, which is true,
then, $\angle_{ss2} = (\angle_{ss} + \angle_f) / 2 = 0.952$ radian
then, Hcn2 $= (4c \sin \angle_{ss2} \cos\angle_f) / [Y(1 - \cos(\angle_{ss2} - \angle_f))] = 47.72$ feet

and if D_{depth} = 54.0 feet > Hcn2 = 47.72 feet, which is true,
then, $\angle_{stable} = (\angle_{ss2} + \angle_f) / 2 = 0.712$ radian
then, WA $= (D_{depth} / \tan \angle_{xx}) - (H / \tan \angle_{int}) = 58.2$ feet of additional widening per side

and Area = WA x L = 58.2 x 2000 = 116,400 $feet^2$

and Acres voided = Acres = Area / 43560 = 2.67 acres per each side

From this example, the channel would have an additional total widening of 116.4 feet due to the potential future degradation of 29.5 feet.

Geotechnical Considerations for Design of Grade Stabilization Structures

Soil properties are determined by field examination of the soils and by laboratory index testing. A few shallow borings should be taken at the proposed project site and examined to identify and classify the soil. Samples should be taken from some typical soil profiles and tested in the laboratory to determine grain-size distribution, plasticity, and compaction characteristics.

Classification of the soils is determined according to the Unified Soil Classification System. The Unified system classifies soils according to properties that affect their use as construction material. Soils are classified according to grain-size distribution of the fraction less than 3 inches in diameter and according to plasticity index, liquid limit, and organic matter content. Sandy and gravelly soils are identified as GW, GP, GM, GC, SW, SP, SM, and SC; silty and clayey soils as ML, CL, OL, MH, CH, and OH; and highly organic soils as PT. Liquid limit and plasticity index (Atterberg limits) indicate the plasticity characteristics of a soil. Compaction characteristics indicate the optimum moisture content and the maximum density at which the soil can be placed.

In general, materials classified at the laboratory as CL or CH are considered excellent construction materials for minimizing the design problems stated above.

When evaluating a potential site for construction, the following geotechnical items should be noted and appropriately addressed during the planning and design phase. Many of these items can be incorporated into the design of structures prior to initiating construction, thereby alleviating potential failure of the proposed project.

<u>Streambed and Streambank Considerations</u>

Determine if various geologic formations exist.
Determine if the channel area rests on fill material.
Determine the content of sand or sand and gravel in the streambed.
Determine if exposed bedrock exists.
Determine the stage of channel evolution.
Determine if the channel is actively meandering.
Determine if point bars are present.
Determine if streambanks are bare or well vegetated.

Channel Condition Considerations

Determine depth of channel.
Determine the top width to channel depth ratio.
Determine if riffles are present in the streambed.
Determine if cracks in the soil parallel the top of bank.

Drainage Feature Considerations

Determine if seepage or piping is present along the streambank.
Determine if drainage structures exist along the streambanks.
Determine if surface runoff is directed toward the channel.
Determine if drainage ditches are present.

Since the primary soil concerns with the design of grade stabilization structures are the potential loss of soil due to piping beneath the structures due to the water head differential and the loss of soil beneath the structure due to flowing water, precautionary measures should be taken during site evaluation and design to avoid significant costs for future repair and maintenance.

Site Evaluation

The design of grade stabilization structures requires considerable knowledge of water resources and engineering principles as well as a familiarity with the proposed structure site.

Before implementing grade stabilization measures, a basinwide evaluation should be conducted to determine the extent of channel instability. The basin evaluation should be conducted in enough detail to determine drainage area, flow characteristics, channel configuration, soil types, and stable streambed slope. After the evaluation has been completed, appropriate measures may be taken to reduce or prevent further degradation and streambank erosion. Without a complete understanding of the current river morphology for a specific river system, individual grade stabilization structures not conducive to the system will have a high probability of failure, could induce unanticipated impacts on the river, or could cause excessive operation and maintenance costs. Construction of grade stabilization structures in a river system that has evolved to a stable streambed with only streambank widening would not be an effective use of resources. The key to implementing the correct remedial measure is developing a thorough knowledge of the entire basin, even if the basin crosses jurisdictional boundaries governed by different counties or communities. Since grade stabilization structures will not be effective in a river system that has reached stable streambed grade, streambank stabilization projects will not be effective if implemented in a river system that is currently degrading.

Developing a thorough understanding of the basin will assist in determining whether future estimated channel degradation is slight, requiring no stabilization structures, or whether degradation is active, requiring implementation of remedial measures. Likewise, initial evaluation of the basin may result in low-cost streambank stabilization projects for evolved river systems, or the initial evaluation may result in large estimates of future streambank widening, requiring costly streambank stabilization projects.

Site Evaluation Procedure. The following procedure has been successfully used by several agencies to evaluate proposed construction sites for rehabilitation of degrading river systems in deep loess regions.

(1) Perform field reconnaissance of the proposed site and surrounding basin to identify dominant geomorphic processes and features. Obtain field surveys and soil samples. Utilize historical data, such as surveys and soil borings developed for bridge construction, and drainage evaluations.
(2) Classify each channel within the basin as degrading, aggrading, or stable (in equilibrium). Incorporate the stream classification system developed by the USGS, which is presented in Chapter 3 of this manual.
(3) Determine hydraulic design parameters and geotechnical soil properties for the streambed and streambank. Identify channel slope, normal discharge, and angle of repose. If subareas of the basin have varying degrees of stabilities, divide them into smaller subsets.
(4) Compare each reach of channel in the basin to the stability parameters determined above and confirm as stable, degrading, or aggrading. Anomalies may require additional investigation.

Site Evaluation Checklist. The following checklist of items presented in Table 10 should be considered in any evaluation of a proposed grade stabilization structure. The evaluator should collect enough information about the site and surrounding basin so as to be able to identify potential areas of concern and the possibilities for mitigation.

Table 10
Site Evaluation Checklist

Recommended Items of Evaluation
Determine size of drainage basin
Determine shape of drainage basin
Determine soil characteristics
Determine land uses
Determine location of specific areas of erosion
Determine sources of sediment
Determine soil conservation measures
Determine channel configuration
Determine lengths of channels and streambed slopes
Determine effect of tributaries and other drainages
Determine historical changes in streambed elevations
Determine location and effect of other grade stabilization structures
Determine basin hydrology
Determine flood history
Determine bankful discharge

Grade Stabilization Selection Matrix

After completion of the site evaluation, a structure or series of structures will need to be considered to provide the most cost-effective remedial measure for that particular site. If the existing streambed has gone through a significant amount of degradation, a series of grade stabilization structures may be required to resolve the problem area.

This design manual discusses the use of four grade stabilization structures: the rock sill, the H-pile structure, the sheetpile structure, and the concrete block structure. Each of the four structures presented may be applicable to only a few specific conditions encountered in nature. Although any of the grade stabilization structures presented in this manual may be incorporated to control streambed degradation, in many instances more than one structure or a combination of structures may be required to achieve the final results. In order to determine which, if any, structures are applicable to a specific channel condition, a grade stabilization design matrix has been developed to assist in selecting the most feasible structure. The matrix, shown in Table 11, presents basic criteria for seven categories of design. By determining the design criteria that correspond to a specific channel condition, a grade stabilization structure may be selected from the four structures listed which would then be considered for further detailed analyses and implementation.

Each of the design categories identified in Table 11 are also presented in more detail below. This matrix should only be used as a tool to assist in determining which structure or structures should be considered for final evaluation. Each final site evaluation will then require pertinent field data and a good working knowledge of engineering fundamentals to complete and implement the final design.

Drop Height. Grade stabilization structures are effective in controlling degradation if the drop height does not result in a hydraulic jump downstream from the structure. The hydraulic jump occurs due to the dissipation of the energy derived from the flow becoming supercritical and then transitioning back to subcritical flow. The larger the hydraulic jump, the greater the requirement for a stilling basin, energy dissipating blocks, and streambank protection. Referring to the grade stabilization selection matrix, three subcategories of drop height have been established for vertical drops of less than or equal to 2 feet, between 2 and 3 feet, and between 3 and 5 feet.

Channel Slope. Although most flows occurring in nature are subcritical, the occurrence of supercritical flow is likely if large, concentrated velocities are conveyed throughout a channel reach or the channel streambed slope becomes very steep, typically through the process of channelization. A channel that conveys supercritical flows is highly susceptible to streambed degradation and streambank erosion, while channels conveying subcritical flows tend to have lower velocities and stable sections. A steeper channel slope will usually require a larger structure or a combination of structures to stabilize the channel. In regard to the grade stabilization selection matrix, three subcategories of channel slope are shown for relatively flat slopes, which correspond to subcritical flow, mild slope for channels approaching critical flow, and steep slopes, where flows become supercritical.

Stream Classification. The stream classification category refers to the USGS system of rating channels. This six-stage system is based on the amount of streambed degradation or aggradation occurring in a channel. The grade stabilization selection matrix refers to only Stages 3, 4, and 5 of the classification system. The other three stages are not considered because they tend to focus on naturally stable streams (Stage 1), streams that have been recently modified by construction activities

(Stage 2), and streams that are not degrading,, but are probably undergoing streambank stabilization problems (Stage 6). *The current stage of stream evolution should be determined during the site* evaluation.

Flow Frequency. Typically, the 2-year peak flood event closely resembles the bankful discharge for many natural, unmodified streams. A grade stabilization structure designed accordingly would easily convey frequent flows while increasing the possibility of incurring damages when conveying larger, infrequent flows. The recent flood events in western Iowa have caused large, infrequent discharges to flow through many of the grade stabilization structures constructed over the last few years. Since these floods were of such large magnitude, many of the rock sill structures and riprapped streambanks were moderately damaged, while the sheetpile and concrete block structures suffered only minor damages due to high velocities and debris.

Ice and Debris Effects. Many grade stabilization structures do not adequately convey flows when ice or debris inadvertently causes blockage. Energy dissipating blocks may cause flows to concentrate in one area of a structure if blocked by ice or debris, causing damage to the structure and the downstream channel reach.

Construction Costs. The initial construction costs are ranked into three categories according to actual construction costs determined from similar projects. Referring to the grade stabilization selection matrix, low costs are considered for structures that have a construction cost of less than \$150,000; medium costs, for structures between \$150,000 and \$300,000; and high costs, for structures greater than \$300,000. The grade stabilization structure unit costs presented in Chapter 3 of this manual should be consulted when estimating structure costs.

Maintenance Costs. Operation and maintenance costs are typically a function of the structure size and materials used during construction. If a structure is designed to convey frequent flows using riprap material, there is a substantial possibility that annual maintenance will require riprap replacement due to the likelihood of high velocities moving the stone downstream. On the other hand, if the structure is built from reinforced concrete, the annual maintenance costs can be expected to be low.

Table 11
Grade Stabilization Selection Matrix

Structure	Drop Height			Channel Slope			Stream Classification			Flow Frequency		Ice/Debris Effects		Construction Costs			Maintenance Costs		Total No. of "Y"
	≤ 2 feet	2 - 3 feet	3 - 5 feet	relatively flat	mild	steep	stage 3	stage 4	stage 5	frequent	infrequent	minor ice	minor debris	low	medium	high	low	high	
Rock Sill	Y	Y	N	Y	Y	N	Y	Y	Y	Y	N	N	N	Y	Y	N	N	Y	
H-pile	Y	Y	Y	Y	Y	N	Y	Y	Y	Y	N	N	N	N	Y	Y	N	Y	
Sheetpile	N	Y	Y	Y	Y	N	Y	Y	N	Y	Y	Y	Y	N	N	Y	Y	Y	
Concrete Block	N	N	Y	Y	Y	N	Y	Y	N	N	Y	Y	Y	N	N	Y	Y	N	

Matrix Key: "Y" represents characteristics of proposed construction site that are applicable to structure

"N" represents characteristics of proposed construction site that are not applicable to structure

Matrix Use: Evaluate four grade stabilization structures according to 7 matrix categories. Circle all applicable "Y" and total.

Select structure type according to total number of "Y". Select only once from each category.

Category Definition

Drop Height	Refers to vertical drop in feet, where maximum drop is not greater than 5 feet.
Channel Slope	Natural grade of stream, where flat approaches subcritical flow, mild approaches critical flow, and steep is supercritical flow.
Stream Classification	Based on USGS method of stream degradation classification. Identifies stages to consider for remedial measures.
Flow Frequency	Refers to whether the structure is typically used to pass frequent (2-year) events or infrequent (>2-year) events.
Ice/Debris Effects	Identifies whether ice or debris blockage causes a concern over the operation of the grade stabilization structure.
Construction Costs	Costs are designated as low, less than $150,000; medium, between $150,000 and $300,000; and high, greater than $300,000.

Grade Stabilization Design Parameters

This section of the manual focuses on design parameters essential for the planning and construction of grade stabilization structures. After completing the site evaluation and determining the type of structure to implement from the grade stabilization selection matrix previously mentioned, or from any other preferred selection method, the structure is ready to be designed. Many of the parameters used to design grade stabilization structures are based on general engineering principles related to the fields of hydrology, hydraulics, and geotechnical engineering. Many of these principles must be used in the design of any type of grade stabilization structure, whether the design is for a small rock sill or a large sheetpile structure.

General Design Guidance

Since this manual was developed to provide assistance in stabilizing degrading streambeds, and to offset the costs associated with construction of grade stabilization structures, it is noteworthy to mention, as previously discussed, that it may be practical to estimate the amount of future streambed degradation and future streambank widening and then maintain and operate existing infrastructure based on these estimations. For instance, if a particular stream has already evolved through Stage 5 of the stream classification system and estimates indicate that the streambanks may only widen a small amount in the future, it may be more feasible to consider constructing bridge approaches rather than implementing a grade stabilization structure at this site.

Streambed degradation is primarily active during Stages 3 and 4 of channel evolution. Grade stabilization structures should only be considered for these two stages, and perhaps Stage 5, depending on basin characteristics. It will not be cost-effective to construct grade stabilization structures for rivers and streams that have already evolved through the degrading stages and are now becoming stable.

Design Parameters. Although grade stabilization structures are intended to alleviate the occurrence of streambed degradation within a specific reach of a stream, there are many instances where a structure failed, did not control degradation, or ultimately increased downstream erosion and channel scouring. The process of straightening the rivers during the early part of the century has ultimately changed the natural equilibrium of the river system. Removal of meanders has shortened the overall length of streams while increasing the channel slope. The increased slope conveys flow at a faster rate, increasing velocities and causing degradation to occur as the flow in the channel carries the bed material downstream. Streambed degradation usually progresses upstream until the slope reaches an equilibrium where degradation and aggradation occur simultaneously, stabilizing the channel.

When a grade stabilization structure is designed for a specific reach, the design must ensure that either a single structure or a series of structures address both the current and the future degradation concerns. If structures are spaced too far apart, there is the potential for degradation to continue between structures. If spaced too close together, the construction and maintenance costs may become excessive.

Streambed and streambank erosion may increase in areas where the amount of energy produced through a vertical drop is not dissipated below the structure, resulting in potential failure of the structure. The structure must be designed so that flows are not concentrated through the structure, causing potential failure because of increased velocities. Likewise, flows larger than the expected design frequency should pass through the structure without causing erosion to occur near the abutments, scouring to occur near the downstream toe of the project, or catastrophic failure of the structure.

Grade stabilization structures can be implemented to stabilize streambeds, thereby reducing the amount of valuable farmland eroded annually, or they can be placed downstream from a bridge to protect the structure from collapse. The procedure for the design of grade stabilization structures, as well as the field data and analyses recommended, is as follows:

(1) Review basinwide evaluation checklist and note stream classifications, identifying Stage 3 and Stage 4 reaches.
(2) Obtain site surveys of proposed project locations. This should include channel sections, stream profiles, and bridge data.
(3) Determine the current channel slope by calculating the vertical fall over the length of the reach to be studied. The channel reach selected should take into account any upstream or downstream anomalies which may affect the remedial measures considered. It is the steepness of this slope that causes the streambed degradation and streambank erosion.
(4) Calculate the desired design channel slope and stable channel section by determining the amount of degradation which has taken place in the study reach and farther downstream. The desired slope may be obtained by determining the amount of degradation that has occurred from historical data, if available. At a minimum, the current channel slope, identified in step 3, should be reviewed along with the soil type to determine the stable slope and channel section.
(5) Determine if one or more structures will be required for the study reach. As a rule of thumb, if the total vertical drop within a specific reach exceeds 5 feet, more than one structure should be used to control the degradation.

(6) Determine which alternatives are to be considered. This may depend on construction costs, operation and maintenance costs, or availability of material. Refer to the grade stabilization selection matrix for recommendations.

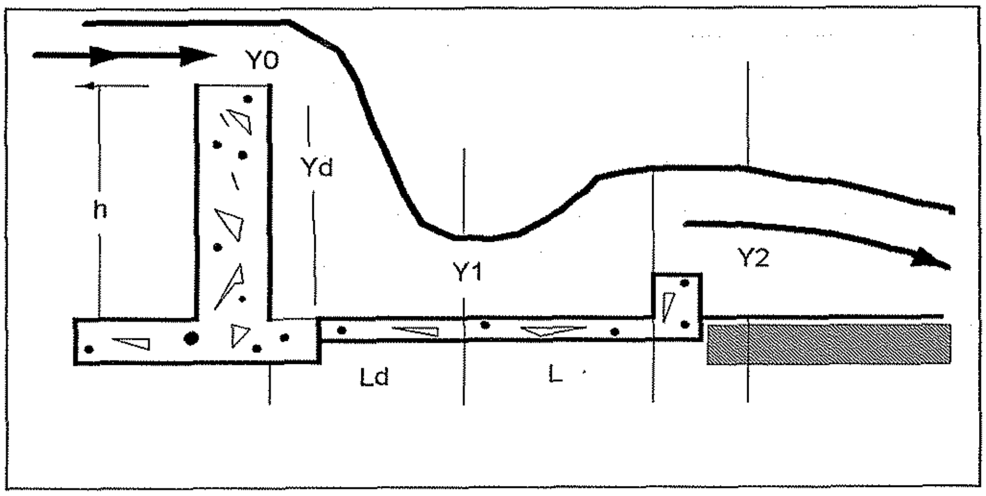
(7) Determine the design discharge for the study reach. Review all available stream gage data for normal channel discharges or determine design discharge by calculating the discharge for the bankfull depth or normal depth from Manning's equation:

$$Q = \frac{1.486AR^{2/3}S^{1/2}}{n}$$ where:

Q = discharge (cfs)
A = cross-sectional area of channel (ft^2)
R = hydraulic radius (ft)
S = slope of channel (ft/ft)
n = roughness coefficient

The 2-year peak flood discharge is typically chosen for the design discharge for streams located in the upper basin since it closely resembles the bankful discharge for most streams. The 2-year flood discharge also forms velocities that may induce streambed degradation when flowing through a steep reach. The design discharge for the lower basin tends to be dependent on water surface elevation due to the lowering of the channel bed and widening of the streambanks. The capacity of the lower basin channel may be in excess of the 100-year event. In some instances, a larger design discharge may be considered so that the risk of failure is minimized.

Typical Grade Stabilization Structure Design. The hydraulic equations that refer to the geometry of a typical grade stabilization structure are presented in the following discussion. All equations are associated with the grade stabilization structure presented in Figure 51.



Grade Stabilization Structure
Figure 51

From Figure 51, it should be noted that depth "Y1 " is the initial depth and depth "Y2" is the sequent depth. The function "Ld " is the drop length, "L" is the length of jump, and "h" is the height of drop. Most drop structures will cause the development of a hydraulic jump downstream from the structure. The flow geometry of the structure may be described in the following terms.

$$\frac{Ld}{h}=4.30\left(\frac{q^2}{gh^3}\right)^{0.27}$$

where "q" is the discharge per unit width of weir and "g" is the acceleration due to gravity

$$\frac{Y1}{h}=0.54\left(\frac{q^2}{gh^3}\right)^{0.425}$$

$$\frac{Y2}{h}=1.66\left(\frac{q^2}{gh^3}\right)^{0.27}$$

$$L=6.9(Y2-Y1)$$

If the tailwater depth is less than "Y2", the jump will recede downstream. If the tailwater depth is greater than "Y2", the jump will become submerged. The spillway will still remain effective until the tailwater exceeds the weir control depth, "Y0".

Economics will be one of the primary driving forces behind the type of structure constructed, and the amount of riprap placed along the sideslopes upstream and downstream from the structure.

Since the flow over the downstream face of a structure is typically supercritical(and in order for the flow to become subcritical), energy must be dissipated. The energy dissipation usually occurs in the form of a hydraulic jump, which was previously discussed. In order to determine the limits of riprap, whether from channel velocities or hydraulic jumps, it is recommended that standard step backwater calculations be conducted for the channel and a series of discharges. Two programs that were developed for determining water surface profiles, HEC-2 and HEC-RAS, are readily available as computer programs for conducting backwater calculations. A distribution of discharges may be developed for the structure location, from which design velocities may be evaluated. The depth of flow and channel velocity will be a function of the channel configuration. In some instances, the difference in depth between the 10-year discharge and the 100-year discharge may be only 1 or 2 feet, while in other instances, the difference may be quite larger.

Solving the above equations for "Y2" will identify the height of the hydraulic jump, thereby indicating the height that riprap should be placed along the channel sideslopes. Additional information regarding the evaluation of riprap is presented elsewhere in this manual.

DNR Structure Requirements

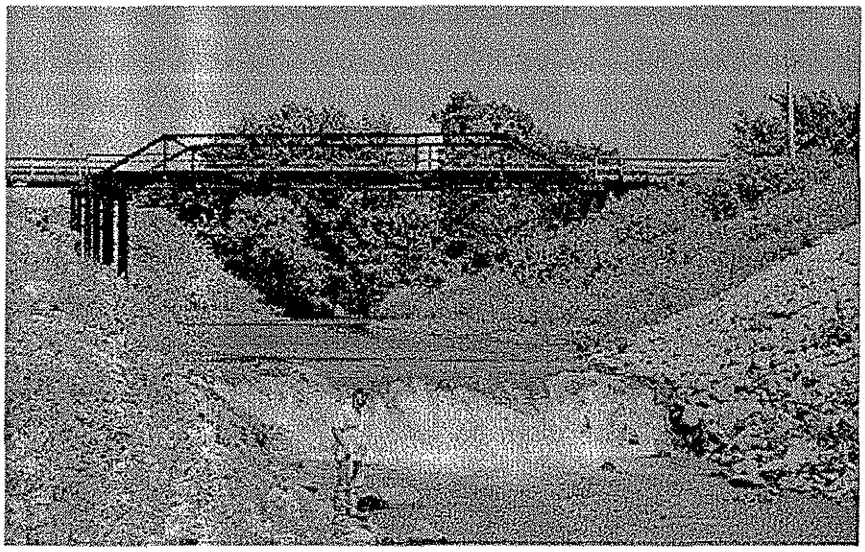
The Iowa Department of Natural Resources (DNR) has issued several memorandums governing the construction of grade stabilization structures in Iowa streams. The memorandums provide recommendations for the design and construction of structures as they relate to fish migration and habitat. Table 12 presents the Iowa DNR recommendations to which the design of all grade stabilization structures in western Iowa will adhere. Although at the time this manual was developed no fisheries classification system had been developed for western Iowa streams, there are numerous locations where the construction of certain types of infrastructure and older grade stabilization structures (greenwood flumes) have essentially prevented the migration of fish and the establishment of fisheries in the upper reaches of some streams and their tributaries. Structures located within these reaches may not be required to meet the Iowa DNR design recommendations. The Iowa DNR Fisheries Bureau in Des Moines, Iowa, should be contacted regarding design recommendations prior to construction of any grade stabilization structure.

Table 12
Iowa DNR Design Recommendations

Prefer full channel width structures - avoid flumes or throated weir sections
Avoid vertical drops - no drop to be greater than 5 feet
Maintain 4 horizontal to 1 vertical or flatter slope on downstream face of structure
Limit design velocities to 4 feet per second at the weir
Provide rough surfaces on downstream face of structure with riffles and pools
Space multiple structure to approximate six times the average bankful width

Rock Sill Structure

The rock sill structure requires the placement of riprap in the channel for it to effectively operate. The size and weight of the riprap will be dependent on the design discharges and the availability of rock in the project vicinity. A typical rock sill is shown in Figure 52.



Rock Sill Stabilization Structure
Figure 52

In some instances, the rock is grouted in place to resist the flow of water, ice, and debris. These structures should not be constructed higher than 3 feet so that failure of the structure does not occur through shifting of the rock. The streambanks at the structure and downstream from the structure require riprap to avoid erosion and headcutting.

H-pile Structure

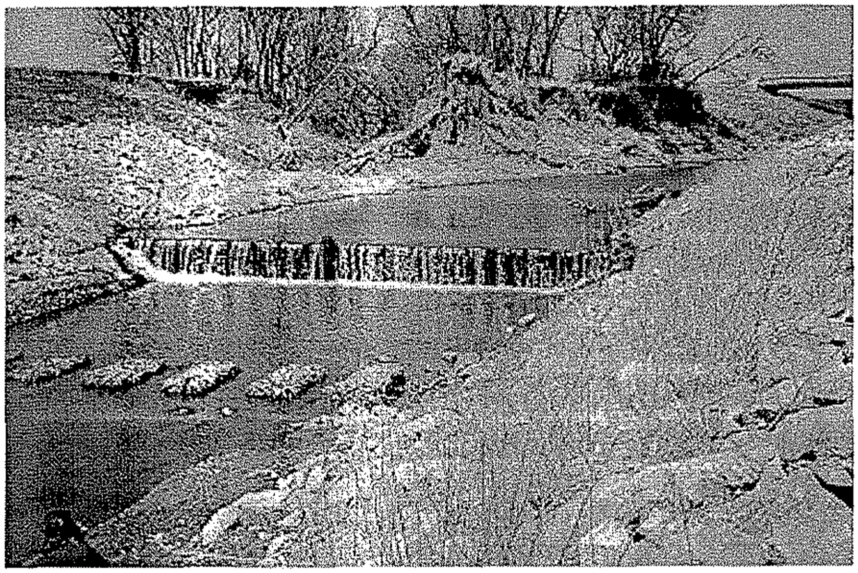
The H-pile grade stabilization structure requires riprap, placed inside of cribs, to act as the weir (similar to what is shown in Figure 53). The H-pile should be placed to a sufficient depth below the streambed to avoid failure of the structure due to erosion or headcutting. The H-pile should also be anchored into the streambanks to deter failure due to concentrated flows and high velocities along the outer edge of the structure.

H-pile Grade Stabilization Structure
Figure 53

Although this type of structure may be constructed to a height of 5 feet, multiple structures have been found to dissipate energy more efficiently through their stairstep configuration. It is recommended that riprap be ramped along the downstream face of the structure for fish migration.

Sheetpile Structure

The sheetpile grade stabilization structure may also be constructed to a height of 5 feet. As with the H-pile structure, the sheetpile should be placed to a sufficient depth below the streambed to avoid failure of the structure due to erosion or headcutting. The sheetpile should be keyed into the streambanks to deter failure from concentrated flows and high velocities along the outer edge of the structure. A sheetpile structure is shown in Figure 54.



Sheetpile Grade Stabilization Structure
Figure 54

Concrete Block Structure

This type of structure has been implemented in a large number of streams, as either a single row of blocks or stacked in a stairstep fashion, to stabilize degrading streams. The blocks, shown in Figure 55, can be formed and cast on site. Eyelets located in the top of the blocks can be used to tie the blocks together or to anchor the structure to the streambanks. The bedding that forms the foundation of such structures must be constructed to withstand the force exerted by the weight of the blocks.

Concrete Block Grade Stabilization Structure
Figure 55

Riprap Evaluation

Riprap has long been used to provide stability and erosion protection for channels and hydraulic structures. It is used extensively for streambed and streambank protection at grade control structures in western Iowa. Riprap performs well if little or no movement of the rock, significant to bed or bank stability occurs. Of the total number of structures evaluated, in-channel movement of riprap was exhibited in 72 percent of the grade control structures utilizing riprap, with mass movement of rock occurring at 12 percent of the structures.

An effective riprap design must consider the following factors: the quality of the rock, the shape of the stone or rock fragments, the weight or size of the individual pieces, and the gradation of the riprap material.

Quality of Riprap

There is no single standard specification to use in determining the quality of riprap. Numerous government agencies, such as the Bureau of Reclamation (BOR), the NRCS, and the Iowa DOT, have different laboratory testing guidelines and inspection procedures to determine the suitability of potential riprap material. Most specifications require the rock be tested for freeze-thaw durability and resistance to abrasion; however the test standards differ. For example, the standard freeze-thaw durability test used by the BOR, (BOR procedure 4666-90) requires that less than 25 percent of the total rock weight be lost at 250 cycles. The Iowa DOT requires that less than 10 percent of the total rock weight be lost at 50 cycles during freeze-thaw testing, using either AASHTO T-96-92 Method A or Method C.

The two standard tests are considerably different. Freeze-thaw tests conducted using BOR 4666-90 consists of 3 in rock cubes cut from rock fragments representative of the riprap material. The cubes are inserted into 3-inch rubber sheaths and water is added to completely submerge the specimen. The samples are subjected to 250 cycles of freezing and thawing or until 25% of the total rock weight is lost.

The Iowa DOT requires that the sample be prepared by breaking or crushing the rock into fragments reasonably uniform in size and shape, with each fragment weighing approximately 100 grams. The rock fragments are placed in a container, where the total test sample should weigh 5000 grams (± 2 percent). Method A requires that the sample be completely submerged, while Method C requires that the test sample be partially submerged. The sample is subjected to 50 cycles of freezing and thawing.

Both the BOR and the Iowa DOT require that the L.A. Abrasion test (ASTM C131.89) be performed. However, the BOR requires that less than 10 percent of the total rock weight be lost at 100 cycles, or less than 40 percent at 500 cycles. The Iowa DOT specifies that less than 50 percent at 500 cycles can be lost during abrasion testing.

Although various specifications exist, the general requirement for riprap material is that the material should consist of individual rock fragments that are dense, durable, and free of cracks, seams, structural planes of weakness, or other defects conducive to weathering. Weathering of the riprap material was evident at four of the stream stabilization structures evaluated for preparation of this manual. The weathered riprap at each site consisted of gray limestone. The weathering was along seams in the rock, with some of the stone weathering more severe than others. The riprap at these site was specified using IDOT standards. Bergeson states that theIDOT freeze-thaw durability test is not as accurate as the BOR test. Breaking or crushing of the sample into fragments occurs along natural planes of weakness, thus providing a test sample that is less prone to further degradation. The cubes prepared for the BOR procedure are more representative of the riprap material, as these are cut from the rock. Therefore, riprap that meets the Iowa DOT specifications may in fact be prone to accelerated weathering.

Also, at most quarries the rock material is determined fit for use long before the riprap order is placed. Proper inspection, whether on-site or at the quarry, and even possible retesting of the material should be performed to ensure the quality of the rock.

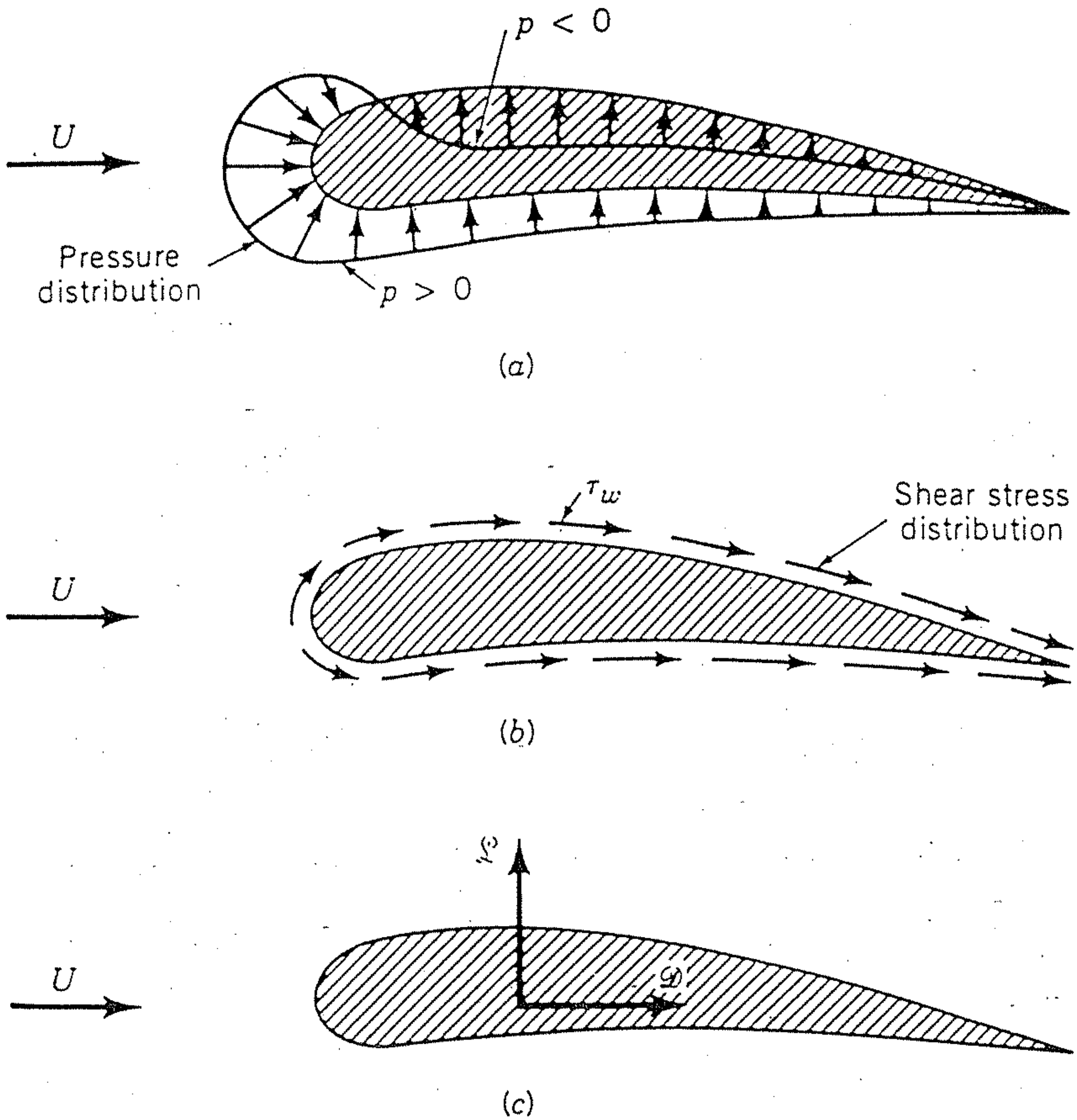
Shape of Riprap Material

With the relative motion of any object through a liquid, shear stresses (τ), due to viscous effects, and normal stresses (p), due to the pressure, occur on the surface of the object, as shown in Figure 56. A detailed distribution of the shear stress and normal stress over the surface of an object is difficult to obtain, either experimentally or theoretically. However, in many cases, only the integrated or resultant effects of these distributions are needed. The resultant force in the direction of the upstream velocity is termed the drag:

Drag = p cos0 dA + sin0 dA

with θ = angle created by the direction of the normal force with respect to the flow direction
A = cross-sectional area of the object. Lift is the resultant force normal to the upstream velocity or:

Lift = - p sin0 dA + cos0 dA



Forces on a Two-Dimensional Object from the Surrounding Fluid: (a) Pressure Force, (b) Viscous Force, (c) Resultant Force (Lift and Drag) (Munson et al., 1994)

Figure 56

To carry out the integrations and determine the lift and drag, θ, as a function of location along the object, must be known. With the extreme difficulty in obtaining the shear and normal force distribution, θ remains unknown.

The lift and drag forces arising from the relative motion of flat stones in the stream and the drag forces on elongated stones are greater in proportion to the stone mass than are forces on the more desirable angular or blocky shapes. No analytical method has been developed to determine optimum stone shape. The selection of stone shape is usually a function of subjective experience. Individual rocks that are flat should be rejected. Also, elongated rock fragments with a maximum dimension three times that of their minimum dimension should not be considered. Riprap should consist of rock fragments that are predominately angular to subrounded in shape. Angular or blocky rock fragments create good interlock between the individual rock fragments and are more resistant to movement.

While none of the major in-channel movement of riprap was a result of riprap shape, the displacement of the concrete barrier rails and slabs can be addressed with that argument. For low-cost grade control structures, the concrete barrier rails and slabs are essentially elongated riprap particles. Barrier rails are typically 2.67 feet in height, have a length of 10 feet, and relatively speaking, are flat. The concrete slabs are flat, with dimensions of 10 feet x 8.5 feet x 0.67 feet. Although it is impossible to calculate the lift and drag forces on a barrier rail without experimentation, using data collected from previous studies on plates, the forces exerted on the concrete slabs can be calculated.

The total drag (F_D) on the slab is the sum of the friction drag and the pressure drag. An equation has been formulated to calculate the total drag without detailed information concerning the shear and pressure distributions on the slab surface:

$\mathbf{F_D = C_d \rho \, (V^2/2) \, A}$, where

C_D = drag coefficient (1.8 for flat plates)
ρ = density of water (1.938 slugs/ft^3 at 60° F)
v = stream velocity
A = cross-sectional area of the slab (5.7 ft^2)

Concrete slabs were utilized for bank protection at only one structure evaluated during this study. The high-flow event that caused movement and displacement of the slabs occurred in July 1996.

From visual observations at this structure, the July flow rate was much higher than the previous estimated maximum flow rate of 1915 cfs that occurred in June of the same year. Unfortunately, the precipitation data for July was unavailable at the time of this study. However, using the design velocity of 18.1 feet per second calculated by the designers for a 50-year flood, the total drag force acting on the concrete slab was approximated to be 3260 pounds.

Although localized turbulent flow occurs along the surface of riprap, for simplicity it is assumed that this flow is laminar. The lift acting upon a flat plate is:

$\mathbf{F_L = C\boldsymbol{p}\ (V^2/2)\ A}$, where

C_L = lift coefficient
ρ = density of water (1.938 slugs/ft^3 at 60° F)
V = stream velocity
A = plane area of the slab (85 ft^2)

It is found that $C_L = 2\pi\sin\alpha$ according to Sabersky et al. (1989), where α is an arbitrary angle formed by the orientation of the plate to the upstream flow. With $\alpha = 0.22°$ (the bed slope angle), the lift on the slab exerted by the moving water is approximately 644 lbs. Therefore, the total force acting upon the concrete slab is 3904 pounds.

The weight of the concrete slabs is approximately 8543 lbs; however, when submerged, the buoyant weight of the slab is approximately 4989 pounds. Although the total force acting on the concrete slab was insufficient to move the slab at the design velocity, it must be remembered that this is a simplified case. The surface of the slab is not smooth; therefore an increase in the drag force is expected. Also, the displaced slabs were placed on the outside of a channel curvature, where the stream velocity is increased. With an observed July discharge that met or exceeded the design discharge of 8900 cubic feet per second, the lift and drag forces were sufficient enough to overcome the adjusted weight of the concrete slab and move the slabs either downstream or downslope.

Sizing of the Riprap

The size or weight of the riprap material is extremely important to its performance. The individual rock fragments that form the bank protection must be heavy enough to resist displacement by hydraulic forces. If the particle size is too small, erosion of the particles will occur, and the riprap may fail.

The majority of failures and the in-channel movement of the riprap on all the structures observed occurred in the same location, within the vicinity of the weir. The riprap within the vicinity of the weir is important to the stability of the structure. If the weir is compromised by the loss of the surrounding rock and soil, the structure could be a total loss. It is important then to place rock of sufficient size in the vicinity of the weir.

The theoretical weight or size of the rocks can be determined from several relationships. The majority of these design procedures are based on the relationship between the median size of the riprap material and either the flow rate or the stream velocity. The flow rate has already been determined at each structure. The average stream velocity was calculated along the rock ramp just downstream from the weir. The average velocity was calculated for the designed 1 vertical to 4 horizontal slope, although the constructed ramp slope may be steeper.

The Manning's equation was used to calculate the average velocity along the ramp, where:

$\mathbf{V = (1.486/n)\, R^{2/3}\, S^{1/2}}$, where

V = velocity
n = Manning's roughness coefficient (1.14 along ramp)
R = hydraulic radius
S = slope of the ramp

Eight procedures for the design of riprap-lined channels were evaluated. The first six of the eight procedures were discussed by Rice. A summary of the eight procedures follows, and a comparison of testing variables for sizing riprap is presented in Table 13.

Riprap Designs

<u>Ishbash Method.</u> Ishbash conducted a series of experiments to determine a relationship for the minimum velocity that will remove loose riprap. Ishbash obtained the data necessary to determine this relationship by depositing rounded riprap into flowing rivers. The objective of this study was to size the rock located on the downstream slope of a rockfill dam. The relationship he determined was:

$$V_{min} = y[2g(G_s - S)/S]^{1/2} D_{50}^{1/2}$$, where

V_{min} = minimum velocity (ft/s)
G_s = specific gravity of riprap
S = specific gravity of water (1.0)
g = acceleration due to gravity (32.2 ft/s)
y = Ishbash coefficient, 1.20 for maximum riprap stability
0.86 for minimum riprap stability

Simons and Senturk Method. Simons and Senturk modified the Ishbash equation to bring into effect streamflow on a sloping bed. Assuming that $y = 1.20$,

$$0.34V^2/[G_s - 1)gD_{50} = \cos\alpha$$

$\alpha = \tan^{-1} S_o$
S_o = the bed slope.

Abt and Johnson Method. Abt and Johnson developed their equation from laboratory tests performed on large flumes. Tests were performed using various shapes and sizes of riprap material. The slope of the bed channel was altered throughout the study, with slopes ranging from 10 to 20 percent. The expression they developed, which related the median riprap size to the bed slope and the discharge at failure, is:

$$D_{50} = 0.436 S_o^{0.43} q_f^{0.56}$$

q_f = unit discharge at riprap failure ($ft^3/s/ft$).

Unit discharge is the discharge divided by the wetted perimeter.

Abt and Johnson also determined a ratio between the onset of movement in the riprap, and riprap failure. Therefore, for design:

$$D_{50} = 0.436 S_o^{0.43} q_{design}^{0.56}$$

$q_{design} = 1.35\, q_f$

Robinson et al Method. Robinson et al. developed a relationship between the medium rock size, the streambed slope, and the unit discharge at failure. The relationship is:

$$\mathbf{D_{50} = 0.402S_o^{0.169}\ q_f^{0.546}}$$

Robinson developed this relationship by performing flume tests on angular riprap material with bedslopes ranging from 10 to 40 percent.

Normann Method. Normann developed the following relationship for the maximum depth of flow for channels lined with riprap:

$$\mathbf{d_{max} = 5D_{50}/[\gamma_w\ S_o]}$$

γ_w = unit weight of water and
d_{max} = maximum flow depth.

Olivier Method. Olivier presented the following relationship for the unit discharge at which riprap displacement begins:

$$\mathbf{q = 0.423\ [(\gamma_s - \gamma_w)/\gamma_w]^{5/3}\ D_{50}^{3/2}\ S_o^{-7/6}}$$

γ_s = unit weight of the riprap.

Olivier developed this relationship from laboratory experiments using short, narrow flumes and slopes ranging from 8 to 45 percent.

Table 13
Testing Variables for Sizing Riprap

Procedure	Nature of Rock	Slope	Nature of Experiment
Ishbash	rounded		natural rivers
Abt and Johnson	angular D_{50}<157.5 mm	10 and 20%	flume
Robinson et al.	angular D_{50} = 15-155 mm	10 - 40%	three flumes 0.76 - 1.83 m in width
Normann	rounded	< 2	channels
Olivier	angular D_{50}<60 mm	8 - 45%	flume 0.56 m wide x 1.52 m long

__Tractive Force - Water Resources.__ Another design procedure was derived from the maximum tractive force equation:

$$\tau = \gamma_w \, \mathbf{d_{max}} \, \mathbf{\sin S_o}.$$

The critical tractive stress is the tractive force that initiates movement of the riprap particles. For a given riprap size, the tractive force required to initiate movement is less for riprap placed on the side slopes of a trapezoidal channel than for riprap placed on the bottom of the channel. The critical tractive stress for riprap on the bottom of the channel is:

$$\tau_{bc} = C_{50} \, D_{50}$$

where C_{50} = coefficient relating critical tractive stress to riprap D_{50} size = 4.0.

Riprap placed on the sideslopes is subjected to the gravitational force, which tends to pull the riprap down the sideslope, in addition to the tractive stress caused by the flow. The critical tractive stress for riprap placed on the sideslope is:

$$\tau_{sc} = K \, C_{50} \, D_{50}$$

$K = (1-(\sin^2\theta/\sin^2\phi))^{0.5}$
θ = streambank angle
ϕ = angle of repose of the riprap.

<u>Tractive Force - International Erosion Control Association (IECA).</u> The IECA (1993) developed an equation to determine the median stone size using tractive force theory. It is used to size riprap based on the assumption of uniform, gradually varying flow. The equation assumes a stability factor, SF, of 1.2:

$$\mathbf{D_{50} = 0.001\ [V^3/d_{max}^{0.5}\ K^{1.5}]}$$

Two correction factors may be applied to the equation. These are for specific gravity and streambank stability. For specific gravities other than 2.65 use:

$\mathbf{C_{sg} = 2.12/(G_s - 1)^{1.5}}$ and for a SF differing from 1.2 use:

$\mathbf{C_{sf} = (SF/1.2)^{1.5}}$ where SF = stability factor as shown in Table 14.

Table 14
Guidelines for the Selection of Stability Factors (IECA, 1993)

Flow Condition	Stability Factor (SF)
Uniform flow; Straight or mildly curving reach (curve radius/channel width > 30); Impact from wave action and floating debris is minimal; Little or no uncertainty in design parameters.	1.0 - 1.2
Gradually varying flow; Moderate bend curvature (30 > curve radius/channel width < 10); Impact from waves or floating debris moderate.	1.3 - 1.6
Approaching rapidly varying flow; Sharp bend curvature (10 > curve radius/channel width); Significant impact potential from floating debris and/or ice; high-flow turbulence; Turbulently mixing flow at bridge abutments; Significant uncertainty in design parameters.	1.6 - 2.0

Comparison of Design Procedures

Tables 15 and 16 show the median weight values (W_{50}) predicted from the various design procedures. Fifty percent of the rock fragments used in the riprap layer must have weights greater than the calculated median weight, and no more than 50 percent of the riprap material can weigh less than the median weight. Table 15 shows the median weight value determined using the design flow rates, and Table 16 shows those determined using the estimated maximum flow rate. The median rock weight determined by each procedure varies, but except for the Normann and Water Resources procedures, they have the same magnitude. The Normann procedure predicted the unreasonably large riprap. As Table 16 shows, this procedure was developed using rounded riprap in channels with small slopes. The slope of the riprap ramp beyond the weir is at least 25 percent, therefore, this procedure is not applicable.

The Olivier procedure also predicts riprap sizes that appear to be unreasonably large; however, this procedure does predict reasonable sizes for small slopes. The reason for this is unknown, but it may be due to the small riprap sizes and the short flume lengths used in his study.

The procedures derived from the tractive force equation also appear to predict riprap sizes that are unreasonable. The explanation for this is unknown, but it may be due to the steep slope and high velocities along the riprap ramp.

Table 16 shows that the procedures developed by Ishbash, Simon and Senturk, Abt and Johnson, and Robinson et al. give similar results. However, Rice conducted three-dimensional field tests on rock chutes using angular riprap with a D_{50} of 188 mm on a slope of 16.7 percent slope, and a D_{50} of 277 mm on a 33.3 percent slope. Each field-scale chute had a drop of 3.66 m, a 2.74 m bottom width, and 2 horizontal to 1 vertical sideslopes. From the results of field-scale tests, Rice recommends that the Abt and Johnson and Robinson et al procedures be used for design. Rice contends that the riprap sizes predicted with the Ishbash procedure, as well as with the Simon and Senturk procedure, are affected by the uncertainty of estimating the Manning roughness coefficient when calculating velocity.

Table 15

Predicted Median Weight for Riprap Using Design Discharge

			W_{50} (lbs)								
				Simons &	Abt &	Robinson			Tractive Force	Tractive Force WRP	
County	Stream	Q_{design} (cfs)	Ishbash	Senturk	Johnson	et. al.	Normann	Olivier	IECA	Channel Bottom	Channel Side
Woodbury	Big Whisky Creek	Not available									
	Elliot Creek	Not available									
		Not available									
		Not available									
	McElhaney Creek	Not available									
	West Wolf Creek	Not available									
		Not available									
		Not available									
	East Fork Wolf Creek	6475	*	*	*	*	*	*	*	*	*
Monona	Jordan Creek	5615	$	$	$	$	$	$	$	$	$
		8900	8060	8811	8297	9531	2296977	69738	54352	4347970	9304898
		9575	6429	7028	9006	10323	2254498	76885	50930	4267561	
		10475	40475	44243	12609	14333	4818862	114780	550248	9121672	
Crawford	Middle Soldier River	3600	1928	2107	3480	4085	896342	24791	^	1696695	^
		3000	1171	1280	2764	3264	665479	18848	7288	1259693	3216115
		5200	1443	1578	4163	4865	865956	30685	8741	1639177	4184974
		Not Available									
	Paradise Creek Trib	Not Available									
	E. Soldier River Trib	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
Shelby	Mosquito Creek	10000	14655	16019	10795	12318	3296579	95398	144944	6240127	
	Elk Creek	3694	1291	1411	5092	5921	1072381	39000	^	2029920	^
	Long Branch Creek	3584	5376	5877	4005	4685	1342864	29300	50462	2541921	6489766
Pottawattamie	Walnut Creek	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
	Graybill Creek	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
Cass	Crooked Creek	6175	1586	1734	5109	5941	1000161	39161	15497	1893215	6759018
	Troublesome Creek	12575	5655	6182	10426	11907	2297097	91524	41628	4348199	
Taylor	102 River	5802									
		5429	13278	14515	6313	7302	2287639	50378	150069	4330295	
		ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
Page	Snake Creek Trib	3851	$	$	$	$	$	$	$	$	$
Burt, NE	Elm Creek	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND

* = No riprap $ = Vertical drop spillway ND = No design plans ^ = Complex number

Table 16

Predicted Median Weight for Riprap Using Estimated Maximum Discharge

			W_{50} (ft)								
				Simons &	Abt &	Robinson			Tractive Force	Tractive Force WRP	
County	Stream	Max Q (cfs)	Ishbash	Senturk	Johnson	et. al.	Normann	Olivier	IECA	Channel Bottom	Channel Side
Woodbury	Big Whisky Creek	3577	*	*	*	*	*	*	*	*	*
	Elliot Creek	4306	5146	5625	2300	2728	10969446	13961	52988	737	1883
		5163	8505	9297	2868	3382	13567332	18150	74577	1051	2684
		6022	6485	7089	3274	3849	18471805	21263	114645	902	2300
	McElhaney Creek	1331	514	562	930	1129	1027819	4754	6608	5103	13024
	West Wolf Creek	3291	11023	12049	2499	2960	5017870	15422	20679	7423	18951
		3361	7530	8231	2477	2935	5572750	15264	23421	5815	14842
		3584	8422	5927	2589	3064	6482896	16084	29053	4996	12759
	East Fork Wolf Creek	5348	*	*	*	*	*	*	*	*	*
Monona	Jordan Creek	1709	$	$	$	$	$	$	$	$	$
		1915	147	161	789	960	717000	3905	1327	7742	14554
		2041	125	136	878	1067	633703	4434	320	10756	27463
		2215	798	872	1336	1606	1529498	7309	5161	5943	15172
Crawford	Middle Soldier River	5699	4630	5061	7281	8389	8283977	55051	^	14251	^
		8702	23518	25707	7241	8346	9689196	54704	83490	50134	127980
		15541	45253	49465	12401	14101	19637919	103791	216703	62196	158773
		16527	$	$	$	$	$	$	$	$	$
	Paradise Creek Trib	2558	1348	1474	1147	1384	3521889	6102	4599	1244	2220
	E. Soldier River Trib	3188	ND	ND	ND	ND	ND	ND	ND	ND	ND
Shelby	Mosquito Creek	12300	25061	27394	8653	9928	19168775	67586	166389	18643	47601
	Elk Creek	4408	1911	2088	4219	4926	10136108	28741	^	1265	^
	Long Branch Creek	2453	1688	1845	1590	1903	3115916	8999	15089	3041	7772
Pottawattamie	Walnut Creek	4256	ND	ND	ND	ND	ND	ND	ND	ND	ND
	Graybill Creek	4550	ND	ND	ND	ND	ND	ND	ND	ND	ND
Cass	Crooked Creek	1404	38	42	505	622	331364	2295	210	6816	24332
	Troublesome Creek	6068	813	889	2364	2802	7489409	14429	46621	1045	2667
Taylor	102 River	4984	3617	3954	2635	3115	10653980	16421	51976	1053	2687
		4308	9803	6280	2508	2969	9834834	15478	47207	1356	3464
		17497	ND	ND	ND	ND	ND	ND	ND	ND	ND
Page	Snake Creek Trib	892	$	$	$	$	$	$	$	$	$
Burt, NE	Elm Creek	2909	ND	ND	ND	ND	ND	ND	ND	ND	ND

* = No riprap $ = Vertical drop spillway ND = No design plans ^ =Complex number

Riprap Design Recommendations

Although there is a disparity in the results of these procedures, the median weight of the riprap material predicted by all the procedures is greater than the median weight recommended for the Iowa grade control structures by the various designers. Table 17 shows the design W_{50} sizes for western Iowa.

The majority of the structures observed and evaluated have a field median weight below that of the design W_{50}. A large percentage of structures exhibited riprap movement at the weir, lending credibility to the argument that the riprap is undersized.

Table 17
Design Median Weight

Design Agency	W_{50}
Iowa Department of Transportation	90
Consultant 1	50
Consultant 2	400

The predicted riprap size may be undersized because the velocity was underestimated due to inadequate allowance for channel curvature, inadequate allowance for the effect of obstructions, or ramp slope. Design calculations for riprap sizing were included in some of the grade control structure design plans. The riprap for these structures was designed using the stream velocity. What was immediately apparent was that the velocity used to predict the riprap sizing was the velocity of the unaltered stream channel. In some instances, the unaltered stream velocity was less than half the velocity calculated at the weir. Also, the design calculations did not take into effect the increased velocity on the outer bank of channel curvatures. The same riprap dimensions were predicted for riprap placed in a channel with straight alignment and for riprap placed within curves. In addition, the detrimental effect of obstructions was nottaken into effect. Two structures had major erosion of riprap material due to large trees becoming entangled within the bridge piers and diverting the flow onto the streambanks.

Hadish and Braster have noted that riprap is becoming a scarce and expensive commodity in western Iowa, and that quarries are having extreme difficulty in producing riprap over 1500 lbs. Grouting of the riprap in the vicinity of the weir is a viable solution. Grouting cements the rock particles together, essentially creates larger stone sizes. The NRCS requires that grout be applied to riprap with velocities exceeding 7.5 ft/s in curved channels and 10 ft/s in channels with straight alignment. Grouting can be a costly expense if riprap sizes produced continue to shrink. Therefore, alternative designs to future channel protection should be explored as suitable riprap disappears.

Riprap Gradation

Although the weight of the individual rock fragments is important to the stability of the riprap, a riprap mixture cannot be defined by a single factor. Previous testing has shown that no single particle fraction size is characteristic of the entire riprap mixture. Two riprap mixtures, a poorly graded and a well-graded mixture, with the same median rock size exhibit differing degrees of stability.

Two different schools of thought exist on riprap gradation. In the past, well-graded riprap was believed to be more stable than poorly graded. A riprap mixture in which most of the rock particles are the same size is poorly graded. Most governmental agencies specify only that the riprap must be well-graded, leaving the actual gradation to the designer. However, recent testing by Abt and Wittler has shown that poorly graded riprap, all other factors being equal, withstands substantially larger flows than well-graded riprap. Poorly graded riprap can withstand flows approximately 1.5 times greater than flows that caused failure of the well-graded material.

Flume testing was performed on both well-graded and poorly graded riprap. The tests showed that well-graded riprap fails over a period of time. Well-graded riprap tends to fill in voids left by eroded particles with riprap particles eroded from upstream or upslope. As the stream velocity decreases, smaller rock particles settle within the voids vacated by larger rock sizes. The term for this void filling process is termed healing. The size distribution of the riprap changes with the loss of rock particles, essentially becoming weaker over time as more riprap material is lost.

Poorly graded riprap fails more suddenly. With the majority of the rock having the same particle size, numerous rocks become mobile at once. Little healing occurs since most of the riprap particles have been eroded. Wittler concludes that the designer should consider the ramifications of gradation specifications both in design and quality control during construction. Although failure of a well-graded riprap occurs at a lower discharge than a poorly graded riprap, the catastrophic failure of a poorly graded riprap is severe, with little riprap material remaining on the sideslopes. It is for this reason that many agencies still require well-graded material.

Riprap Performance at Grade Stabilization Structures

The performance rating of the riprap was separated from the performance number determined for each structure in order to determine if any correlations could be established, linking the stability or instability of the riprap and the design specifications.

In Table 18, the riprap performance rating determined for each structure and the corresponding discharge ratios (Q_E/Q_D) are presented. The discharge ratio is the ratio of the estimated discharge, Q_E, versus the design discharge, Q_D used to normalize the discharge at the different sites. A discharge ratio ≥ 1 acknowledges that the maximum discharge has exceeded the design discharge.

The performance of riprap is dependent on numerous variables besides velocity or stream flow. These variables include sizing, gradation, the slope of the streambanks or channel bed, the thickness of the riprap layer, the stability and effectiveness of the filter on which the riprap is placed, and construction techniques. One particular design aspect cannot predict the performance of the riprap material, and proper inspection is necessary in all aspects of riprap production and construction to ensure high performance.

Gradation Evaluation

A design deviation that occurred at the majority of the 31 grade stabilization structures evaluated during this study was the riprap gradation. The observed rock sizes at most structures was smaller than the design sizes. Therefore, very few of the structures had gradations that met or were close to the gradation specifications. The uniformity coefficient (C_u) was calculated for each structure with known design gradations, excluding those structures with grouted riprap, or incorporated concrete blocks for channel stability. No correlation could be determined between riprap performance at the structures evaluated and the uniformity coefficient. This lends credibility to Abt's and Wittler's conclusion that poorly graded riprap can withstand higher flow rates than well-graded riprap. However, the results are inconclusive with only one structure evaluated having verified their testing.

Table 18
Riprap Performance Rating

Structure Number	County	Stream	Riprap Performance Number	(Q_E/Q_D)	Peak Velocity (ft/s)
1	Woodbury	Big Whiskey Creek	NA	NA	NA
2		Elliot Creek	22	NA	22.9
3			26	NA	24.9
4			22	NA	23.8
5		McElhaney Creek	44	NA	15.6
6		West Wolf Creek	48	NA	26.0
7			56	NA	24.4
8			41	NA	23.1
9		East Fork Wolf Creek	NA	0.83	NA
10	Monona	Jordan Creek	22	0.17	NA
11			72	0.22	N
12			83	0.21	N
13			22	0.21	N
14	Crawford	Middle Soldier River	89	1.58	N
15			72	2.90	29.5
16			41	2.99	32.9
17			39	NA	NA
18		Paradise Creek Trib	78	NA	20
19		East Soldier River Tri	67	NA	NA
20	Shelby	Mosquito Creek	NA	1.23	NA
21		Elk Creek	67	1.19	19.6
22		Long Branch Creek	67	0.69	19.2
23	Pottowattamie	Walnut Creek	33	NA	NA
24		Graybill Creek	NA	NA	NA
25	Cass	Crooked Creek	33	0.23	10.2
26		Troublesome Creek	37	0.48	17.0
27	Taylor	102 River	41	0.86	21.8
28			44	0.79	23.9
29			72	NA	NA
30	Page	Snake Creek Trib	22	0.16	NA
31	Burt, NE	Elm Creek	44	NA	NA
NA - not available		N - not applicable			

Factors of Influence on Riprap Performance

An effective riprap design must consider numerous variables, including size, gradation, discharge or velocity, rock shape, the slope of the streambank or channel bed, the thickness of the riprap layer, the quality of the rock, and the stability and effectiveness of the filter.

For western Iowa, it was possible to isolate some of the structures based on the riprap design variables. The majority of the low-cost structures evaluated use engineering fabric for the filter. Those structures with filters constructed of sand or gravel were excluded, as were grade control structures with a riprap layer thickness differing from approximately 2 feet in thickness. Riprap stability is affected by the slope of the channel banks. Riprap should not be placed on channel banks with slopes steeper than 2 horizontal to 1 vertical due to the possibility of failure as the angle of repose for the riprap may be exceeded. Therefore, structures with sideslopes other than 2 horizontal to 1 vertical were eliminated. The structures with riprap exhibiting accelerated weathering were also eliminated. Thirteen grade stabilization structures evaluated had riprap design variables remaining of stream velocity, median rock size, and the riprap gradation. A dimensionless number was developed utilizing these three design variables. This number, known as the erosion control ratio, is:

Erosion Control Ratio = $[D_{85} / D_{30}]\, D_{50} / [V^2/2g]$

D_{85}/D_{30} = the gradation
D_{50} = the median rock size
$V^2/2g$ = the velocity head.

A correlation exists between the erosion control ratio and the riprap performance for the structures evaluated. As the performance of the riprap increases, the erosion control ratio also increases. The correlation further verifies that riprap performance is related to all the variable design factors. A failure to adequately design for even one of these variables could have a detrimental effect on the riprap performance and could lead to a possible failure of the weir itself.

Construction of the Riprap Ramp

Construction is as important to riprap performance as is the design. Improper construction techniques, misplacement of critical stone, or failure to follow the design plans can lead to riprap erosion and failure. Regulations for Iowa require that a ramp with a 4:1 slope be constructed starting immediately downstream from the weir. Although the regulations do not specify the type of material that can be used in the ramp construction, the majority of grade control structures in western Iowa use riprap.

At many of the structures evaluated, the original slope of the completed ramp was steeper than the required 4 horizontal to 1 vertical slope. This was observed at sites throughout western Iowa, and verified by observing the construction of a new grade control structure. The weir, ramp and channel bed riprap were completed, and construction of the streambanks was underway, with riprap being placed on the sideslopes. Although measurements were not taken, it was apparent that the finished ramp was steeper than a 4 horizontal to 1 vertical slope. This deviation creates a situation in which the riprap is underdesigned and increases the probability for erosion and failure of not only the streambanks but also of the weir as well.

Construction of a grade control structure typically begins with the weir. With the weir completed and the streambanks shaped, riprap is placed on the channel bed and the rock ramp constructed. Riprap material is placed along the sideslopes after the ramp and channel bed are finished. Thus, the ramp and the riprap placed on the channel bed are an important component of the sideslope stability. The ramp or the riprap on the channel bed forms the toe of the sideslope. If erosion of the ramp or channel bed occurs, failure of the streambank riprap or the underlying material may occur.

Placement of the riprap is critical with ramp construction. The rock should be arranged such that there is a good distribution of larger pieces on the surface to anchor and support the other sizes. If the design or constructed riprap is undersized, the force of the stream will cause erosion of the riprap ramp and potentially produce a stability problem for the weir. Therefore, the velocity of the stream is of great importance.

Riprap size and/or gradation should be designed with the stream velocity determined for a 4 horizontal to 1 vertical slope. If the ramp is constructed with a steeper slope, there is an increase in the stream velocity. If the design velocity is exceeded, the riprap particles will be eroded and transported downstream. The ramp slope will increase as riprap is removed, in turn increasing the velocity and erosive power of the stream. The ramp slope will continue to steepen as the design velocity is exceeded, until eventually a vertical drop weir is formed. Once the drop becomes vertical, the mechanics of flow over the weir is altered. The stream will be in free fall, where the velocity will be a maximum under all flow conditions. The stream impacts directly on the streambed, causing a scour hole to form in the vicinity from the weir. As the scour hole deepens, the toe of the sideslope is undermined, and failure of the streambanks occurs. This mechanism of failure corresponds with the field observations. The area of most riprap displacement or failure took place just downstream of the weir. As the steepness of the ramp increases, so does the riprap performance rating. The structures with the poorest riprap performances were those in which the ramp of the structure had been completely eroded and a vertical drop formed.

Chapter 7
Grade Stabilization Monitoring and Evaluation

A large-scale evaluation of grade stabilization structures in western Iowa had never been conducted prior to this study. A methodology for evaluating each structure was formulated, and information crucial for proper evaluation of each structure was identified and collected. The first task was to identify the design attributes relevant to a structure's performance.

Although grade control structures vary widely in design, each structure has common design features of importance. These features include the overall height of drop of the structure, the slope angle of the constructed streambanks, the gradation of the riprap material and the condition of the riprap. Another point of importance is the distance upstream from the structure where the backwater effect ends. This distance provides an estimation of the amount of protection the structure provides.

A grade stabilization structure performance evaluation form was designed to quantify the field performance of each structure. The form is also used to aid in quickly measuring the design attributes of interest. A set of performance criteria was developed from previous observations of grade stabilization structures to rank each individual structure. The performance evaluation form and performance criteria are shown in Figures 57a and 57b. Each criterion suggests the different elements that can affect the field performance of a structure. For example, the stability of the upstream streambanks and streambed suggest the structure is providing the necessary grade control, while the possible instability of the downstream streambanks and streambed can infer that a headcut is moving upstream toward the structure. As with all the criteria, each is of equal importance, and each is significant to the performance of the grade stabilization structure.

A numerical ranking from 1 to 9 is applied to each evaluation criterion. The numerical ranking given for each criterion is subjective, but by following the descriptions presented in Figure 57b, comparable scores between two or more inspectors may be produced. The overall grade stabilization performance number is calculated by the following formula:

Performance Number (Pn) = [cumulative points/(number of applicable criteria x 9)] x100

The final percentage determines the overall condition of the structure. A structure with a performance number of 1 to 33% is rated in good condition; 34 to 67%, in average condition; and a performance number of 68% or greater, in poor condition.

Grade Stabilization Performance Evaluation Chart

Structure Description:______________________________________ Date of Evaluation: ________

____

Stream:______________________________ Name of Contact: ______________
County:______________________________ Telephone: ______________
Location: T_____N R_____W SEC _____
Year Constructed: ______________________
Construction Cost: $______________________ Design Discharge: _____ cfs
Average Annual Maintenance Costs: $______________ Frequency: _________ -year

Maximum Estimated Discharge since Construction: ________ cfs
Date of when Maximum Discharge Occurred: ____________________
Estimated Maintenance Costs Associated with Maximum Discharge Event: $______________

Vertical Distance of Upstream Side of Bridge Deck to Channel Invert: _____ feet
Overall Height of Drop: _____ feet
Distance of Grade Control Structure from Infrastructure: _____ feet
Distance from Structure to Upstream end of Backwater Affect: _____ feet
Average Diameter of Riprap Material: _____ inches
Condition of Riprap Material (circle all that apply): no problems cracking spalling dissolving disintergrating
Streambed Material (circle all appropriate): clays silts sands gravels

Performance Evaluation	good	average	poor	n/a
Apply Numerical Ranking to the Following Evaluation Criteria	1-3	4-6	7-9	X
Stability of upstream streambanks				
Stability of downstream streambanks				
Stability of upstream channel invert				
Stability of downstream channel invert				
Impact of structure on protecting infrastructure				
Structural integrity of grade control structure				
Structural integrity of upstream riprap				
Structural integrity of downstream riprap				
Condition of stilling basin or scour hole				
Performance Evaluation descriptions provided on back of this sheet				

Identify the Percentage of Total from the above Evaluation Criteria	1-33%	34-67%	68-100%
Overall condition of grade control structure			

Notes (observations, maintenance requirements, etc.): ______________________________

____

____

____

____

Figure 57a

Performance Evaluation Descriptions

Stability of upstream streambanks-
1-3 good - appear stable with over 60% vegetative cover, no noticeable erosion
4-6 average - limited erosion along toe, 30-60% vegetative cover, minor streambank sloughing occurring
7-9 poor - significant indication of erosion along toe, 0-30% vegetative cover, active streambank sloughing
n/a - not applicable

Stability of downstream streambanks -
1-3 good - appear stable with over 60% vegetative cover, no noticeable erosion
4-6 average - limited erosion along toe, 30-60% vegetative cover, minor streambank sloughing occurring
7-9 poor - significant indication of erosion along toe, 0-30% vegetative cover, active streambank sloughing
n/a - not applicable

Stability of upstream channel invert -
1-3 good - no indication of erosion, scouring, or headcutting taking place
4-6 average - minor erosion along toe of banks, or ripples and small falls indicating minor headcutting
7-9 poor - indication of active erosion along toe of banks, and/or substantial headcutting is occurring
n/a - not applicable

Stability of downstream channel invert -
1-3 good - no indication of erosion, scouring, or headcutting taking place
4-6 average - minor erosion along toe of banks, or ripples and small falls indicating minor headcutting
7-9 poor - indication of active erosion along toe of banks, and/or substantial headcutting is occurring
n/a - not applicable

Impact of structure on protecting infrastructure -
1-3 good - all piers and abutments appear to be stable and erosion and bank widening is not noticeable
4-6 average - indication of minor erosion occurring in vicinity of piers or abutments
7-9 poor - indication of substantial erosion occurring in vicinity of piers and abutments
n/a - not applicable

Structural integrity of grade control structure -
1-3 good - all riprap appears to be stable and secure, sheetpile appears stable, and all concrete and grout appears intact
4-6 average - indication of minor displacement of riprap, sheetpile being flanked, minor cracks in concrete and grout
7-9 poor - substantial displacement of stone, flows flanking sheetpile, failure of concrete and grout sections
n/a - not applicable

Structural integrity of upstream riprap -
1-3 good - all riprap appears to be stable and well placed with no sign of cracking, spalling, or disintegration
4-6 average - minor displacement of riprap in several areas, indications of cracking, spalling, or disintegration
7-9 poor - significant displacement of riprap, severe cracking, spalling, and/or disintegration occurring
n/a - not applicable

Structural integrity of downstream riprap -
1-3 good - all riprap appears to be stable and well placed with no sign of cracking, spalling or disintegration
4-6 average - minor displacement of riprap in several areas, indications of cracking, spalling or disintegration
7-9 poor - significant displacement of riprap, severe cracking, spalling, and/or disintegration occurring
n/a - not applicable

Condition of stilling basin or scour hole -
1-3 good - no sloughing, erosion, or debris blockage of stilling basin, or widening or lengthening of scour hole occurring
4-6 average - minor sloughing, erosion, or debris blockage of basin, or minor widening and lengthening of scour hole
7-9 poor - significant sloughing, erosion, or debris blockage, or significant widening and lengthening of scour hole
n/a - not applicable

Overall condition of grade control structure -
1-33% good - structure appears to be stable and functioning as designed
34-67% average - minor damage to structure identified, requires minimal maintenance to repair
68-100% poor - significant operational problems occurring, requires extensive remedial measures to prevent failure

Figure 57b

Performance Evaluation Techniques

A surveyor's hand level and a Philadelphia measuring rod were used to measure and calculate the overall drop of each structure. The slope of the streambanks was determined using a pocket transit. A 100-foot measuring tape was used to determine short distances, and longer distances were estimated by pacing.

A grid was used to measure the gradation of the riprap at each structure. The grid was constructed of 6-inch blocks created on a 2-foot x 4-foot sheet of thin Plexiglas. The various sizes of the riprap material are segmented into five categories: less than 6 inches, 6 to12 inches, 12 to18 inches, 18 to24 inches, and greater than 24 inches. The grid is placed directly on the constructed riprap in various locations on both streambanks, mainly within the vicinity of the control weir(s), where bank protection is most crucial. The amount of riprap material in each category was counted, and divided by the total amount of riprap beneath the grid. The final field gradation was calculated by taking an average of the percentages computed at each location.

The weight range of the riprap falling into each size category was determined by the following formula developed by Mark Looschen of the IDOT:

Riprap Weight = $(0.762) \times (G_s) \times (\gamma_W) \times (\text{riprap size})^3$

G_s = the specific gravity of the stone
γ_W = the unit weight of water (62.4 lbs/ft^3)

The specific gravity of various riprap materials found within the western Iowa study area was determined in the laboratory from numerous samples collected in the field and is presented in Table 58.

Table 19

Weight and Specific Gravity of Western Iowa Riprap

Riprap Size (ft)	Riprap type			
	Northern Sioux Quartzite	Southern Grey Limestone	Grey Limestone	Tan Limestone
	Specific Gravity (Gs)			
	2.64	2.6	2.68	2.65
	Weight of Riprap (lb)			
0.5	15.7	15.5	15.9	15.8
1.0	125.5	123.6	127.4	126.0
1.5	423.7	417.2	430.1	425.3
2.0	1004.2	989.0	1019.5	1008.0

Structure Evaluations

The summer of 1996 produced two large streamflow events in western Iowa. These above-average precipitation events occurred within a 2-month period spanning from mid-May until mid-July. The majority of the grade stabilization structures in western Iowa were constructed with Federal assistance after 1993. The two high-flow events of 1996 came close to or exceeded the design discharge of these new structures. In the northern region of the study area, the two 1996 events rivaled those of the 1993 floods. While these floods were unfortunate for the counties affected, they provided a unique opportunity for data collection and evaluation during this study.

Summary of the Structural Concerns

Every grade stabilization structure either evaluated using the grade stabilization performance evaluationform or observed exhibited some type of structural concern, ranging from minor and of no real consequence or to that which requires immediate attention. Table 59 provides a summary of the concerns that were detected at 43 grade stabilization sites. The table also includes the percent of those structures affected by those particular concerns.

Table 20
Summary of Concerns Resulting from Grade Stabilization Evaluation

Type of Concern	Number of Structures	Percent of Structures Affected
Erosion downstream of stilling basin	26	61
Erosion around weirs	3	7
Erosion under grouted riprap	3	7
Displacement of in-channel riprap, barrier rails, and concrete blocks	29	67
Displacement of engineering fabric	11	26
Upstream sideslope instability	4	9
Downstream sideslope instability	14	33
Mass movement of sideslope riprap	5	12
Settlement of concrete blocks	2	5
Seepage under concrete blocks and through grouted riprap	4	9
Cracking of grouted riprap	2	5

Erosion beyond the stilling basin and movement of the riprap, barrier rails, and concrete blocks are the most common concerns exhibited by the low-cost grade stabilization structures. Downstream slope instability and displacement of engineering fabric are also common concerns, but these concerns are associated with the loss of riprap and the erosion that occurs downstream beyond the limits of the streambank protection.

Chapter 8
Permit Requirements

When considering the construction of structures and/or streambank protection measures to control streambed degradation or streambank widening, the following State and Federal general permit requirements should be complied with. Through direct contact with the office of concern, specific permit requirements may be obtained, alleviating delays in construction schedules.

Archeological and Historical Preservation Act, as amended, 16 U.S.C. 469, et seq. The State Historic Preservation Officer (SHPO) should be notified of the location and design of the proposed project. A field reconnaissance of the project location should be conducted by a professional archeologist.

State Historical Society of Iowa
Capitol Complex
East 6th & Locust Street
Des Moines, Ia 50319

Flood Plain Management. Executive Order 11988 states that all actions located within a flood plain shall be undertaken so as to avoid adverse impacts associated with human safety, health, and welfare. The State flood plain management office should be notified of proposed construction occurring in the flood plain to determine impacts to adjoining lands.

U.S. Army Corps of Engineers
Hydraulics and Flood Plain Management Services
215 North 17th Street
Omaha, NE 68102
(402) 221-4596

Iowa Department of Natural Resources
Water Resources Section
Wallace Building
East 9th & Grand Avenue
Des Moines, IA 50319-0034
(515) 281-5145

Clean Water Act, as amended, 33 U.S.C. 1251, et seq. The COE regulatory office should be informed of construction activities that may involve work in a waterway or wetland of the United States. Many streambank protection projects qualify for Nationwide 13 permits. If the proposed project does not qualify for a Nationwide 13 permit (determined by the COE regulatory office), then a 404(b)1 evaluation must be completed.

For Counties Adjacent to Missouri River:
U.S. Army Corps of Engineers
Salt Creek/Papio Office
Omaha, NE, 68138-3621
(402) 221-4133

For Counties Not Adjacent to Missouri River:
U.S. Army Corps of Engineers
Clock Tower Building
Rock Island, IL 61201-2004
(309) 788-6361 x-6370

Endangered Species Act, as amended, 16 U.S.C. 1531, et seq. The U.S. Fish and Wildlife Service (FWS) and the State agency responsible for fish and wildlife resources must be notified and provided a brief description of the proposed project. The FWS will provide a list of threatened and endangered (T&E) species that might be present in the project area. A biological assessment may be required if there are significant T&E resources in the project area.

U.S. Fish and Wildlife Service
4469 48th Avenue Court
Rock Island, IL 61201

Clean Air Act, as amended, 33 U.S.C. 1857h-7, et seq. The Environmental Protection Agency (EPA) should be notified and furnished a brief description of the proposed project.

Environmental Protection Agency, Region VII
726 Minnesota Avenue
Kansas City, KS 66101
(913) 551-7006

National Pollution Discharge Elimination System (NPDES). The NPDES permits are required from the State Environmental Protection Division for land disturbances greater than 5 acres. Each State has its own policy, and some States require 90 days' notification before initiating construction.

Environmental Protection Division
Wallace Building
East 9th & Grand Avenue
Des Moines, IA 50319-0034

Protection of Wetlands. Executive Order 11990 requires that National Wetland Inventory (NWI) maps be reviewed to determine if any wetlands will be affected by proposed construction. Mitigation is required for significant wetlands impacts.

Environmental Protection Agency, Region VII
726 Minnesota Avenue
Kansas City, KS 66101

Watershed Protection and Flood Prevention Act, 16 U.S.C. 1101, et seq. This statute requires coordination with the NRCS. Soil conservation measures should be incorporated into the design of the proposed project.

Iowa Natural Resources Conservation Service
63 Federal Building
210 Walnut Street
Des Moines, Iowa 50309

Chapter 9
Additional Planning and Design Resources

For additional information regarding planning, design, and possible funding for grade stabilization projects, please contact the following entities.

Golden Hills Resource Conservation and Development, Inc.
RR#2, Box 237
Oakland, Iowa 51560
Contact: Shirley Frederiksen
Pam Neenan
712-482-3029

Iowa State University
Dept. of Civil & Construction Engineering
Ames, Iowa 50011
Contact: Dr. Robert Lohnes
Dr. Ruochuan Gu
515-294-2140

US Army Corps of Engineers - Omaha District
Technical Planning and Engineering Services
215 North 17th Street
Omaha, Nebraska 68102
Provides technical information on projects impacting public facilities.
Contact: 402-221-4596

Iowa State University Forestry Extension Service
251 Bessey Hall
Ames, Iowa 50011
Provides information and publications on reforestation, weed control, buffer strip design, etc.
Contact: Dr. Paul Wray
515-294-1168
515-294-2995 (FAX)
web site: http://www.ag.iastate.edu/departments/forestry/ext.html

Iowa Buffer Initiative

ISU Department of Forestry

A cooperative agreement between Iowa State University, Iowa Department of Natural Resources, Trees Forever, US Environmental Protection Agency, NRCS, Farm Bureau, and Novartis Crop Protection. The project will assist 100 demonstration projects with planning, design, and management of forestry areas.

Contact: Dr. Tom Isenhart
515-294-0856

Trees Forever

Assists in the planning, design, and management of reforestation projects. Also a key partner in the Iowa Buffer Initiative.

Contact: Shannon Ramsey
Executive Director, Trees Forever
319-373-0650
Del Christensen, Field Coordinator, Trees Forever
515-993-3422

For fertilizer and seed mixture recommendations contact the following:

NRCS District Offices

Iowa Dept. of Natural Resources, Forestry Division

2402 S. Duff
Ames, Iowa 50010
Contact: 800-865-2477
515-233-1131 (FAX)

Iowa Prairie Network

P.O. Box 261
Cedar Falls, IA 50613
Assists in planning, design and management of prairies.
Contact: Carole Kern
319-276-3082

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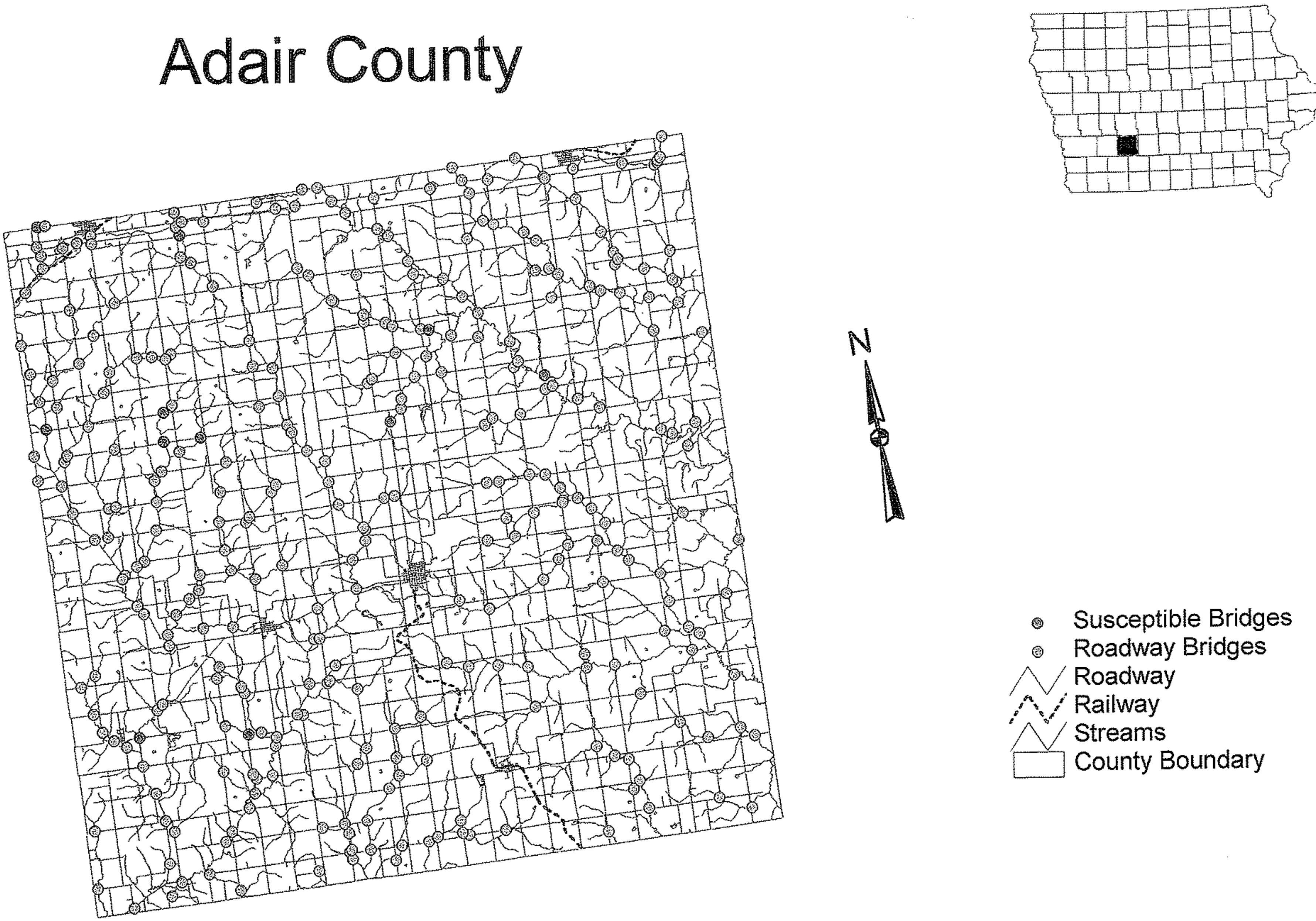
Adair County



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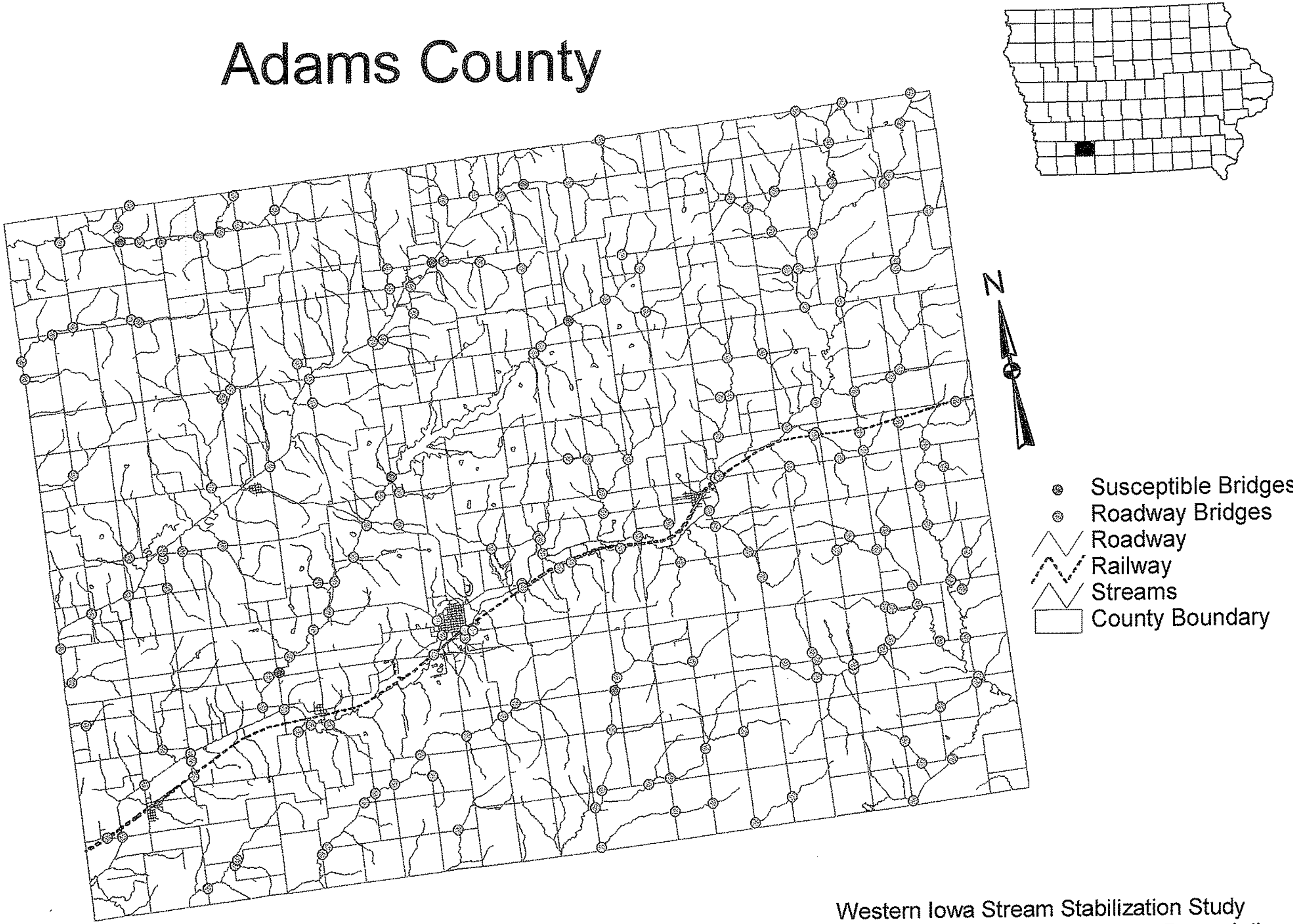
Bridges Susceptible to Damage from Degradation
March 1998

Adams County



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Bridges Susceptible to Damage from Degradation
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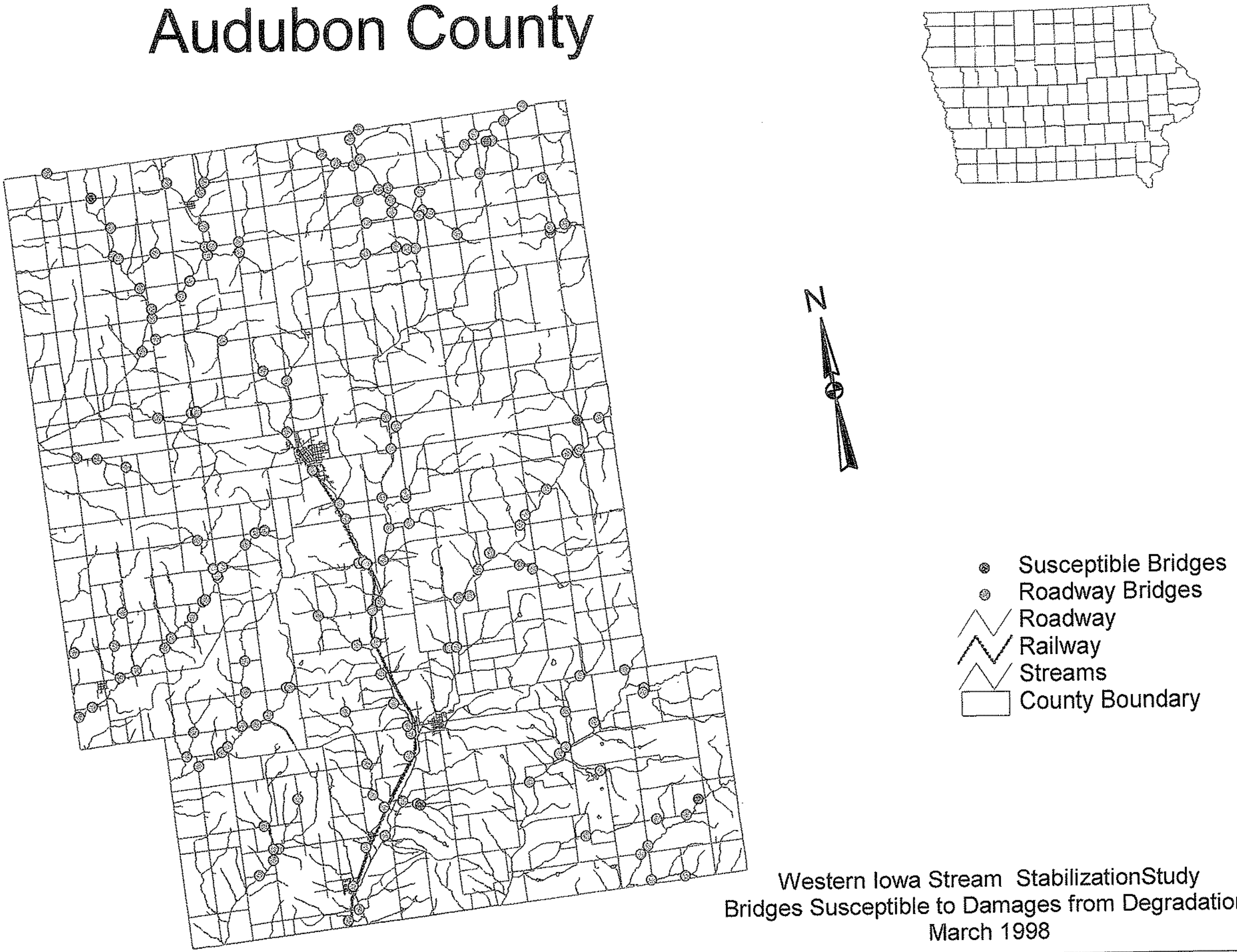
Audubon County



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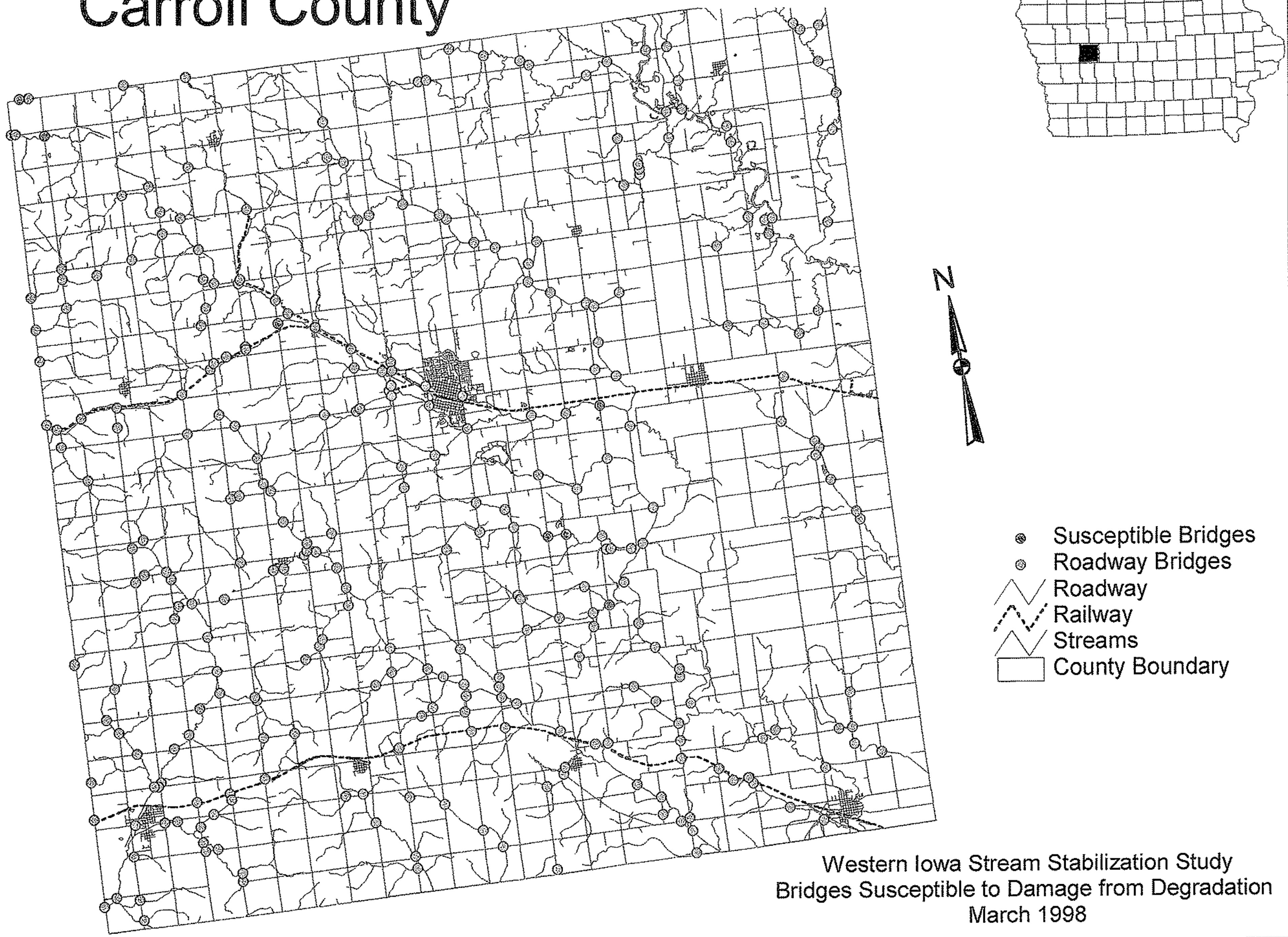
Carroll County



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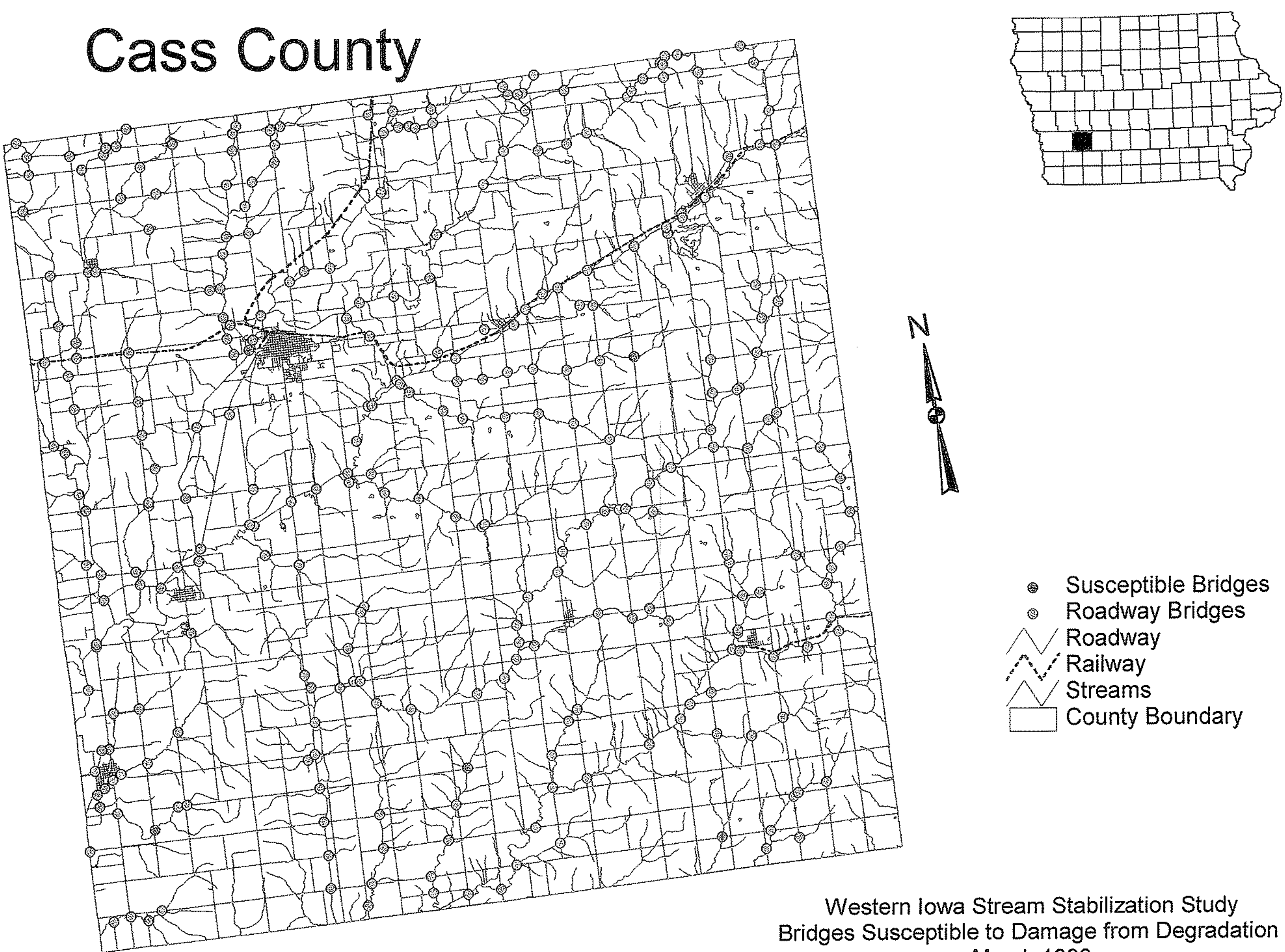
Cass County



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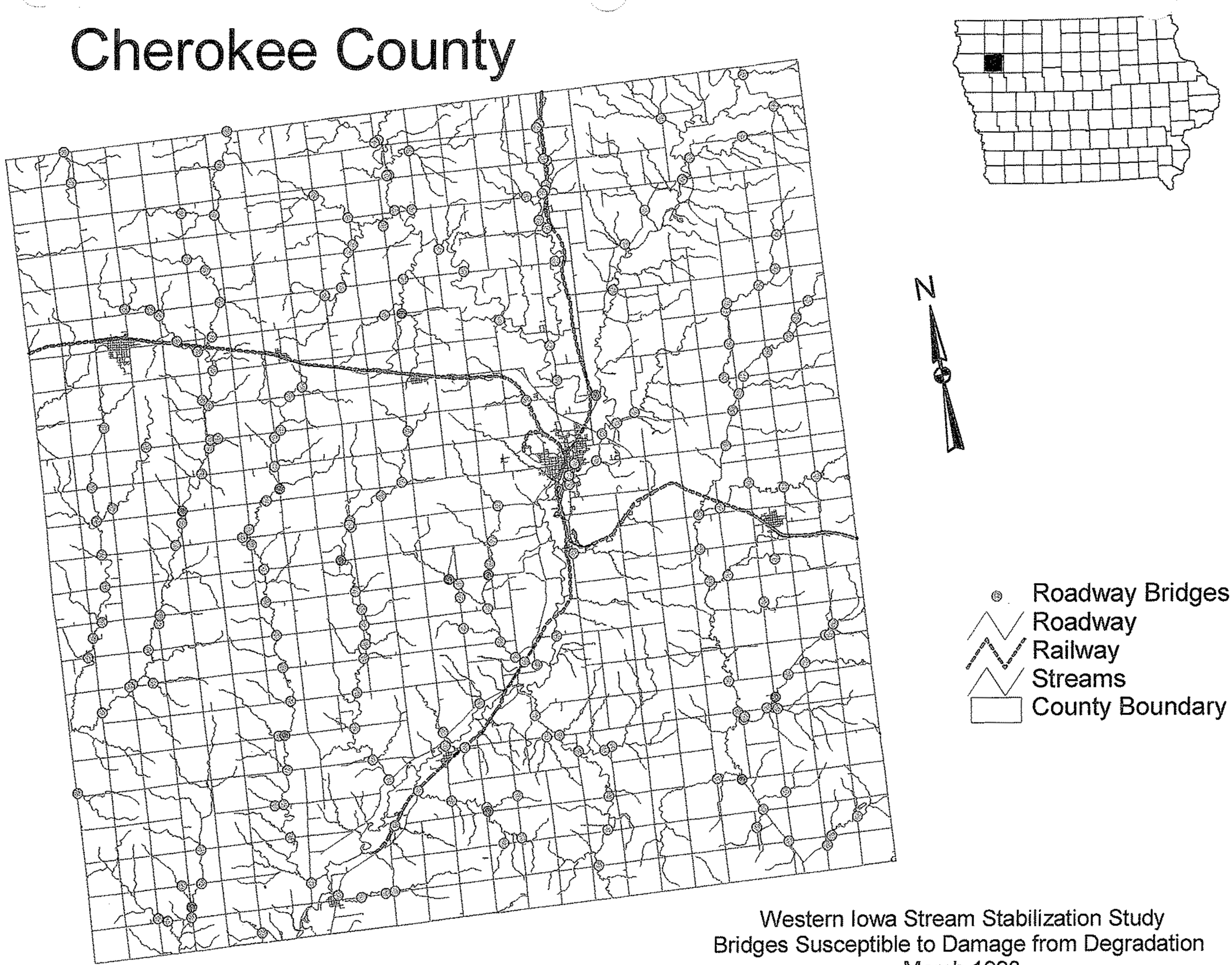
Cherokee County



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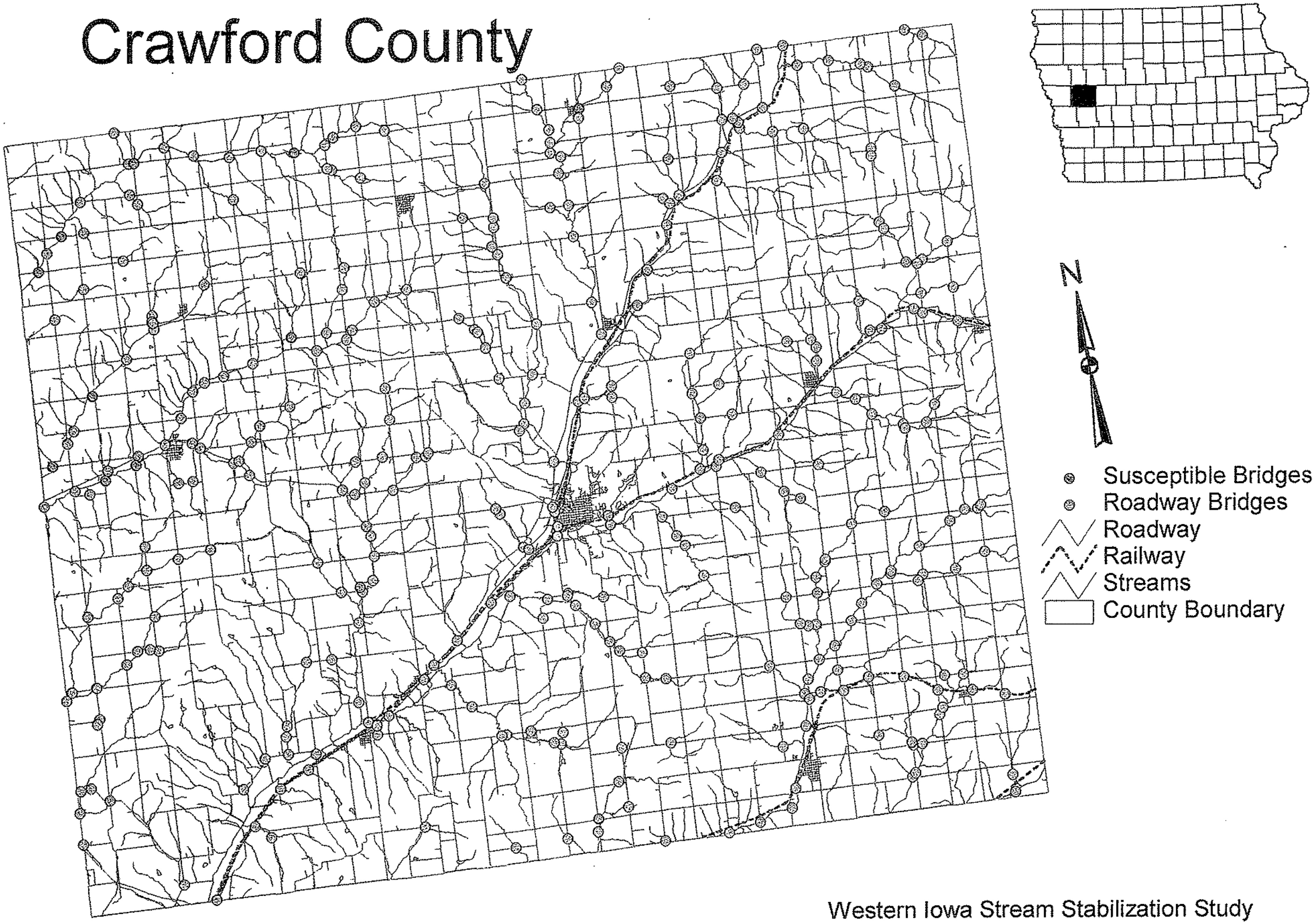
Crawford County



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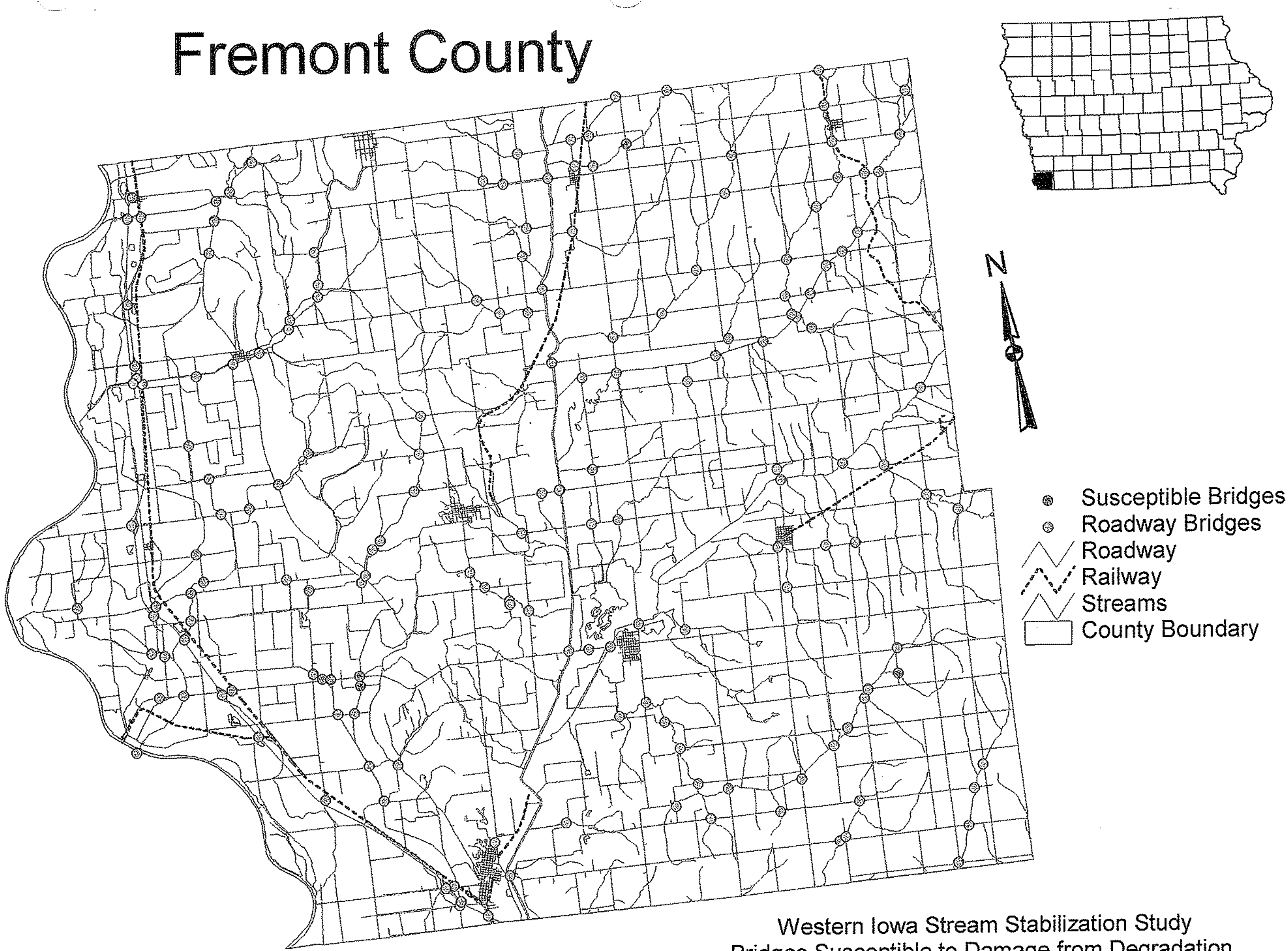
Fremont County



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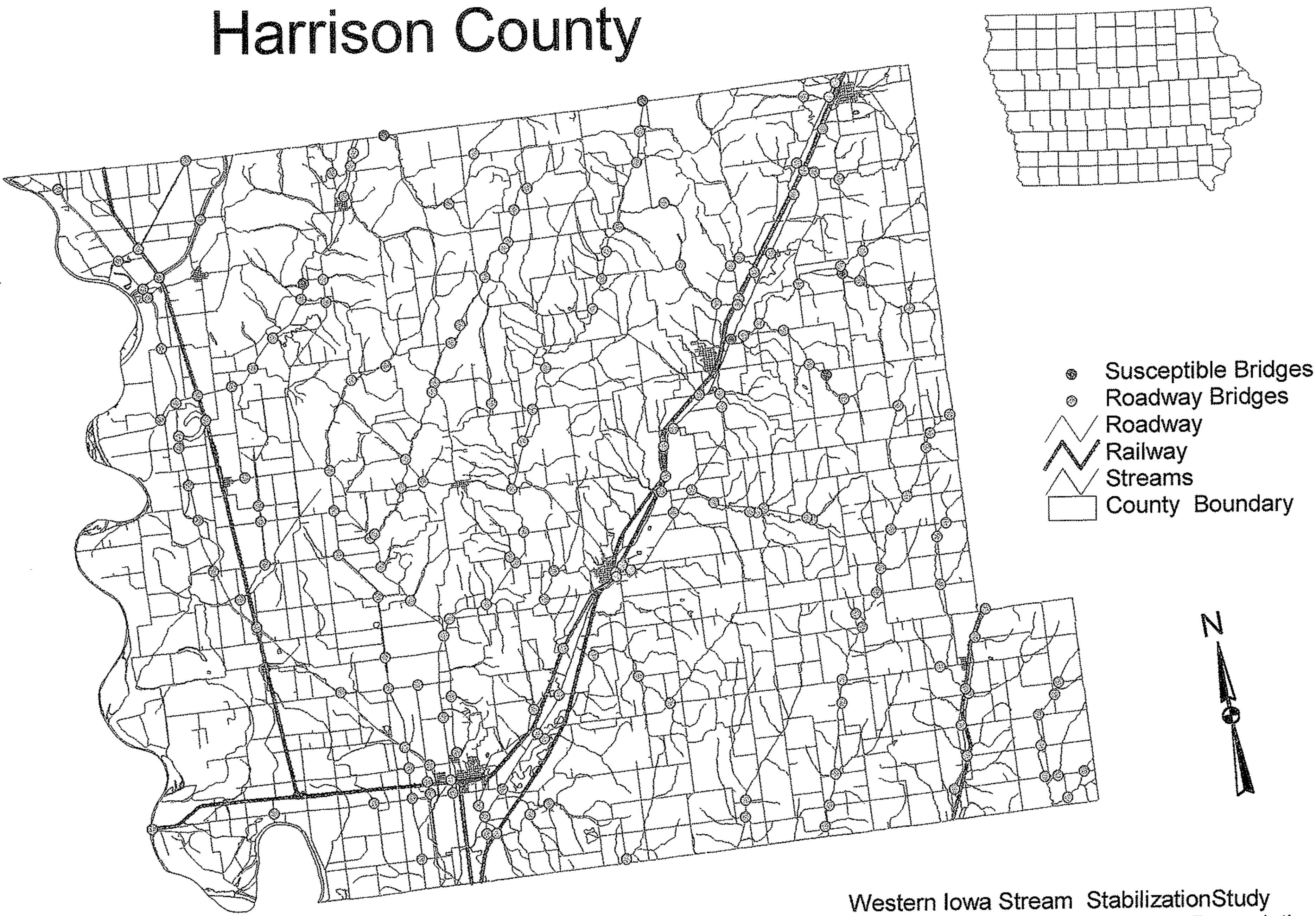
Harrison County



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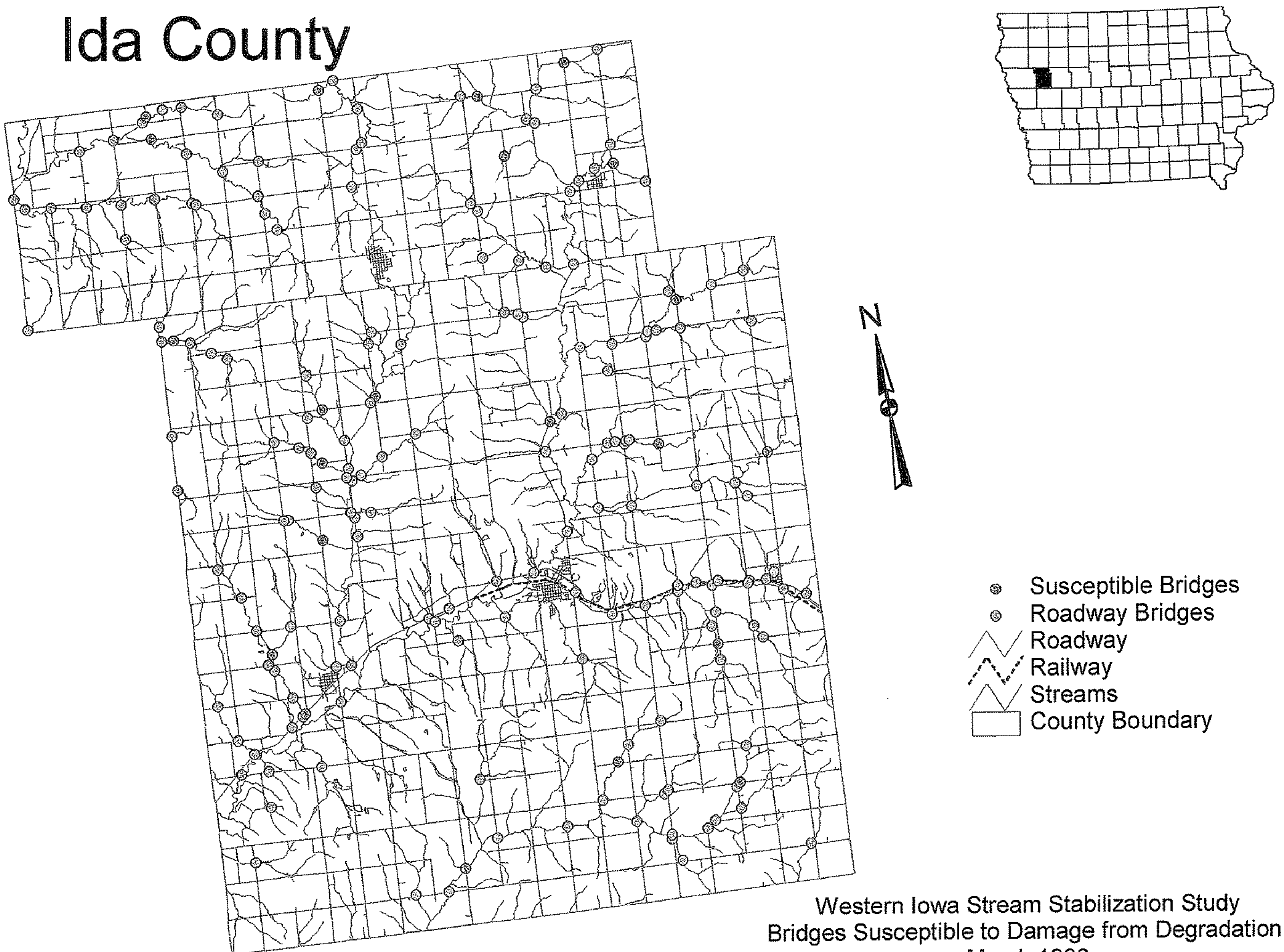
Ida County



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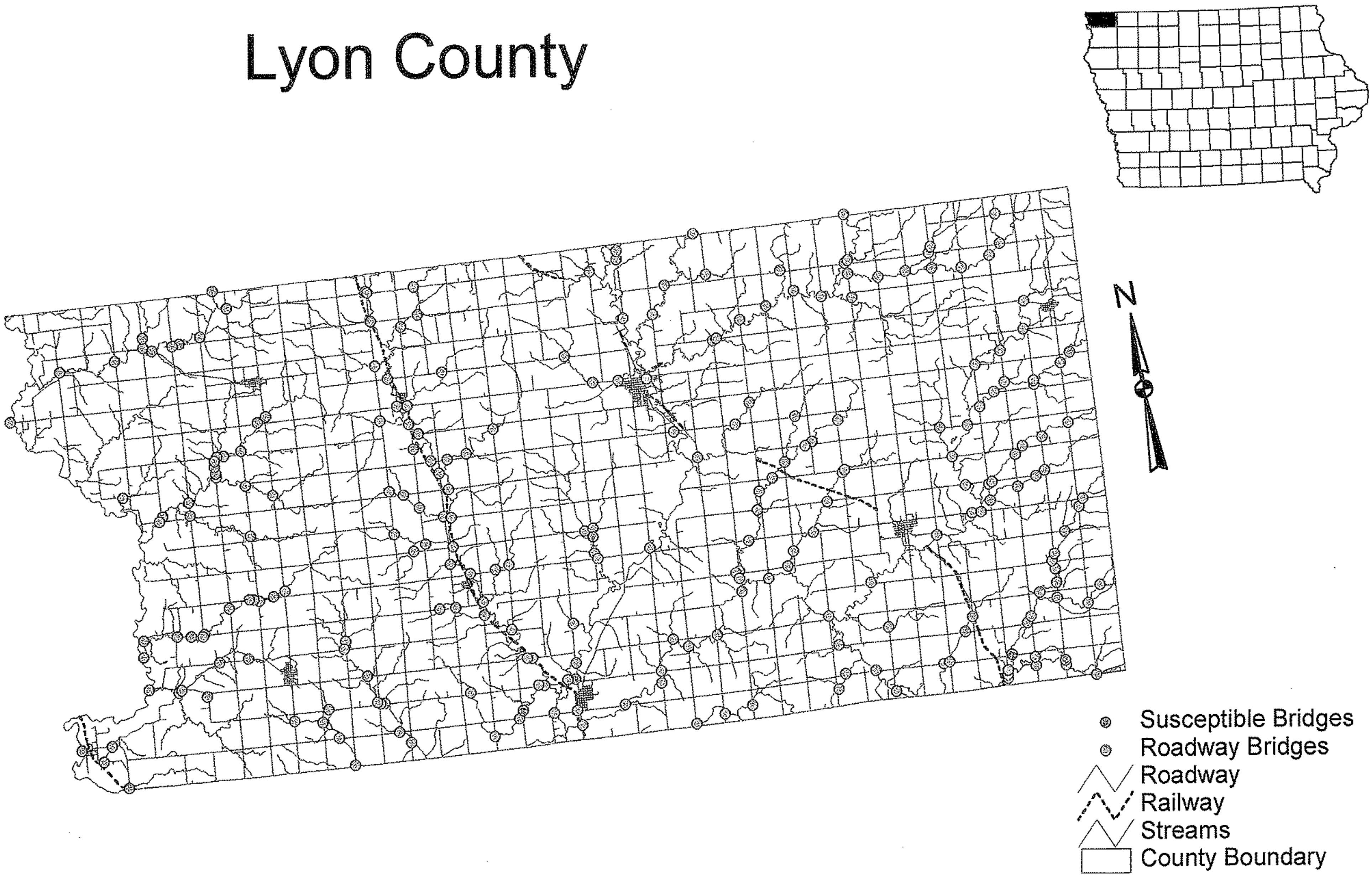
Lyon County



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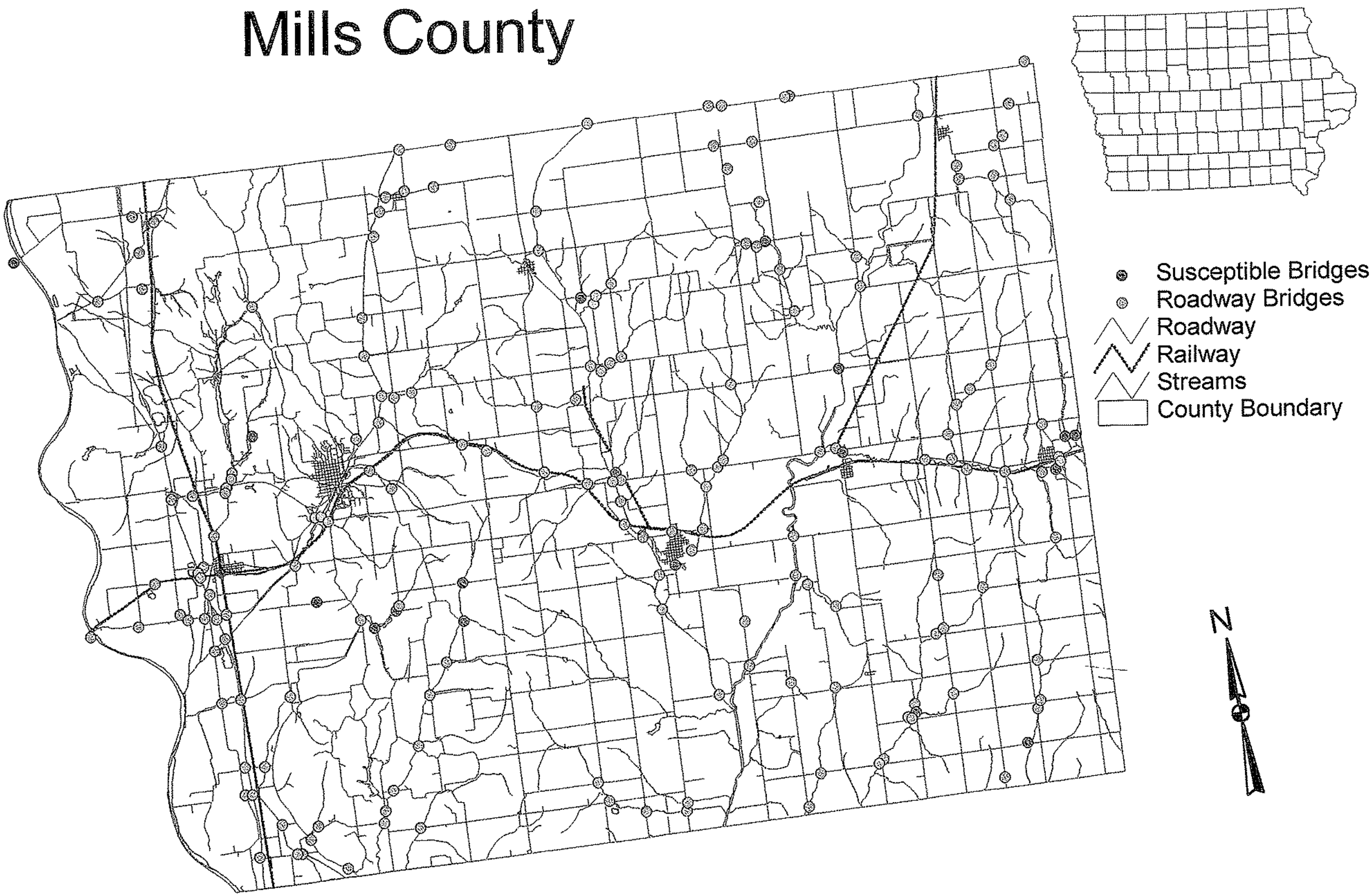
Mills County



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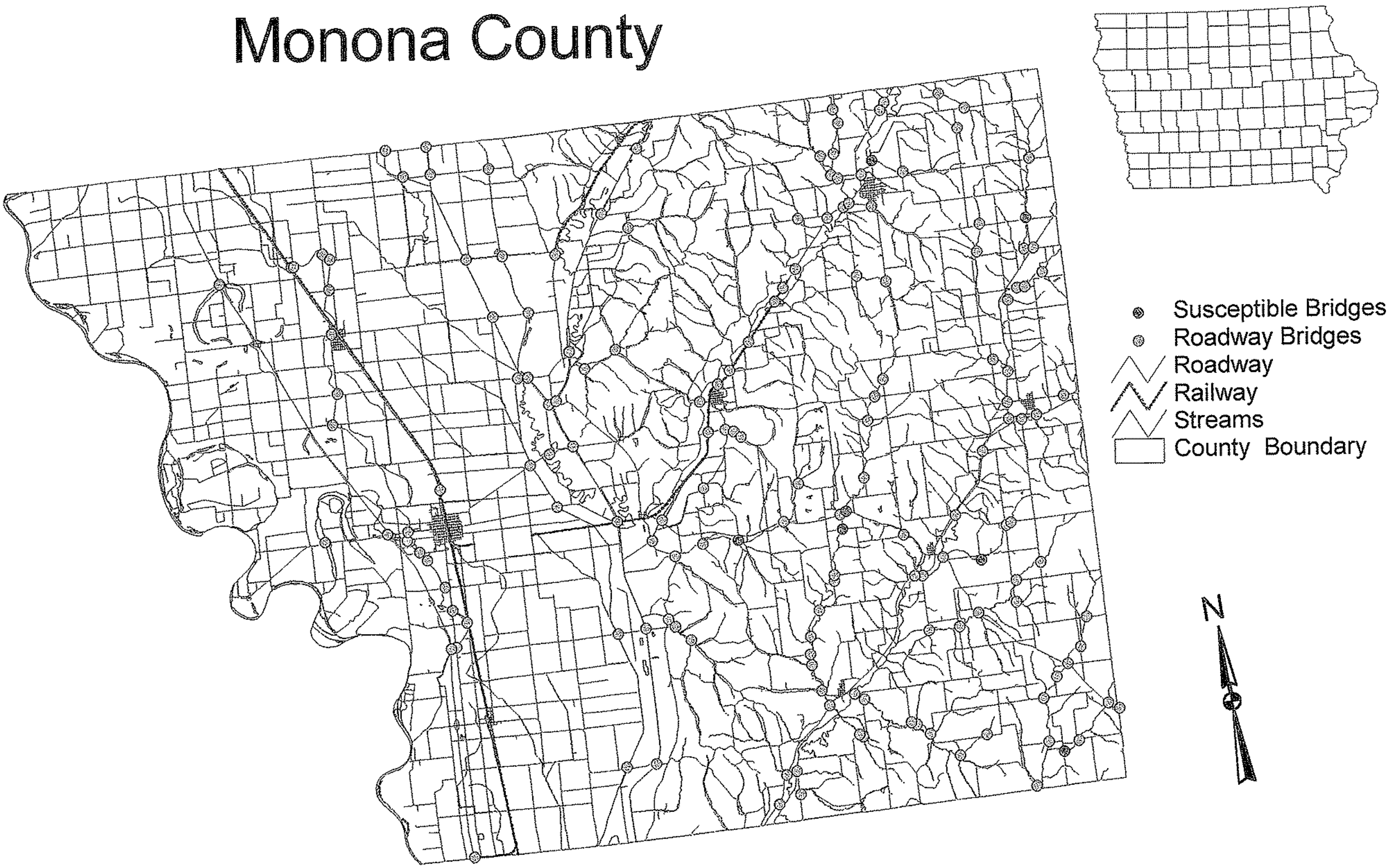
Monona County



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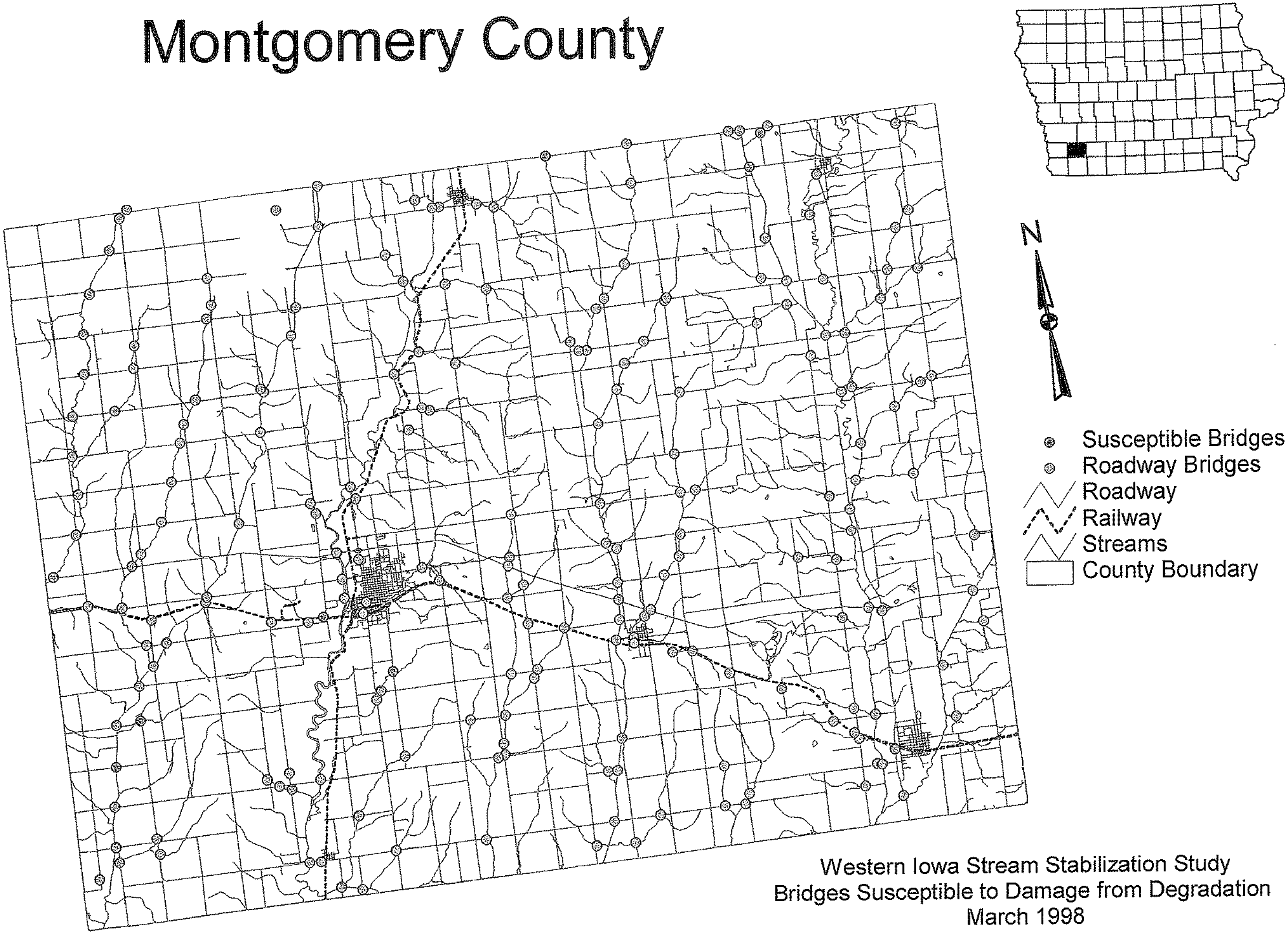
Montgomery County



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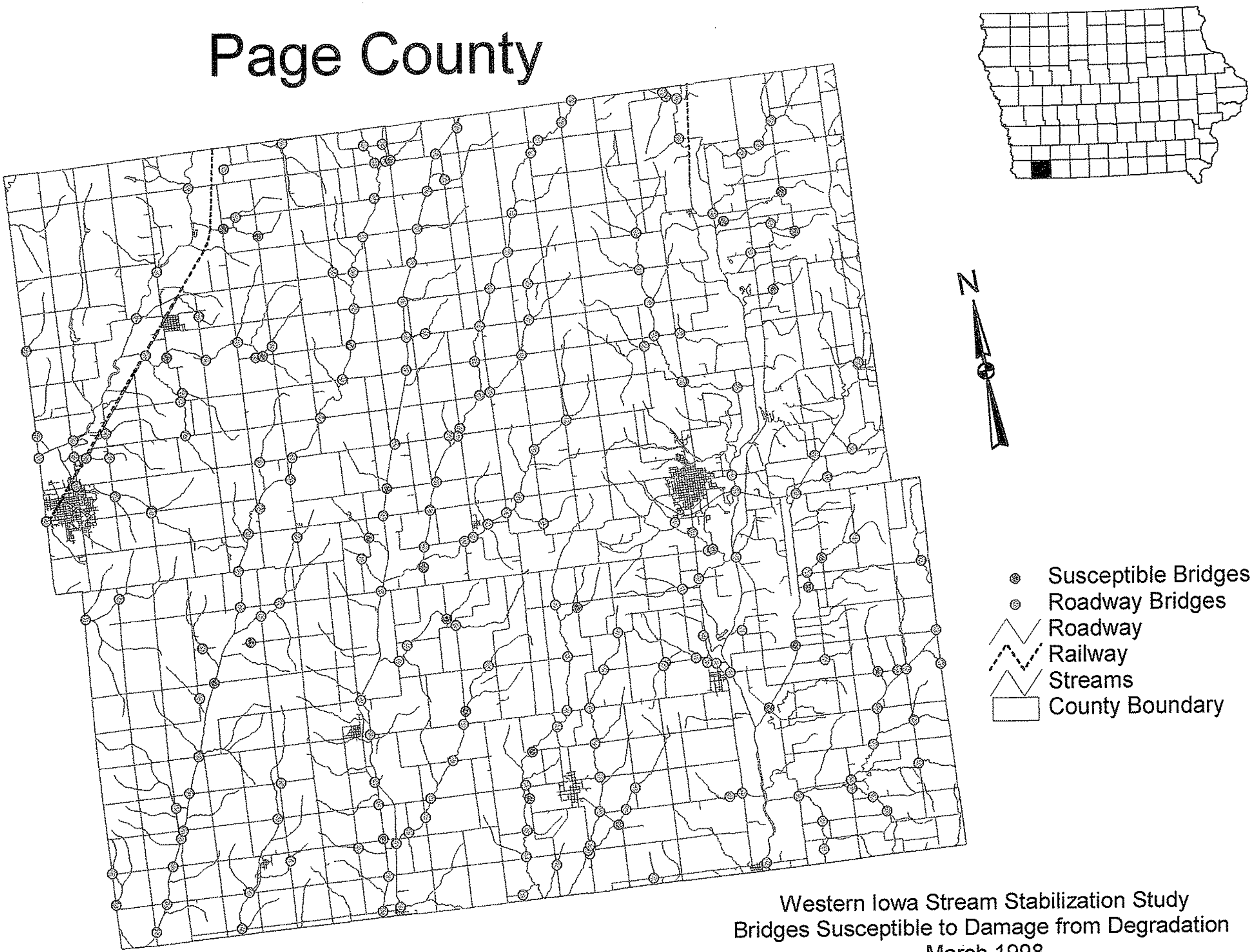
Page County



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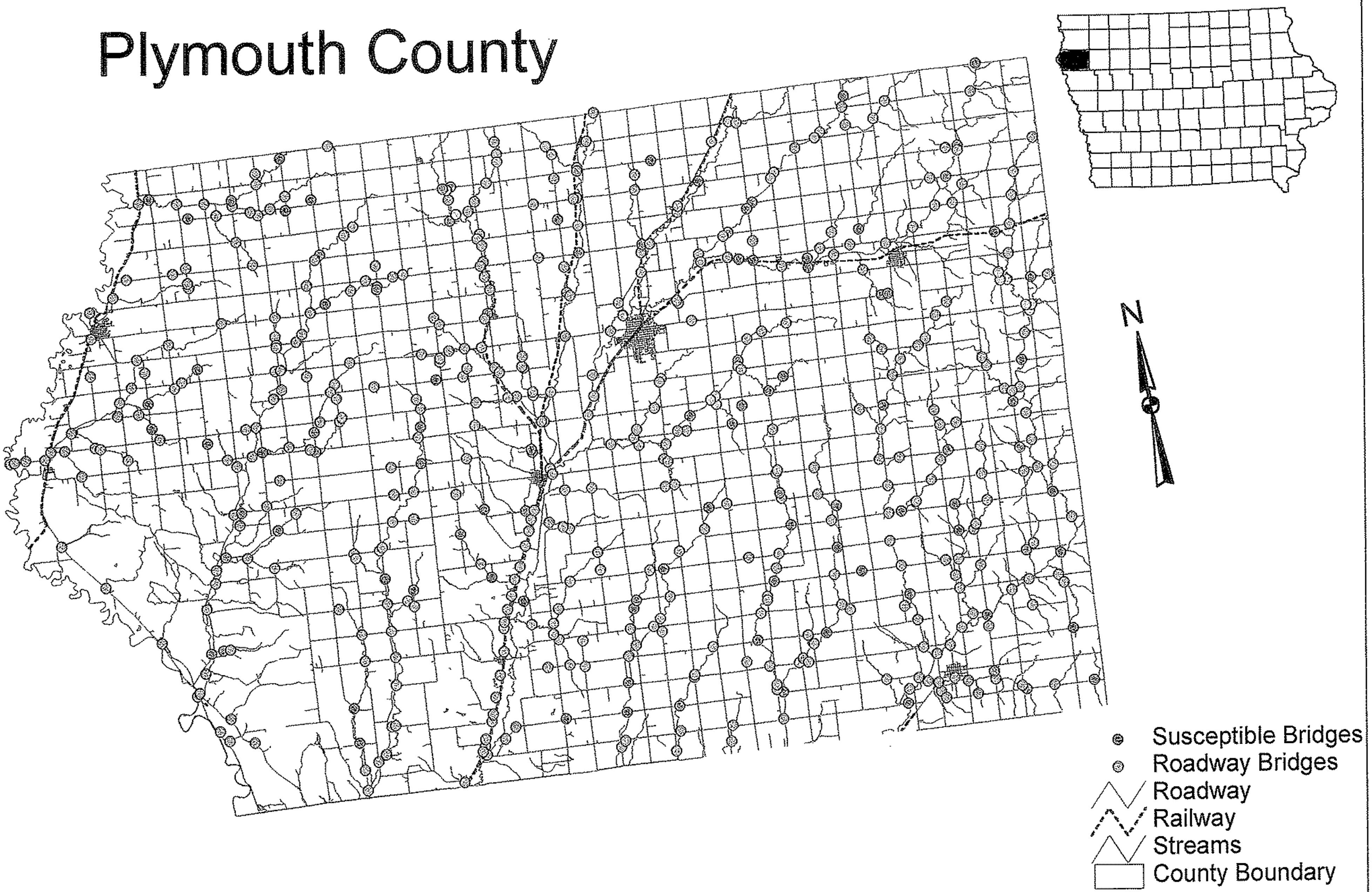
Plymouth County



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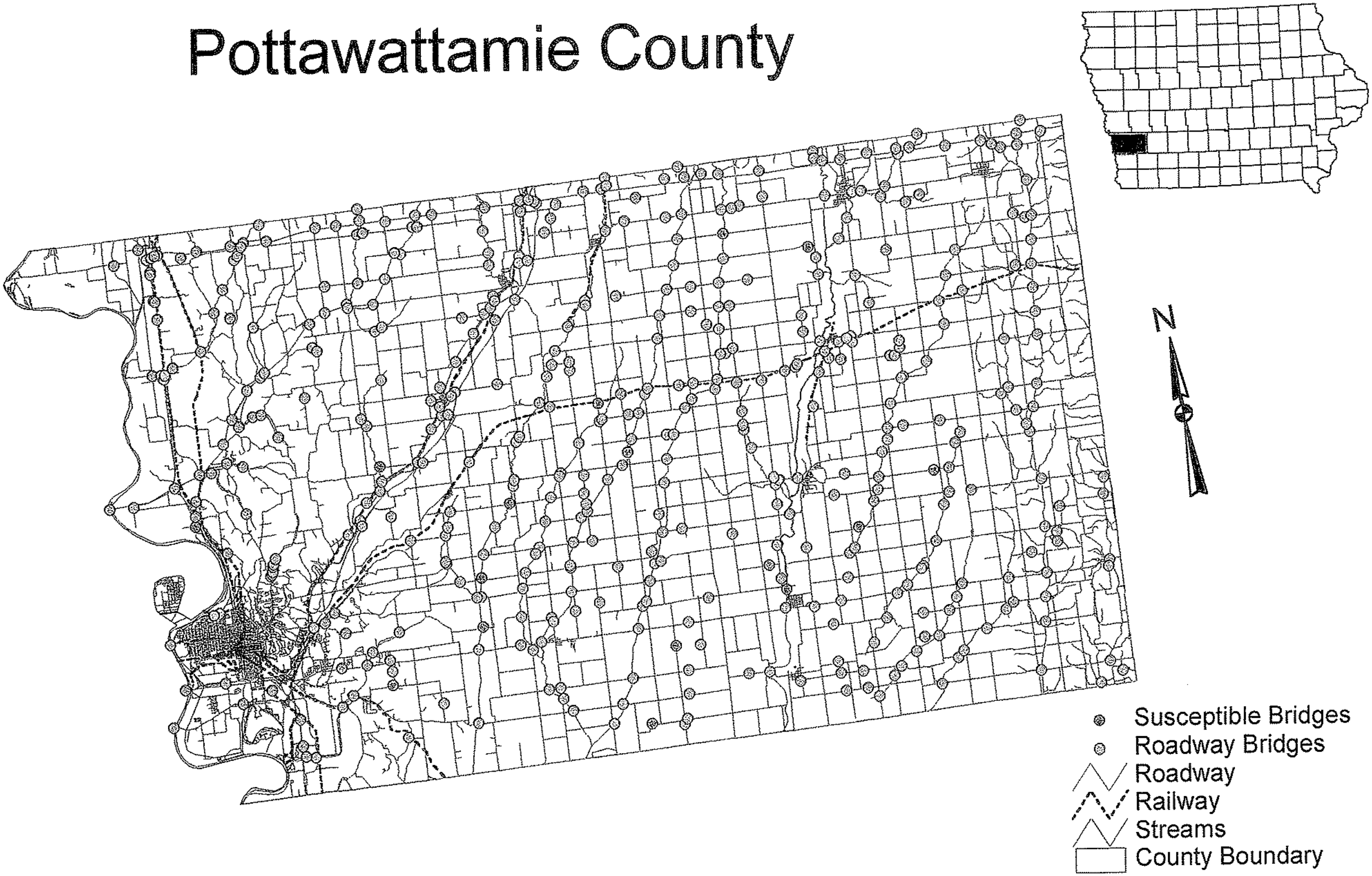
Pottawattamie County



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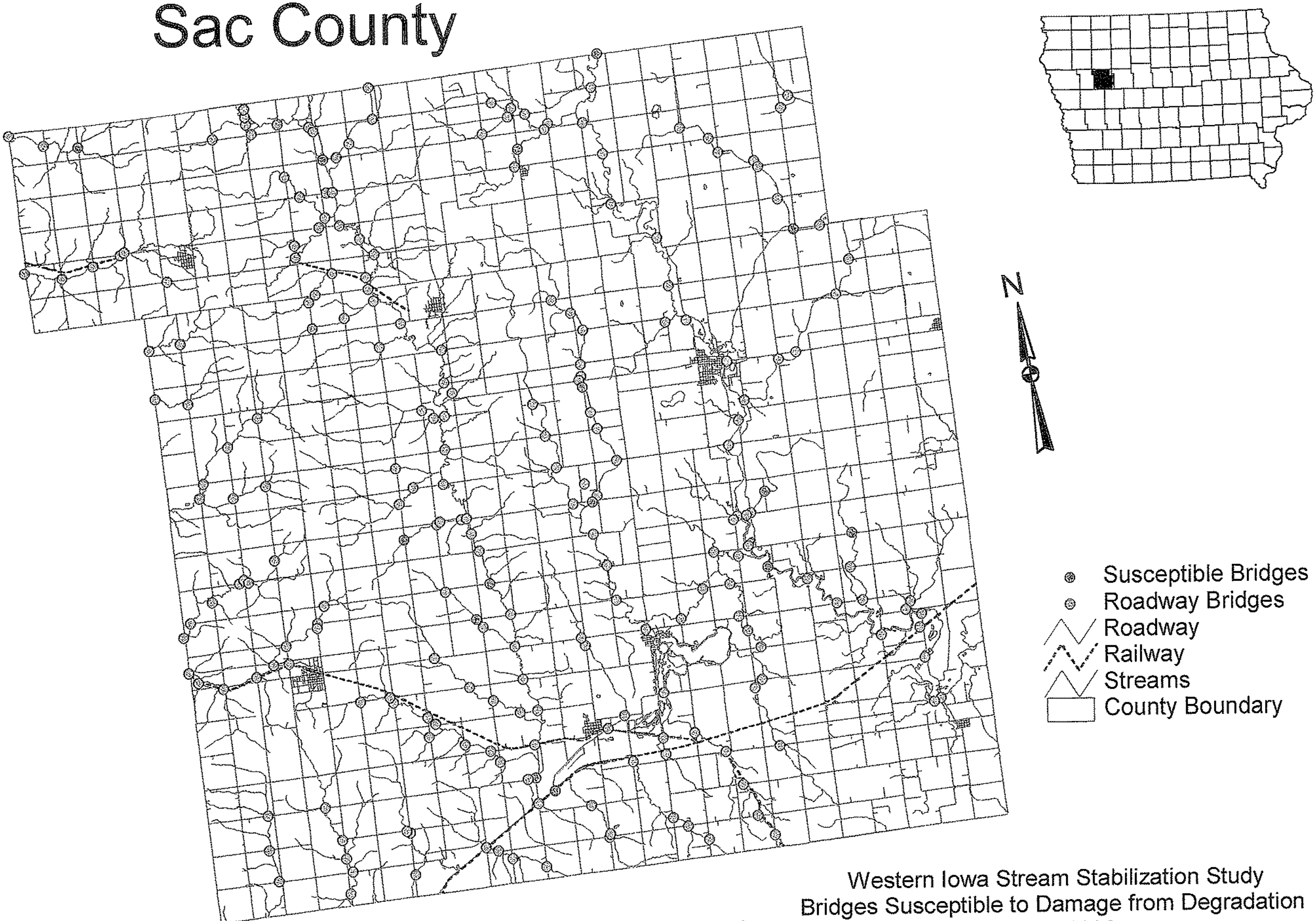
Sac County



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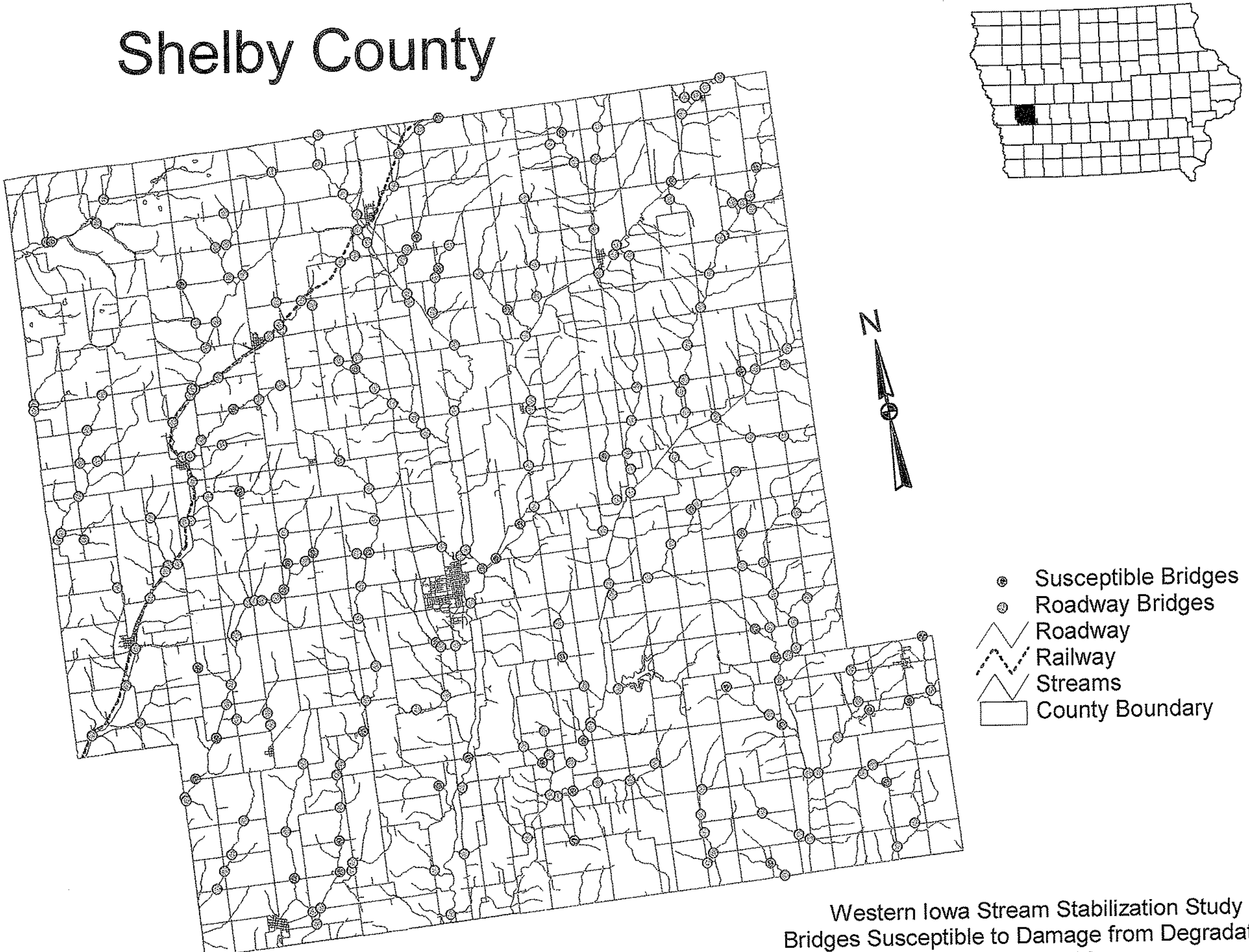
Shelby County



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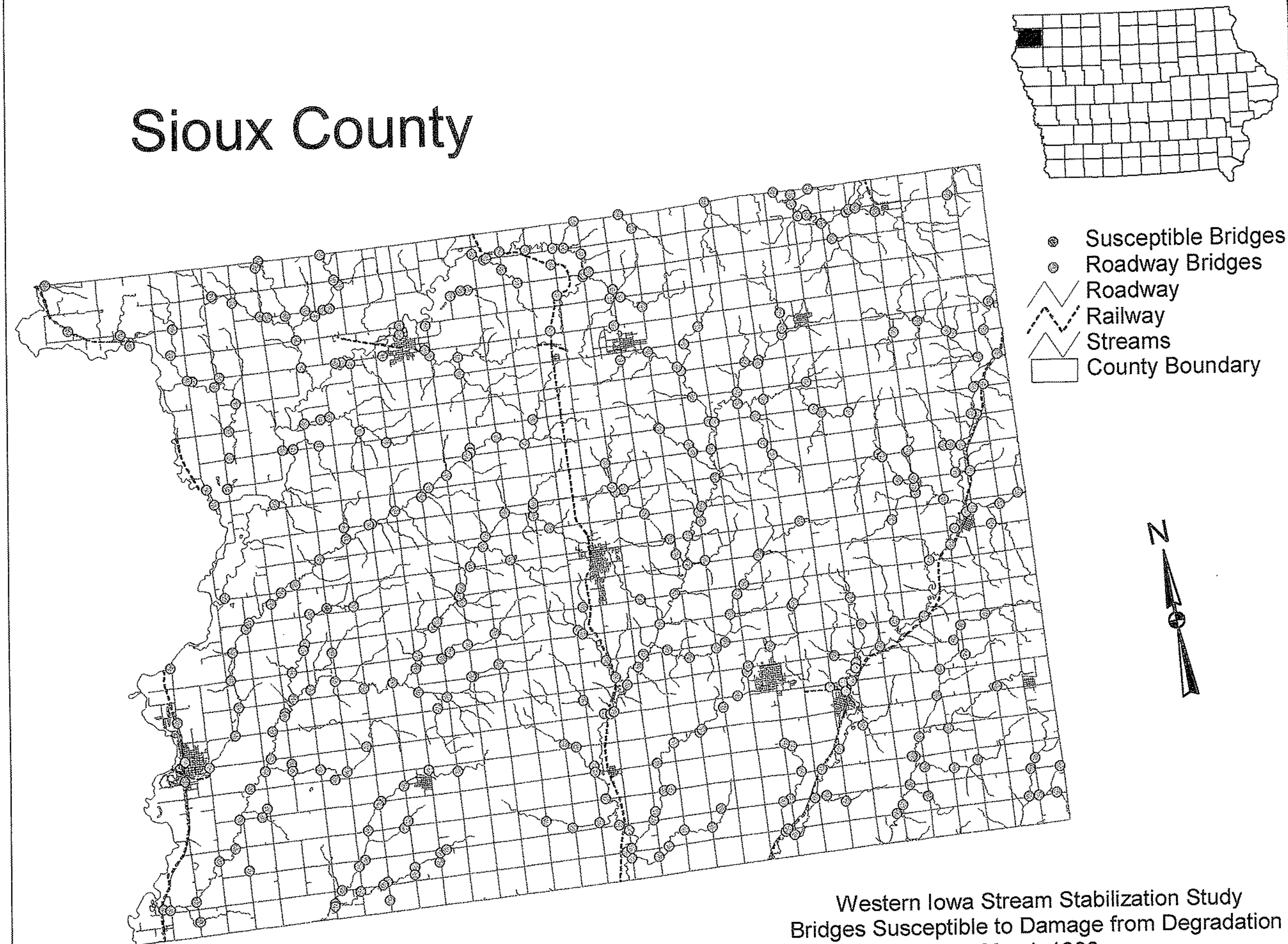
Sioux County



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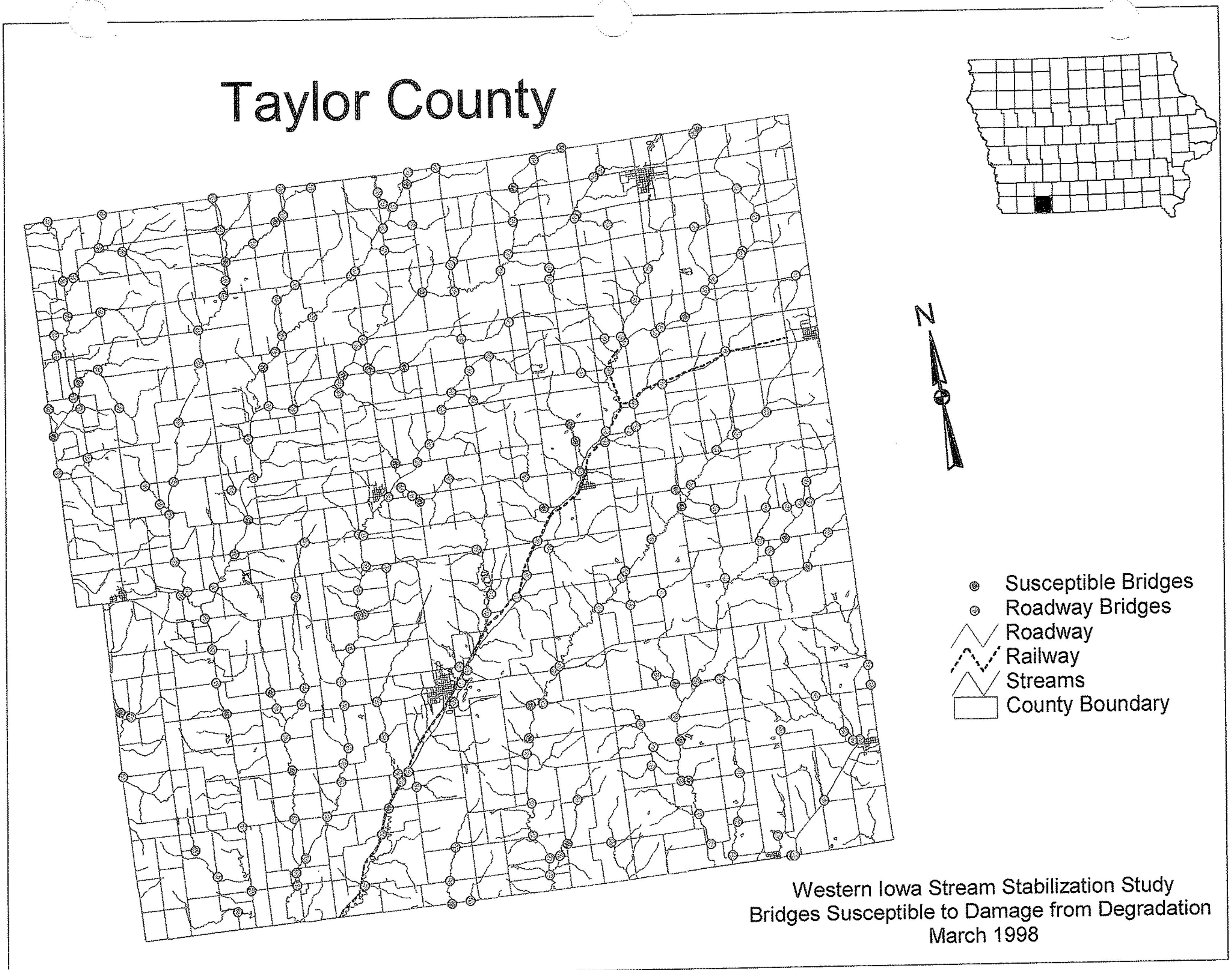
Taylor County
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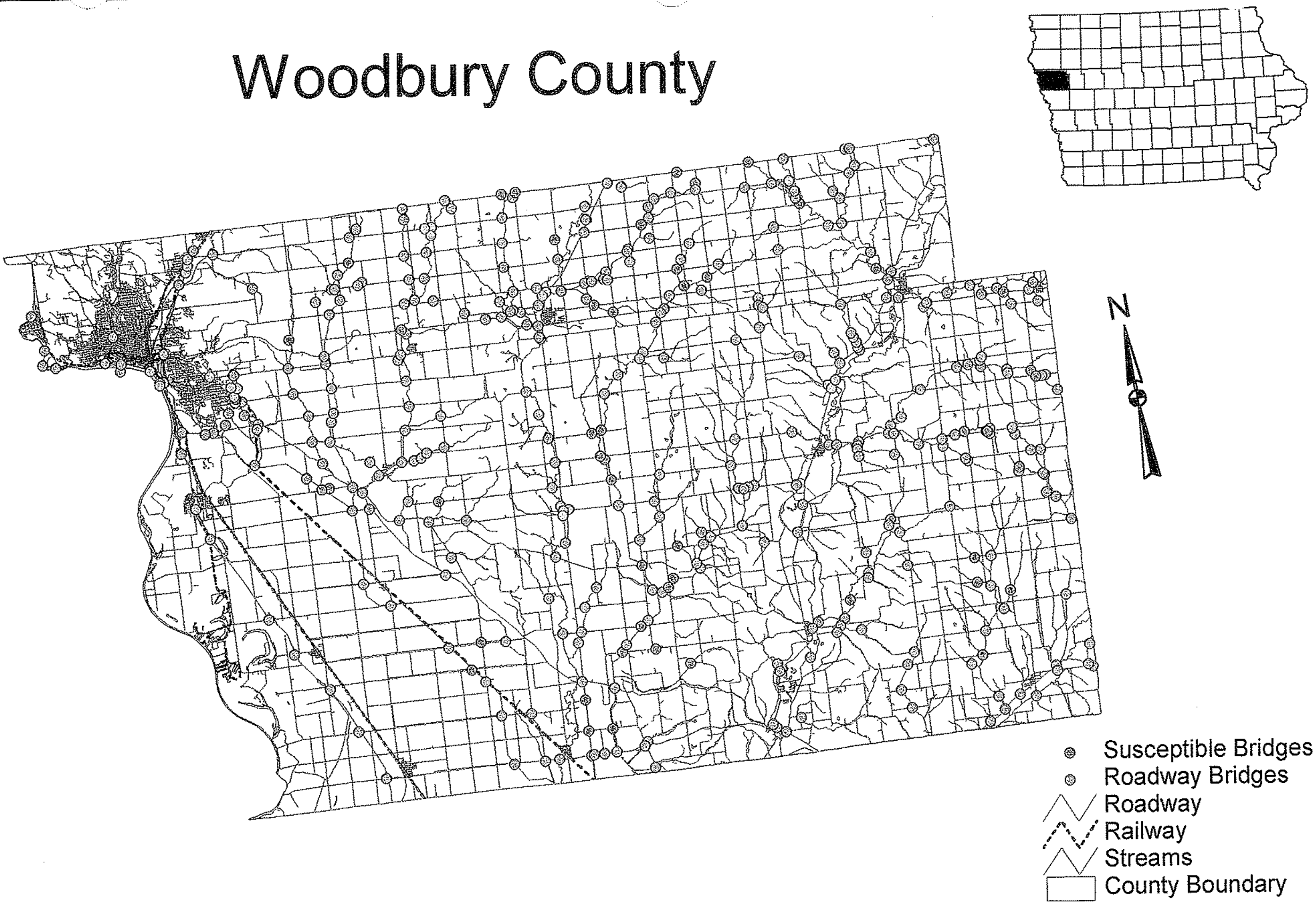
Susceptible Bridges
Roadway Bridges
Roadway
Railway
Streams
County Boundary
Western Iowa Stream Stabilization Study
Bridges Susceptible to Damage from Degradation
March 1998

Woodbury County



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