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upper bank erosion	boat waves	shear stress	rock riprap
lower bank erosion	revetments	flow control	vegetation

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

Joint data compilation, hydrologic and geotechnical information study assembled by the U.S. Army Corps of Engineers, NED and Colorado State University, to assess streambank erosion along 141-mile reach of Conn. River from Turners Falls Dam, Mass. to headwaters of Wilder Reservoir, Haverhill, N.H. and Wells River, VT.

Report includes literature and historical review of Conn. River; existing data collection, collation and evaluation; infield short-term data collection and evaluation; definition and analysis of bank erosion causal factors; and

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19. (cont.)

flood frequency
energy slope
headcut
seive analysis
sloughing banks
climatic factors

20. (cont.)

solution alternatives. Report identifies variables causing streambank erosion; summarizes and analyzes causes; distinguishes between general and upper-bank erosion; considers tractive and surgace forces; and specifies common techniques used to control bank erosion. Non-structural measures also discussed.

Maps, charts, tables, glossary, bibliography, and suggested protection measures.

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02154

REPLY TO
ATTENTION OF

NEDPL-BU

21 November 1979

LTG John W. Morris
Chief of Engineers
Department of the Army
Washington, DC 20314

Dear General Morris:

This report brings to a conclusion the Connecticut River Streambank Erosion Study which was authorized by a resolution adopted by the Committee on Public Works of the United States House of Representatives on 11 April 1974.

The study resolution was introduced and adopted after much public concern was expressed over erosion along the banks of the Connecticut River. This erosion is a chronic and damaging occurrence; however, the causes and possible solutions were not understood. The public wanted to know why the bank was eroding and how the erosion could be stopped.

A public meeting was held in Hanover, New Hampshire, in April 1975 to explain the purpose and scope of the study and to gain insight on the public's view of the streambank erosion problem and its social and economic damages.

Soon after, study efforts revealed that there was not enough hydrologic and geotechnical information available to assess the causes of erosion and possible corrective measures. Accordingly, a data collection program was undertaken.

Six erosion areas were chosen to be intensely studied. New data on these areas were collected. These data came from subsurface explorations and semi-annual topographic and hydrographic surveys, river velocity measurements, groundwater level observations and updated photographic and written records. Existing data used came from survey records, photography (area and ground), flow records, recorded observations of erosion and historical records of the operation of four hydroelectric plants.

After a period of three years, everything was put together and a comprehensive data bank was established.

In July 1978 Colorado State University (CSU) was engaged to determine whether the collected data was adequate to properly assess the causes of erosion and to make general predictions regarding the effectiveness of certain corrective measures. CSU's report stated that there were sufficient data available to make the assessment. CSU was then put under contract to visit the areas, attend public workshop sessions and report on causes of erosion and corrective measures.

During the entire study, considerable public concern was expressed regarding the erosion that is taking place along the banks of the Connecticut River. This concern resulted in a very active and successful public involvement program. The mailing list of those concerned grew to a total of 150. Eleven separate mailings over the course of the study included status reports, a review draft of the CSU contract, and a review draft of the final consultant report. A formal public hearing and seven public workshop meetings were held.

Two particular groups, For Land's Sake, Thetford, Vermont, and the Citizens Advisory Committee on the Proposed Northfield Diversion Project, Springfield, Massachusetts, provided critical reviews of work presented at the public workshop meetings and the resulting draft report. Many other organizations and individuals also provided us with helpful review comments on all facets of our work up to and including the draft of this document.

The For Land's Sake group has reviewed the draft of this report and provided us with some very valuable criticism of the document. The report is undoubtedly a better document because of the group's effort. I must note, however, that the For Land's Sake organization is still not satisfied with all of the reported findings and conclusions.

In conclusion, I concur with the consultant's (CSU's) approach to the study, their findings and their conclusions. I consider that the report is presented to the highest degree of detail and accuracy commensurate with the data available. The findings and conclusions are adequately supported in the text with tables and charts. I believe that the report fully meets the requirements of the contract and responds to questions raised during the draft review period, and completely fulfills the letter and intent of the authorizing resolution.

NEDPL-BU 21 November 1979

LTG John W. Morris

I recommend that this document be accepted as the final report to be forwarded to the Congress. I further recommend that this report be transmitted to the Department of Energy, to serve as an informational source for hydroelectric licensing on the Connecticut River.

Finally, I express my thanks to all, in both the private and the public sector, who have helped during the course of the study and in the preparation of this report.

Sincerely,

MAX B. SCHEIDER
Colonel, Corps of Engineers
Division Engineer

CONNECTICUT RIVER STREAMBANK EROSION STUDY
MASSACHUSETTS, NEW HAMPSHIRE AND VERMONT

CONTRACT NO. DACW 33-78-C-0297

Prepared for

U.S. Army Corps of Engineers
New England Division
Waltham, Massachusetts

Prepared by

D. B. Simons
J. W. Andrew
R. M. Li
M. A. Alawady

AUTHORIZATION

This investigation was conducted for the U.S. Army Corps of Engineers, New England Division, under Contract No. DACW33-78-C-0297. John Smith was the authorized Project Manager for the U.S. Army Corps of Engineers and Daryl B. Simons was the Principal Investigator for Colorado State University Research Institute. Primary technical assistants to Dr. Simons were Dr. Ruh-Ming Li, Co-Principal Investigator from Colorado State University Research Institute and Dr. John W. Andrew, General Manager from Water and Environment Consultants, Inc., a subcontractor for Colorado State University Research Institute. The purpose of this investigation was to collect and analyze data with regard to streambank erosion along a 141-mile reach of the Connecticut River extending from Turners Falls Dam, Massachusetts (River Mile 122) to the headwaters of Wilder Reservoir in Haverhill, New Hampshire and Wells River, Vermont (River Mile 263). In accordance with the contract, the report describing the study and conclusions is submitted.

EXECUTIVE SUMMARY

The Connecticut River Streambank Erosion Study was conducted in accordance with Contract No. DACW33-78-C-0297 between the U.S. Army Corps of Engineers, New England Division, and Colorado State University Research Institute. Colorado State University Research Institute was the contractor and was supported by the subcontractor, Water and Environment Consultants, Inc. The study reach extends from Turners Falls Dam, Massachusetts (river mile 122) to the headwaters of Wilder Hydro Pool in Haverhill, New Hampshire and Wells River, Vermont (river mile 263). The scope of work of the study includes (1) literature and historical review of the Connecticut River, (2) existing data collection, collation and evaluation, (3) infield short-term data collection and evaluation, (4) definition and analysis of bank erosion causal factors, and (5) solution alternatives. The contractor was also requested to conduct three pairs of public meetings at two different locations within the study area. To conduct this study, the U.S. Army Corps of Engineers allocated a total sum of $83,998.00 to the contractor, of which $45,160.00 was subcontracted to Water and Environment Consultants, Inc. to conduct Items (1), (2) and (3) of the scope of work and to generally assist in all phases of the study.

The basic data upon which the analysis is based was provided by the U.S. Army Corps of Engineers, New England Division, and supplemented as required and where economically feasible with field data and historic documents assembled by Water and Environment Consultants, Inc. The identification and examination of the data are treated in detail in Section 2 of this report.

The data base provided by the Corps of Engineers was, in general, adequate for a general analysis. The few exceptions include:

- Data describing pre-hydropower dam river conditions.
- Detailed information describing the complete geometry of the channel cross sections at the index sites.
- Information describing the quantity and characteristics of sediment transported in the system.
- Surveys of possible aggradation in the pools.

Considering the importance of the Connecticut River to the New England region, procurement of additional data as identified above and collection of water quality data are suggested to meet future needs.

The tractive force method of evaluating bank stability as applied by the Corps of Engineers was investigated. This method is widely accepted nationally and internationally. However, this method as applied does not account for all of the factors known to contribute to the erosion process. Because of the shortcomings of the tractive force method, the Scope of Work required a more detailed analysis of factors causing bank erosion along the Connecticut River.

The important variables causing bank erosion are identified (pages 65-67). Note that the causes are numerous and can act more or less individually or in unison with one another, making it very difficult to precisely identify the relative importance of each cause or variable. Also, the environment within which these forces act is incredibly complex involving a wide array of sizes, gradations, and stratifications of bank material that may or may not involve cohesive strength. Additionally, the cohesive strength may be altered by loading and degree of saturation. Then there is the complexity of pools, bends, straight reaches, stage changes, pool operation, wind and boat generated waves, mass wasting, freezing and thawing, vegetation, etc. In fact, it should be noted that identification of causes of bank erosion is a long-term goal of the Streambank Erosion Control Evaluation and Demonstration Act of 1974. This goal has not been achieved thus far, but with a national effort involving millions of dollars, improved appreciation and understanding of the bank erosion problem and bank protection measures may be forthcoming.

In accordance with the Scope of Work, an attempt was made to better quantify the numerous causes of bank erosion utilizing available data, current theory, personal experience and sound professional judgment. The summary of this analysis is presented in Table 2 (page 81) and from this table it is possible to qualitatively evaluate the major variables causing bank erosion. This application of the results presented in Table 2 is applied to six general river conditions in Table 3 (page 89) and to the six index sites in Table 6 (page 109). Note that forces exerted on the bank of a channel by the flowing water can be increased as much as 60 percent by such factors as flood stage variations, pool fluctuations, boat and wind waves, etc. Evaluation of forces causing bank erosion verifies the relative importance of causative factors. In descending order of importance they are: shear stress (velocity), pool fluctuations, boat waves, gravitational forces, seepage forces, natural stage variations, wind waves, ice, flood variations, and freeze-thaw.

Analysis of the causes of bank erosion shows that these causes can be subdivided into those that cause general bank erosion and those that cause upper bank erosion. Tractive forces exerted by flowing water cause general bank erosion, with their maximum attack occurring at about two-thirds of the depth below the water surface. Hence, even if the upper bank is stable or stabilized, the flow can erode the lower bank causing failure of the lower and upper banks. Forces such as wind waves, boat waves, pool fluctuations, ice, etc., are the most common causes of upper bank erosion. If these forces are of sufficient magnitude and if the bank material is erodible, erosion of the upper bank can occur. In time, a berm or beach is formed and an improved level of upper bank stability is achieved. However, during subsequent periods of high flow, the tractive force may erode the total bank line, re-establishing conditions favorable for upper bank erosion and the possibility of repetition of the erosion cycle.

To protect banks from erosion it is essential to consider the tractive force exerted by the flowing water as well as surface forces causing erosion of the upper bank. Protection of the upper bank line by any means will fail when lower bank erosion occurs. Hence, stabilization of the total bank or at least the lowest two-thirds of the bank is essential to prevent general and upper bank erosion.

The report specifies how the stability of any bank line in the Connecticut River can be assessed to determine the potential for additional bank erosion. It also specifies the most common techniques that can be utilized to control bank erosion. However, it should be stressed that structural measures are very expensive.

Non-structural measures for controlling erosion are also identified and discussed. The non-structural measures are more useful for protection of the upper banks. However, the success of all non-structural measures depends on the lower bank stability. That is, by controlling boat waves and wind waves, by limiting pool fluctuations and by encouraging growth of vegetation on the upper banks, the upper bank erosion problems can be significantly reduced. However, during periods of flooding the lower banks may yield to the attack of the tractive force exerted on the banks by the flowing water. If this occurs, the upper bank will be subject to erosion.

In conclusion, the methodology presented and corrective measures identified can be utilized to design specific structural or non-structural measures to control erosion both where it has occurred, and where there is high potential for it. Furthermore, limited control of upper bank erosion can be achieved by limiting pool fluctuations associated with hydropower development and by limiting the amount of river traffic, particularly high speed pleasure craft. However, adoption of such measures will not eliminate major bank erosion that may occur during periods of flooding.

ACKNOWLEDGMENTS

Many people contributed to the completion of this study. The writers would like to extend their appreciation to all those who assisted in this effort and mention a few who were particularly helpful.

John Smith was the Project Manager representing the U.S. Army Corps of Engineers. He provided the guidance and data required to conduct the study. The group, "For Land's Sake", is acknowledged for their critical review of draft reports and guidance and assistance throughout the study. Special thanks are due to Dr. T. J. Ward, J. Y. Lu, and S. T. Combs of Colorado State University for providing valuable assistance with the analysis of the data. Ms. L. Y. Li of Water and Environment Consultants, Inc. is acknowledged for her work in compiling and preparing the Connecticut River data. Ms. Annette Ward of Colorado State University was particularly helpful in preparing and editing the manuscript, along with Tamra McFall. Technical staff of great service were Arlene Nelson, Head of Technical Typing; Susan Barnes and Callie Snyder, Technical Typists; and Hanae Akari, Head of Drafting.

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SECTION 1

INTRODUCTION

GENERAL

A predominant characteristic of alluvial channels is the change in location and shape that the channel and cross sections experience with time. These changes are particularly significant during periods when alluvial channels are subjected to comparatively high flows. The converse situation exists during relatively dry periods. Although other erosive forces may act in conjunction with the discharge forces, the quantitative flow rate and its relative duration have a capacity approximately 100 times greater than those forces acting during periods of intermediate and low flow to erode banks and transport sediment during flood stages. In most instances when considering the instability of alluvial rivers, it can be shown that approximately 90 percent of all river changes occur during that 5 to 10 percent of the time when large flows occur.

Regardless of the fact that the majority of bank changes occur during comparatively short time periods, there may also be regions within a river in which some degree of instability is exhibited for all flow conditions. Raw banks may develop on the outside of bends as a consequence of direct impingement of the flowing water. Sloughing banks may occur as a result of seepage and other secondary forces created by water draining back through the banks to the river. Continuous wave action, generated either naturally or by man's activities, may also perpetuate erosion problems. Despite the numerous types of secondary bank erosion, it must be stressed that the causes of bank erosion are normally small compared to the magnitude of erosion that can occur during those periods of comparatively high discharges.

During the last thirty years, serious physical and engineering studies have been undertaken to initially qualify the interaction between sediment and water movement within the riverine environment. Only during the last 20 years has the quantitative evaluation of this interaction been attempted and achieved with some degree of success. Unfortunately, a majority of these studies have concentrated on the interaction of sediment and water within the fluid suspended above the bed of the river and its relation to the bed itself. Information related to the delineation of the types of bank erosion and their quantification is almost nonexistent. However, delineation of the types of bank erosion and comparative evaluation of the causative forces may be evaluated for alluvial streams within a qualitative matrix. Within an acceptable degree of validity the forces may also be evaluated in tabular form for quantitative comparison.

The above comments are generally applicable to all alluvial rivers, including the Connecticut River bordering Vermont and New Hampshire and extending into Greenfield, Massachusetts. One of the most beautiful river reaches within the United States, the Connecticut River may be classified as relatively stable, though in some locations significant bank erosion is occurring by one or many natural or man-induced events. Regardless of the degree of channel stability, man's local activities may produce major changes in river characteristics, both locally and throughout an entire reach. The primary objective of any river engineering design is to achieve acceptable stability of the river. In order to meet this objective, an understanding of the direction and magnitude of change in channel characteristics caused by the actions of man and nature is required.

This study introduces and discusses the forces affecting the behavior of rivers, analyzes existing pertinent hydrologic and related data, delineates the causes of bank erosion on the Connecticut River within the study reach, and compares the various types of erosional forces to the types of erosion found on the river. In addition, remedial measures to control bank erosion are suggested, together with their potential impacts upon the river.

OBJECTIVES OF THE STUDY

The objectives of the study were to delineate the causes of bank erosion on the Connecticut River between Turners Falls Dam in Massachusetts and the headwaters of Wilder Dam in New Hampshire and Vermont, and to determine and evaluate the various erosional forces acting on the banks of the river. In addition, comparison of the various erosional forces with the types of erosion found on the river was made. For each type of erosion, remedial measures are suggested, together with their potential impact on the river. Figure 1 indicates the study reach as defined in conjunction with the watershed area. The erosion sites subjected to detailed study are identified in Appendix A.

Utilizing all available data sources, a comprehensive study from a historical perspective was made from hydrologic, hydraulic, and geomorphic pictures of the river. This study assisted in defining the past and present river dynamic problems within the study reach and has provided an overview of the entire basin. This literature and data overview and analysis aided in defining the data available from the study reach and also allowed recommendations to be made pertaining to additional data requirements for any current work or future maintenance on the river.

To achieve the above objectives, the following detailed study was conducted.

1. All existing data pertinent to the study objectives were collected and analyzed. These data included geometric, hydraulic, hydrologic, soils, hydro-pool operation, aerial and surface photographs, geologic and geomorphic data, and general data gathered from residents with property adjoining the river.

2. The existing tractive force analysis, initially conducted by the U.S. Army Corps of Engineers, New England Division, was evaluated and relationships between tractive force, soil type, and stream bank erosion within the study are delineated.

3. Information gaps and data-deficient areas were determined within the study reach.

4. Reaches of river having similar causative erosion factors were identified.

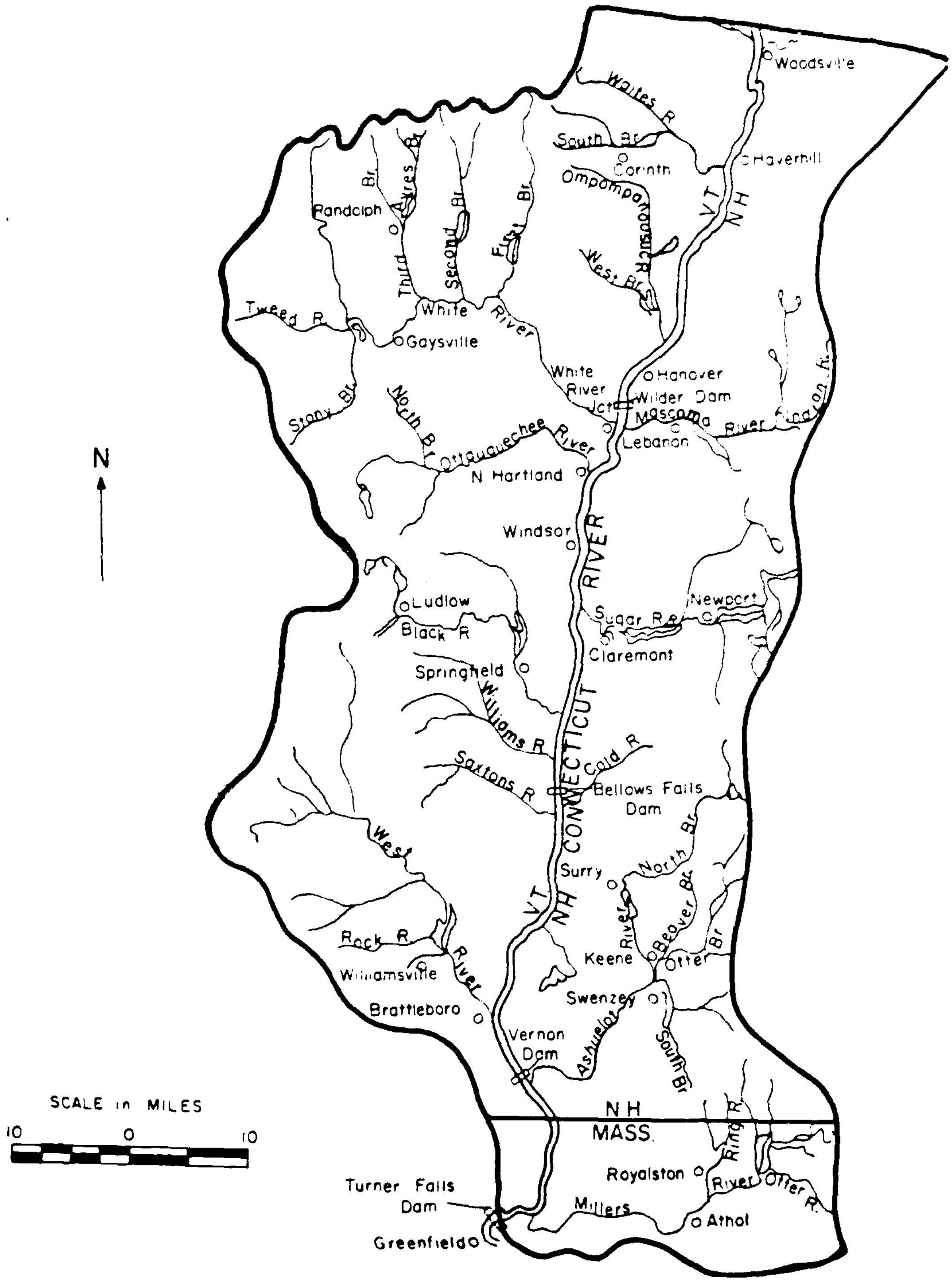


Figure 1. Study reach showing watershed boundaries.

5. Remedial and corrective measures for the various types of erosion found within the study reach were suggested.

6. A brief economic, social, and environmental impact examination of the alternative corrective measures was also undertaken.

SECTION 2

IDENTIFICATION AND EXAMINATION OF THE EXISTING DATA

GENERAL

In order to conduct a complete hydraulic, hydrologic, and sediment movement evaluation of the river, a complete inventory of the study reach was conducted. The broad types of data required for the hydraulic analysis of this river system are categorized as: topographic and hydrographic, geologic and soils, hydrologic, hydraulic and sediment, environmental, and climatological. These individual categories may be further subdivided into specific data categories as described in the following subsections.

Extensive data are available for both the general study area and specific index sites within the 141-mile study reach. Except for detailed quantitative analysis of the sediment routing through the system, the available data are adequate to evaluate the causes of erosion qualitatively. However, it must be stressed that without extensive cross-sectional and sediment data, the causal effects of erosion and the evaluation of comparative variables causing erosion can only be made subjectively. A complete listing of data used in the following analysis may be found in Appendix B, and recommendations regarding additional data requirements that could further delineate causes of erosion quantitatively are presented in Section 9.

TOPOGRAPHIC AND HYDROGRAPHIC DATA

Maps and Charts

Topographic mapping is available for the study area from the states of Massachusetts, Vermont, New Hampshire, and also from the U.S. Geological Survey. Maps from the U.S. Geological survey were used extensively in compilation of river mileages, bank and river form, and general location of four dam sites within the study area. The Corps of Engineers gathered topographic information on a semi-annual basis at the following six index sites:

Area #	Location
147	Newbury, Vermont
51	Hanover, New Hampshire
31	Cornish, New Hampshire
26	Claremont, New Hampshire
301	Dummerston, Vermont
255	Gill, Massachusetts

Partial cross sections at areas 147, 31 and 26 provide excellent representation of the types and extent of bank erosion that occurred between November 1975 and May 1976. These partial sections are shown in Figure 2.

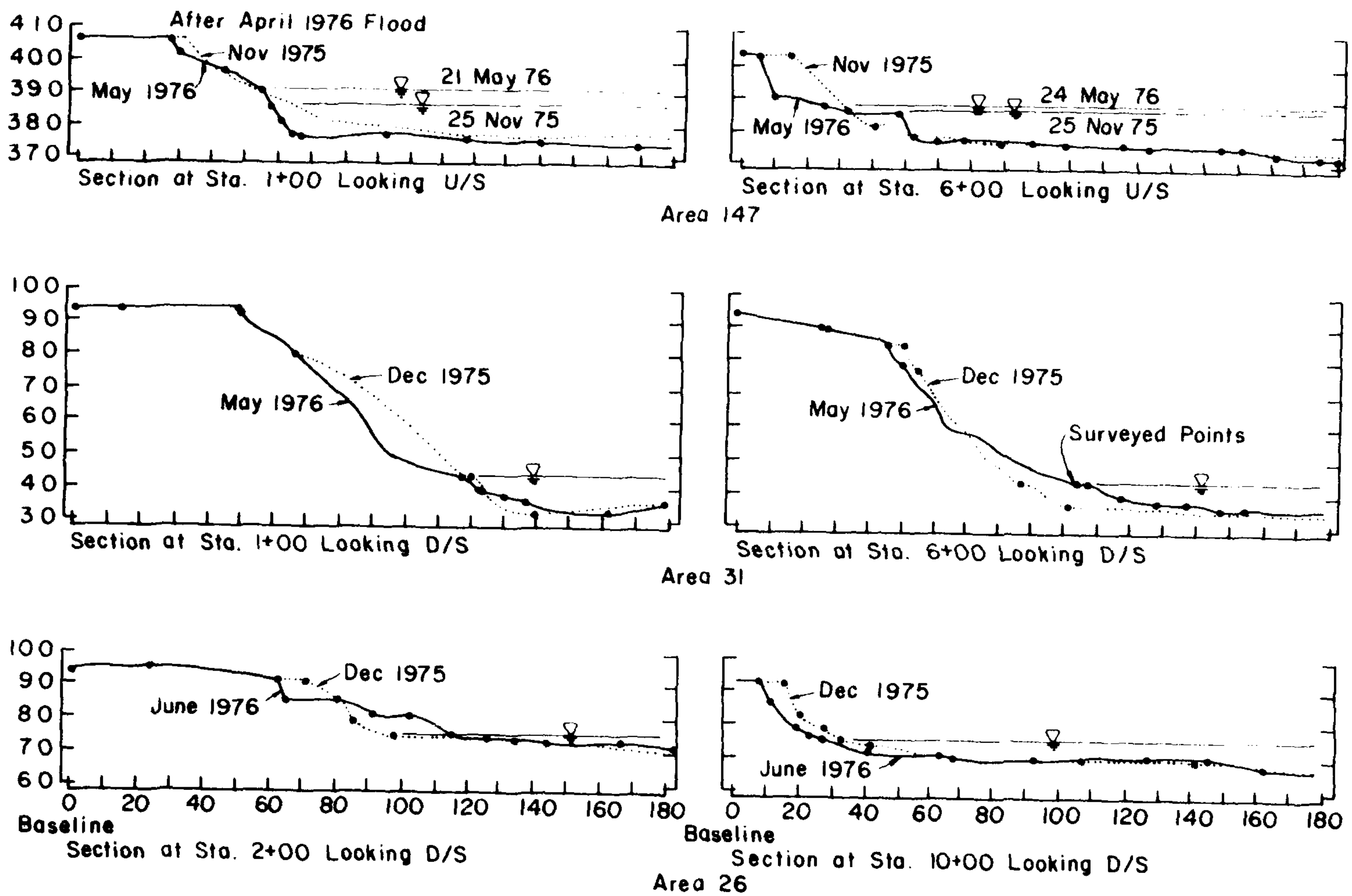


Figure 2. Typical cross sections utilized for tractive force analysis by U.S. Army Corps of Engineers.

Aerial Photos

Very little extensive aerial photo coverage is available for the study area; however, some infrared photographs were available for the reach upstream from Wilder Dam. These photographs were provided by the Cold Regions Research and Engineering Laboratory at Hanover. The Soil Conservation Service throughout New Hampshire and Vermont was also contacted regarding the availability of aerial photographs of the study area. Unfortunately, very little systematic aerial photographic work appears to have been conducted on the study reach, though many local erosion sites have been photographed.

Ground Photos

Extensive ground photography is available from about 1900. The Lyme Historical Society and the Brattleboro Library provided extensive surface coverage and pictorial presentation of the history of floods and structures constructed on the Connecticut River. These photographs provided an excellent source for determining bank conditions during previous years along with some insight into possible changes in the meandering pattern of the river and its degree of stability.

In addition to the ground photos obtained from private individuals and libraries, extensive ground photo coverage on the river itself was provided by the New England Division of the U.S. Army Corps of Engineers. Within the study reach, approximately 154 erosional sites were identified by the Corps, and of these approximately 80 were photographed in detail. Photographs were taken at the index sites semi-annually. These photographs were used extensively in delineating erosion types on the river. Additionally, a majority of these locations were visited over an 18-month period. The last trip to the area was conducted during September 1978.

Design Charts on Existing Structures, Hydro-Power Facilities, Power Dams, Revetments, Diversions, and Outfalls.

Extensive data on these topics were provided by power companies, primarily the New England Power Company. Additional data were obtained from textbooks and historical information on the development of the Connecticut River. Data provided by the power companies were used in determining dam height, pool fluctuations, outflow discharges from their generating turbines, and the original location of the river prior to construction. The preconstruction maps were also used to delineate the location of the bank erosion and to assist in determining the effect of the pools upon the various types of bank erosion.

GEOLOGIC AND SOILS DATA

Extensive geologic, geomorphic, and soil data are available for the study reach from the U.S. Geological Survey (USGS) and the Soil Conservation Service. A detailed description of the geology and flood plain soils types at the lower end of the study reach was provided by R. H. Jahns (1974). The geology of the Connecticut River Basin can be subdivided into two distinct periods. The first period is prior to continental glaciation and the second period follows continental glaciation.

Pre-glacial history of the Connecticut River is quite diverse. Bedrock of the area consists of heavily folded and faulted metamorphic and igneous rocks. The metamorphic rocks include phyllites, schists, and gneisses. The igneous bodies are granite, granodiorite, and quartz monzanite with occasional intrusions of volcanic materials. The trends of major structural features in Vermont and New Hampshire are in a north-northeasterly direction. This coincides with the Connecticut River which probably follows an ancient drainage way.

Pre-glacial geology .ndicates extensive periods of erosion associated with the uplift of the Appalachian Mountains near the close of the Paleozoic Era, and with other periods when the land was emergent. It is assumed that the present topography was well established prior to continental glaciation, including a well-developed soil layer with superimposed streams including their meandering patterns.

Massive continental glaciation wore the topography into the currently existing subdued forms. Highlands were rounded on the upper side facing the glacier and steepened on the lower side away from the glacier. Stream valleys were eroded and smoothed, sometimes into the classic V-shaped glacial valleys. Ice fronts, lakes and floods also altered the erosional features during and following glaciation.

The retreating ice redeposited morainal materials over the entire surface of the area. Stagnant ice blocks and frontal moraines created lakes that became sites of further deposition. The Connecticut River within the study area may be subdivided into about 10 depositional areas. Some of these areas are characterized by morainal deposits while others are characterized by a combination of lake and morainal deposits. These basins have been partially formed by water, ice erosion and deposition. Between some of these basins the river flows through relatively narrow gorges where bedrock controls predominate. This is particularly evident in the Bellows Falls area.

Glacial and post-glacial deposits have a tremendous influence on river form. Where basins exist, the river formed in glacial and floodplain deposits consisting of sands, silts, and boulders. However, in some areas, clay deposits are present and the relatively non-cohesive nature of this material allows the river to move across the glaciated plane where sufficient area is available. In some areas the tributary streams influence river form by pushing the river against the opposite banks from the confluence. Generally, the geology of the Connecticut River influences the river form significantly through structure and bedrock conditions and as a consequence of glacial and post-glacial activity.

The Soil Conservation Service also provided extensive data describing the type of alluvial material forming the floodplains adjacent to the river. Additionally, they described the type of material deposited during the recession of floods when water covered extensive portions of the floodplain. The data also included sieve analysis. These data have provided insight into the type of material that the river is capable of carrying during periods of flood and the source of these materials from the upper valley areas.

In addition, soil core samples were collected and analyzed by the Corps of Engineers at the index sites where topographic information was taken. In each case the bank material primarily consists of stratified, non-cohesive fine sands and silts.

HYDROLOGIC DATA

Discharge Records

Discharge data are available for the study area including some of the tributaries above Turners Falls Dam. The primary discharge stations are shown in Table 1.

Table 1. Gaging station location, identification, and period of record.

STATION LOCATION	STATION NUMBER	PERIOD OF RECORD
Wells River, Vermont	01138500	1949 to 1977
White River Junction, Vermont	01144500	1911 to 1977
North Walpole, New Hampshire	01154500	1942 to 1977
Vernon, Vermont	01156500	1944 to 1977
Turners Falls Dam	01167000	1915 to 1977

These discharge records were used as the primary source of information for the tractive force analysis and also for weighing the erosive forces causing the various types of erosion within the study reach. Average annual hydrographs compiled from the available records have been prepared. Figures 3 through 7 show the similarity in the shape of average annual hydrographs at the five gaging stations listed in Table 1.

A 1974 analysis by the U.S. Army Corps of Engineers, New England Division, at site 147, analyzed the average annual, average summertime, and annual peak discharges for 1959 through 1974. Results show that from 1964 through 1968 both the average annual and average summertime flows were low relative to their respective flows between 1969 and 1974; obviously a result of the mid-1960's drought. Although the annual peak flows did not exceed 35,000 cfs between 1959 and 1968, this flow rate has been exceeded considerably on four separate occasions. Plots of the results are presented in Figure 8. Both the normal and the erosive peak discharges have increased considerably since 1969.

Stage Records

Stage records for the above discharge stations provided additional data on overbank flow. Also, the stages recorded at the power-generating dams within the study reach provided additional information on both stage and variation of stage with time.

Example plots of pool stage versus time at the four power dams are presented in Figure 9. The curves indicate that the Vernon, Bellows Falls, and Wilder Pools fluctuate approximately one foot per day. However, the Turners Falls pool fluctuation is out of phase with the upstream pools and has a magnitude of fluctuation of 3.5 feet per day. The magnitude of fluctuation is much larger if a shorter time period is considered (Figure 9).

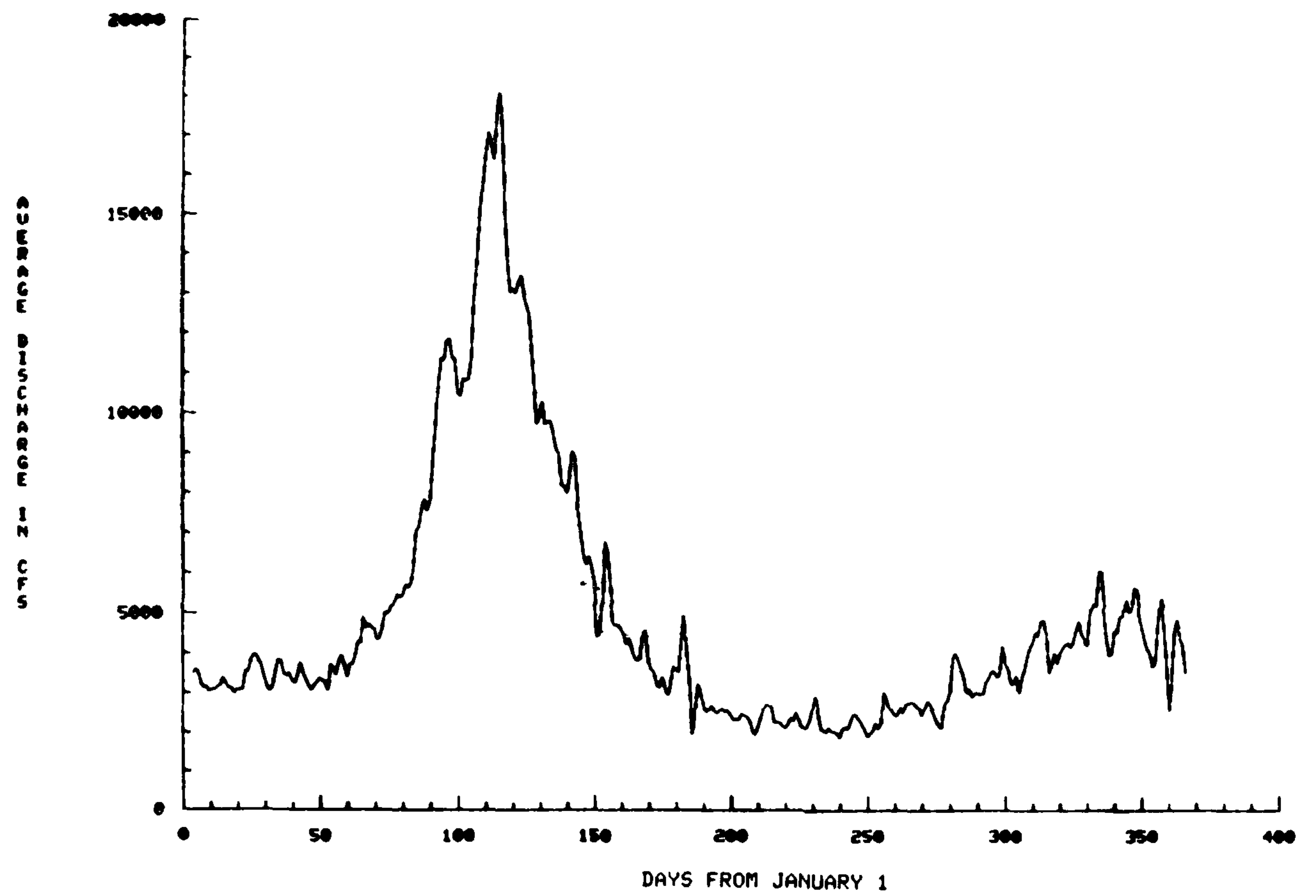


Figure 3. Average annual hydrograph for Wells River, Vermont.

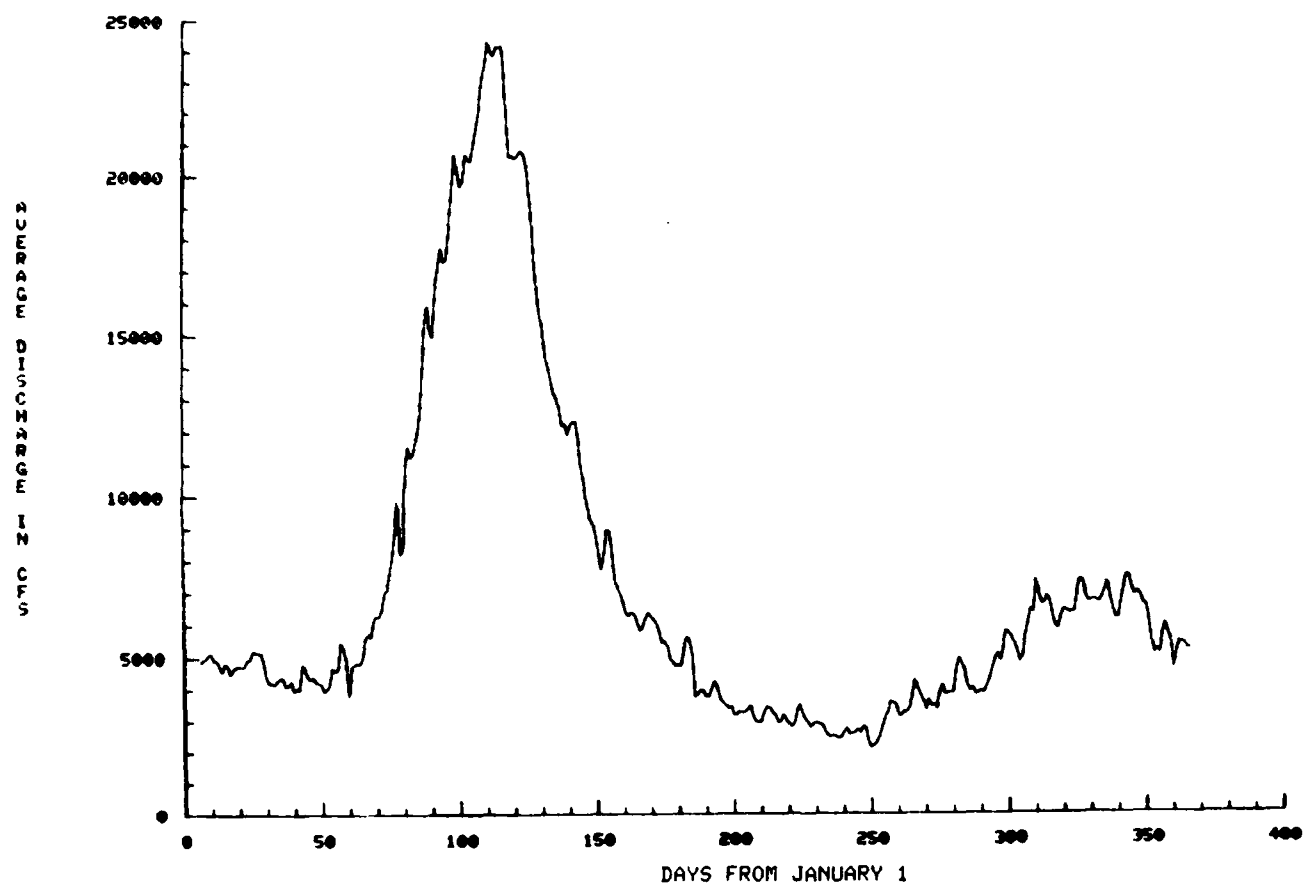


Figure 4. Average annual hydrograph for White River Junction, Vermont.

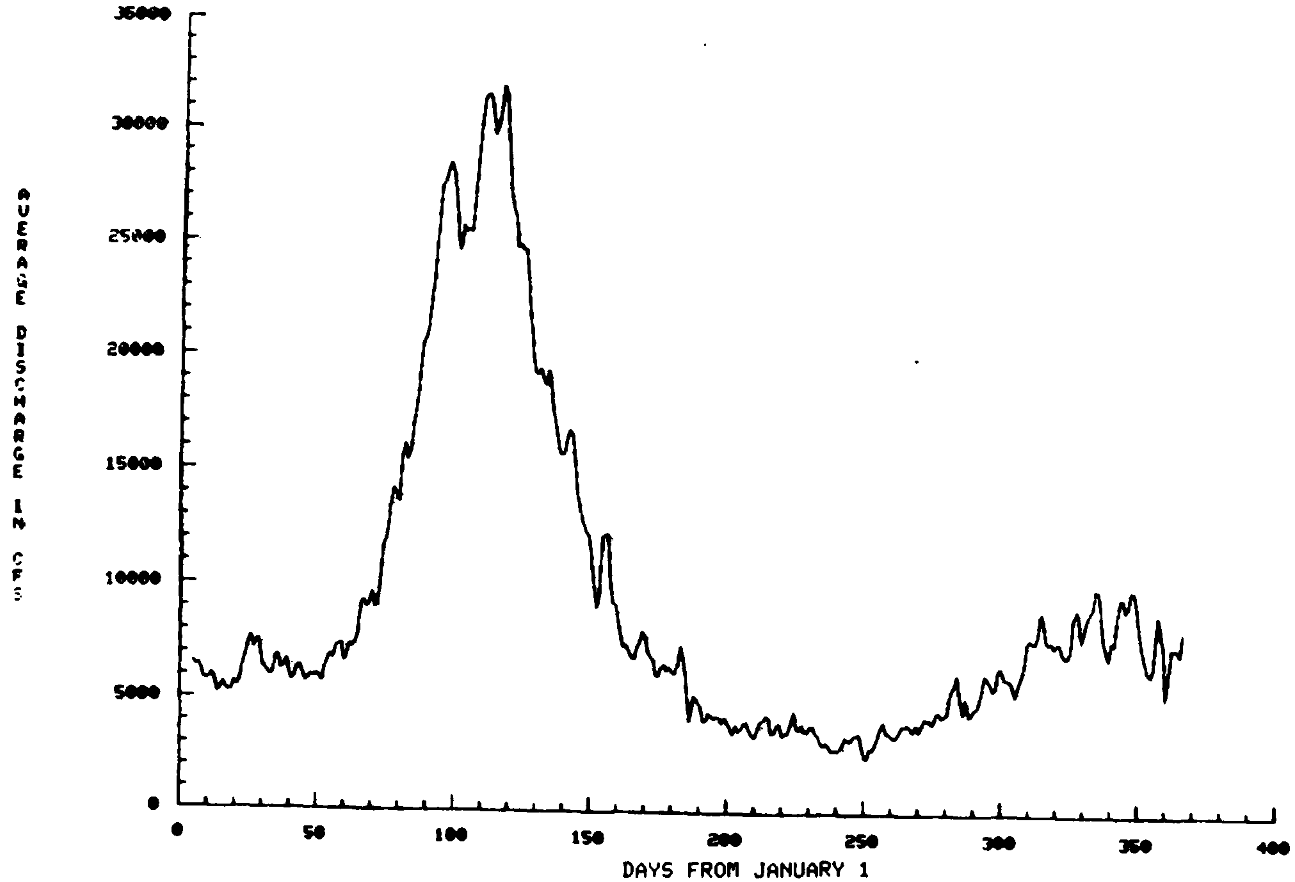


Figure 5. Average annual hydrograph for North Walpole, New Hampshire.

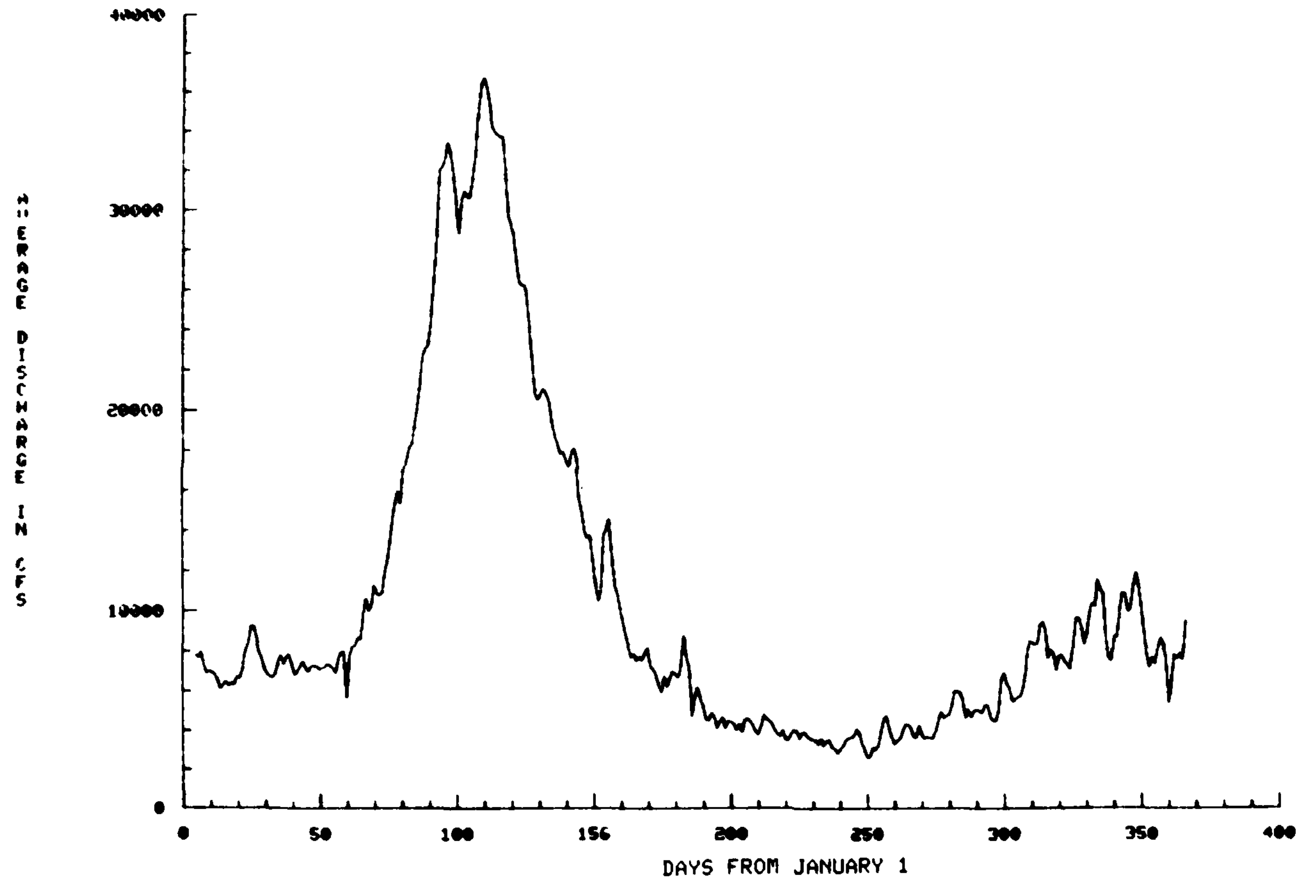


Figure 6. Average annual hydrograph for Vernon, Vermont.

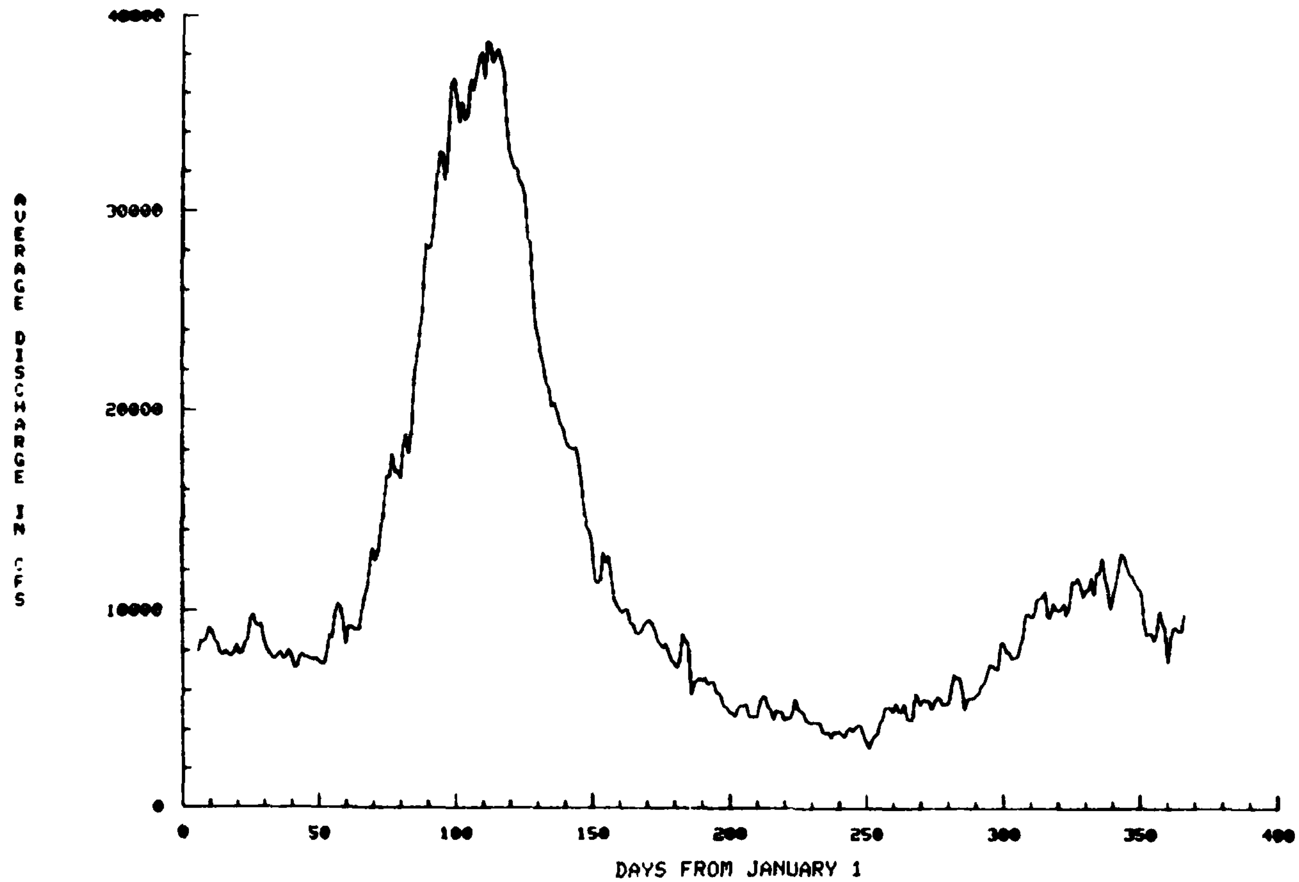


Figure 7. Average annual hydrograph for Turners Falls Dam.

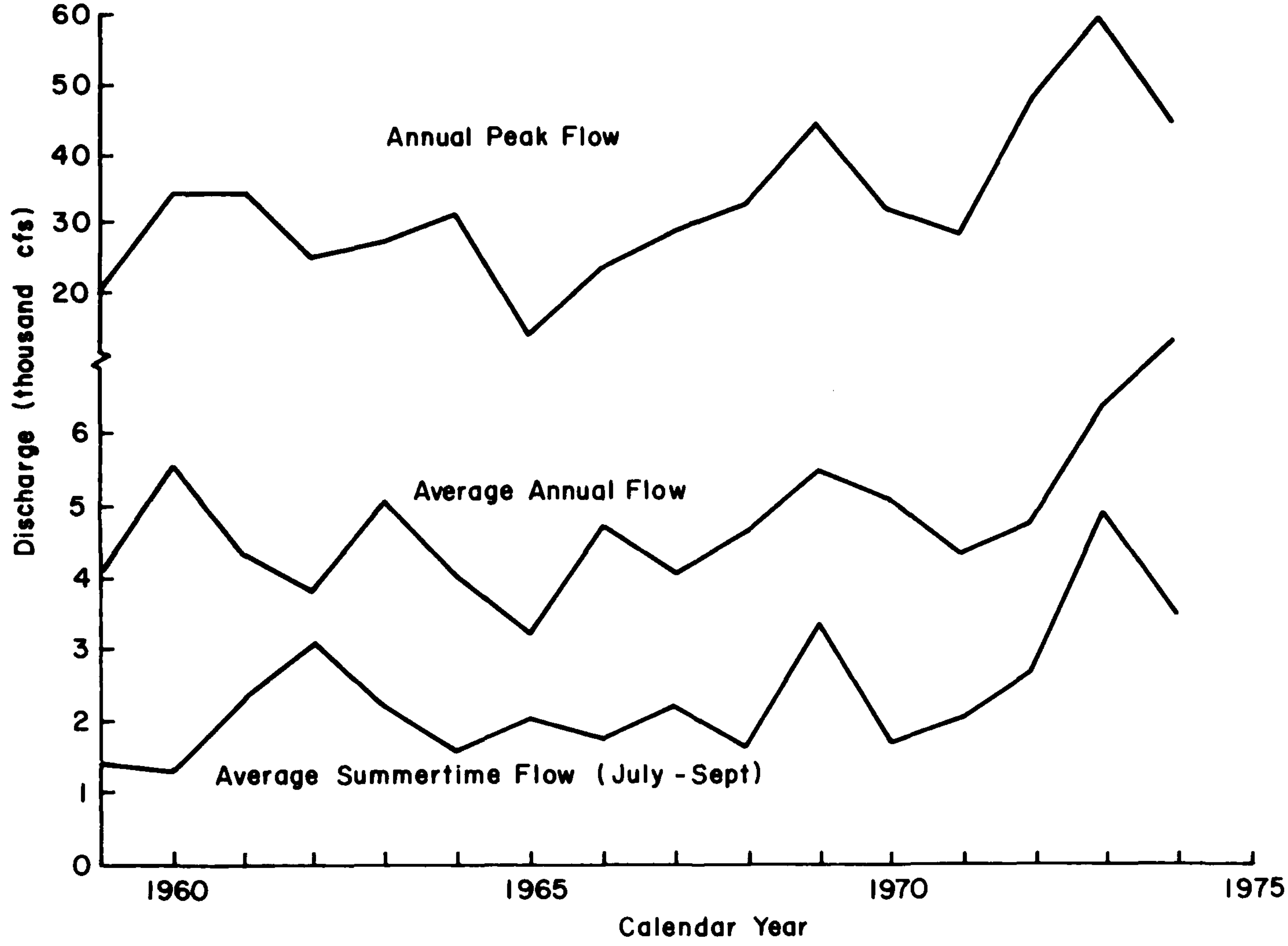


Figure 8. Connecticut River flows at Wells River, Vermont.

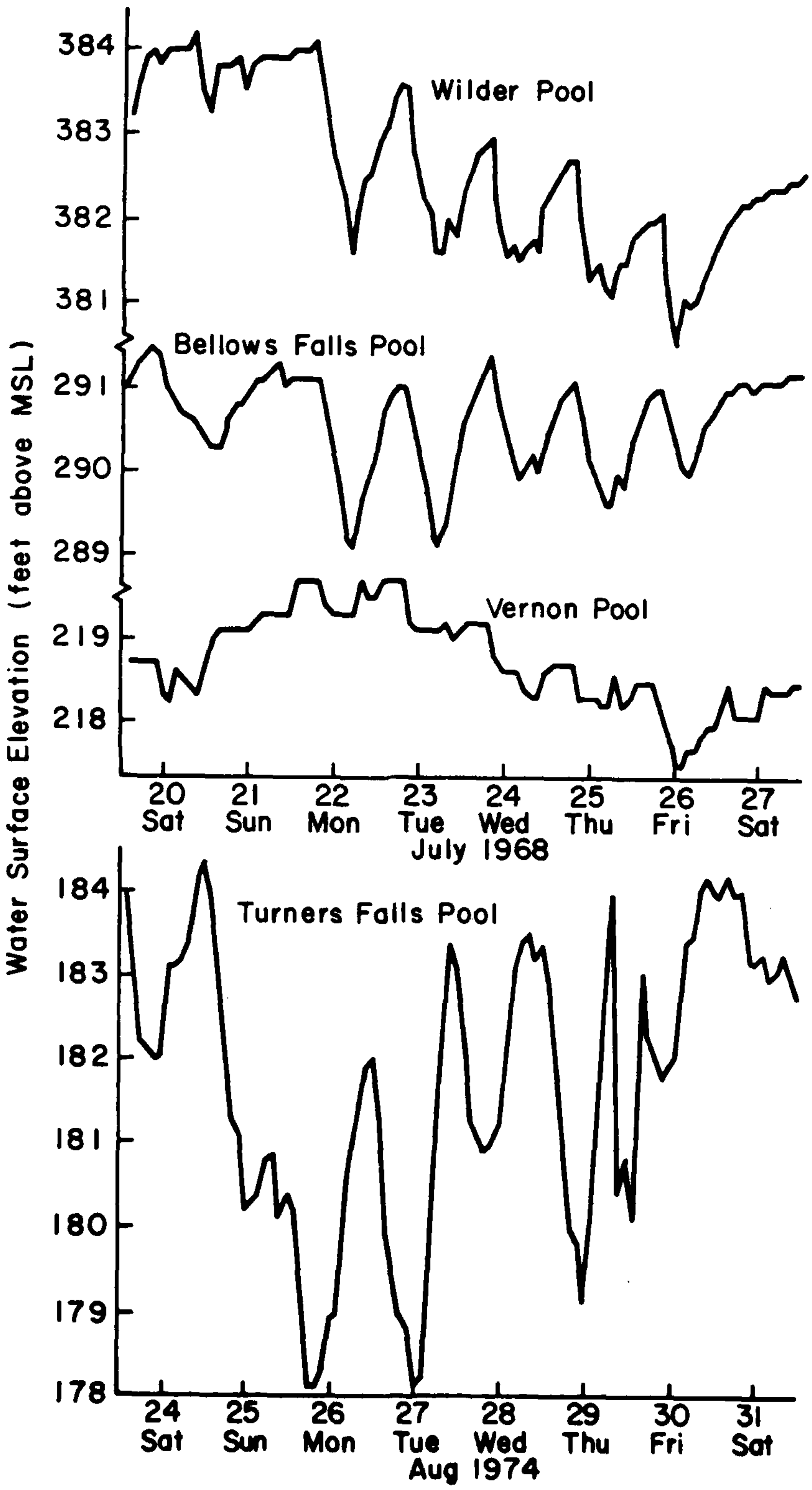


Figure 9. Pool stage compared to time at the four study reaches.

In a hydraulic analysis, the New England Division of the U.S. Army Corps of Engineers (1976) applied an unsteady flow model to evaluate the pool stages with time for the Wilder Pool during a typical period of operation. The period from November 1 through 15, 1972 was analyzed because it included both continuous low flow and high flow periods. In addition, the U.S. Geological Survey had maintained three water level recording gages on the Wilder Pool during that period. Data from these gages were used to calibrate the function coefficients used in the computer model. Pool profiles at two-hour intervals were calculated using this program and are plotted in Figure 10. In addition, pool stages at Wilder Dam have been recorded by the New England Power Company since construction of the project in 1952. Wilder Dam makes flow releases during weekday power demand periods, usually between 1100 and 1800 hours, and closes for the remainder of the day and on weekends. Figures 11 and 12 show the average weekday pool fluctuations during the months of July and December for several three-year periods. Because hydraulic conditions are more severe in Turners Falls Pool additional background data are provided.

Turners Falls Pool*

Turners Falls Pool is approximately 20 miles long, extending from Turners Falls Dam in Massachusetts northward along the Connecticut River to Vernon Dam in Vermont. The main tributaries to the pool aside from the Connecticut River are the Millers and Ashuelot Rivers located approximately 4 and 18 miles above Turners Falls Dam, respectively. The tailrace of the Northfield Mountain Pumped Storage Project is located approximately 5 miles above Turners Falls Dam. Normally, inflows due to Vernon Dam discharges vary between 1,000 cfs and 10,400 cfs during off-peak and peak power demand periods respectively. The Northfield Mountain Project withdraws water from Turners Falls Pool at a maximum rate of 12,000 cfs during low demand periods and discharges at a maximum rate of 18,000 cfs during peak demand periods. Discharges at Turners Falls Dam range from near zero to about 10,600 cfs during low and high demand periods respectively. Inflows from the Miller and Ashuelot Rivers vary from a few hundred cfs to a few thousand depending on time and type of year. These variations in flow cause a very dynamic situation to exist in Turners Falls Pool that significantly affects bank erosion.

The Corps of Engineers studied several periods of documented historic flows using the "Gradually Varied Unsteady Flow Profiles" program to help form a basis for evaluating erosion processes.

During the low flow periods of July 17-22, 1976, conditions in the pool were less dynamic than at other times. This was due to the limited generation or pumping that occurred simply because usable water was not available. During Northfield pumping operations negative velocities were computed from the Northfield tailrace to Turners Falls Dam, the maximum being -0.25 fps near the tailrace with velocities becoming much less nearer to Turners Falls Dam. Average velocities upstream from the tailrace were increased during pumping but only reached a maximum of 0.46 fps. Average velocities of this magnitude are not associated with significant erosion. During generation at Northfield, flows downstream of the tailrace were nearly double those upstream. The maximum average velocity, however, was 2.81 fps, which is considered quite small. Maximum pool fluctuations during this period were about 2.5 to 3.0

*Details regarding the physical system, flows and pool changes were provided by U.S. Army Corps of Engineers, New England Division.

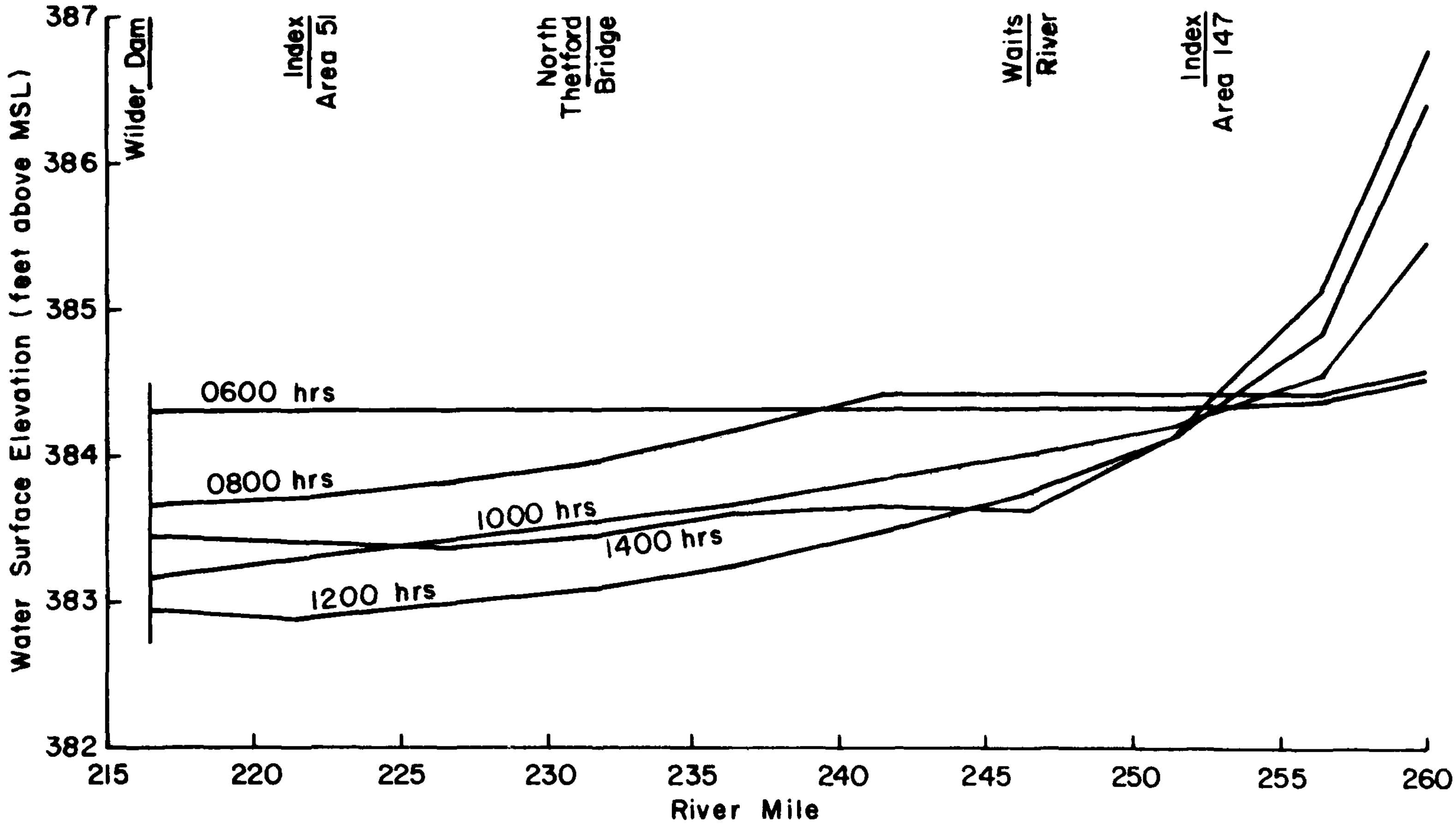


Figure 10. Wilder Dam operation showing pool fluctuations at the dam and water surface profile along the pool.

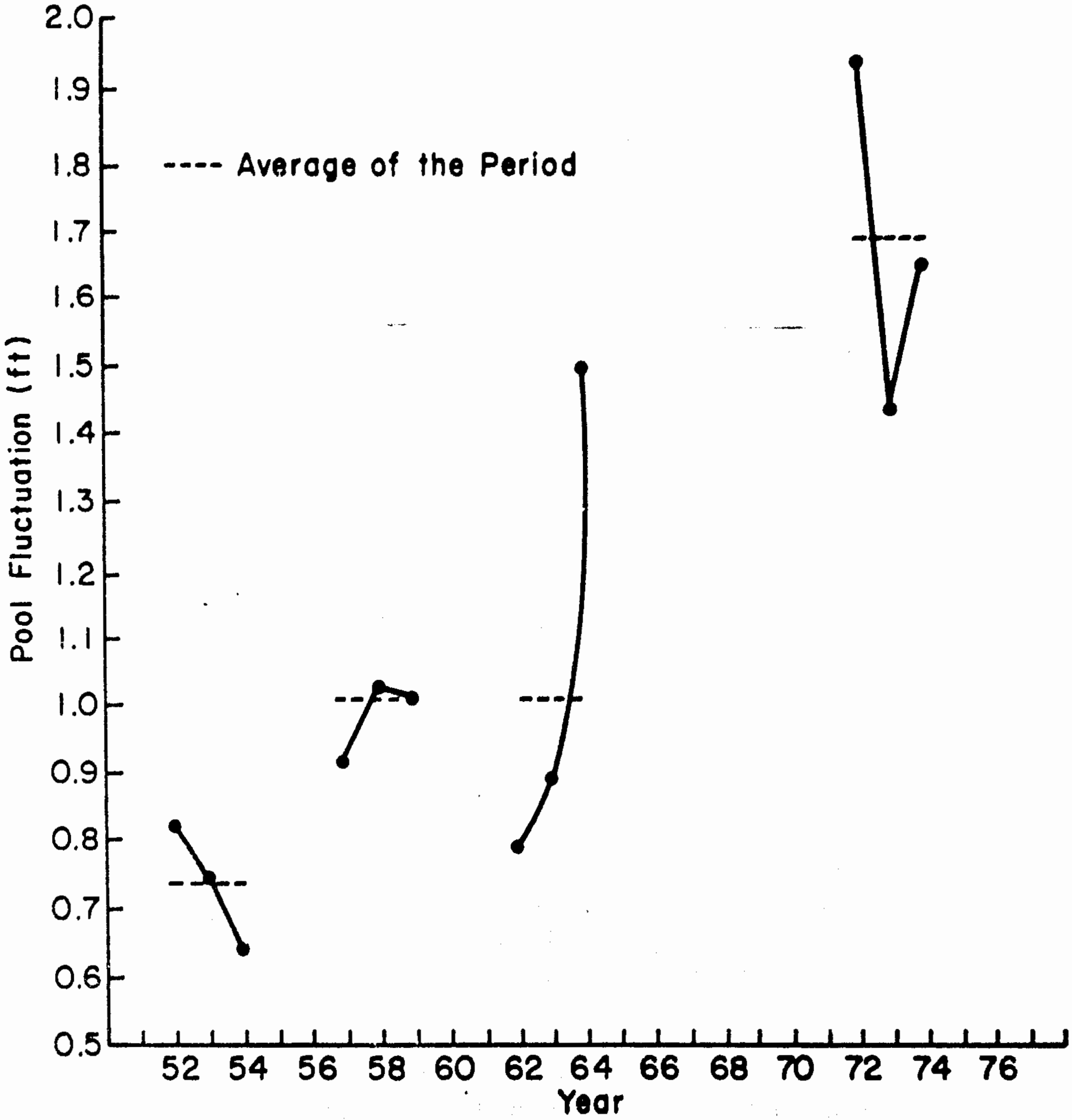


Figure 11. Average weekday pool fluctuations at Wilder Dam for July.

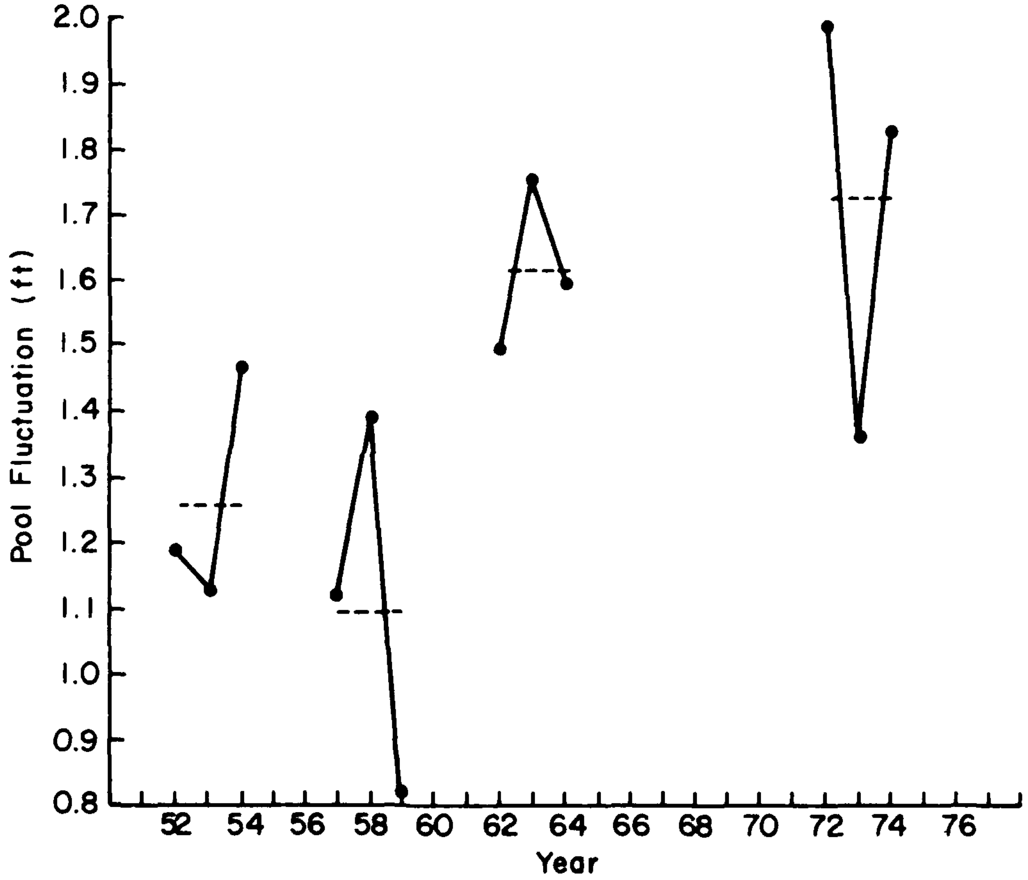


Figure 12. Average weekday pool fluctuations at Wilder Dam for December.

feet. Localized eddy action occurred in the tailrace area as was predicted by the physical model study of Turners Falls Pool prepared by Alden Labs of Worcester Polytechnic Institute.

The normal to moderate flow period of September 2-7, 1974 seemed to be similar to the previous low flow period. Average velocities were slightly higher with a maximum average velocity of 3.12 fps being calculated during power generation periods. Maximum average velocities downstream and upstream of the tailrace during pumping operations were -0.70 and 0.61 fps, respectively. These velocities are not associated with scour erosion but do indicate that eddy currents and other patterns formed in the tailrace. A maximum pool fluctuation of over 5 feet was observed on September 3rd. During this period the pool elevation seemed to undergo its most dynamic fluctuations.

The typical spring runoff period of April 3-8, 1974 was also examined. During this time interval the Turners Falls and Vernon Dams operated basically on a run-of-the-river basis. Therefore, their effect on flows is minimal. The Northfield plant had the most water available for pumping at this time so considerable water was pumped. Generation also occurred but at lesser rates. Tractive force analysis was performed at area 255 by the Corps of Engineers using the same methods utilized at Wilder Dam. With a maximum discharge of 65,500 cfs (1.6 year return period event) they computed a local tractive force of 0.09 psf. Using an allowable value of 0.075-0.10 psf, it is readily apparent that tractive force erosion would occur at vulnerable sites during the spring freshet event. Pumping operation of Northfield Mountain during this type of event probably added to the tractive erosion force exerted on the banks upstream of the tailrace with an opposite effect in the downstream direction.

The largest typical annual fluctuation in pool elevation occurs during the spring event. For this particular event the water elevation increased from 179.3 feet msl at 0600 on April 3rd to approximately 190.4 feet msl at 1200 on April 6th.

Turners Falls Dam was raised by 5.5 feet in 1971 as a part of the Northfield Mountain Project. Prior to that time it operated similarly to the three upstream dams. Conditions have dramatically changed since completion of this project. Soils that were rarely wet are subject to frequent inundation. Pool fluctuations and variations in discharges and velocities have increased. In fact, the entire hydraulics of the system has changed.

STAGE-DISCHARGE RELATIONSHIPS

Stage-discharge relationships are available and were studied only to identify how discharge varies and possible changes with time.

FLOOD FREQUENCY AND DURATION CURVES

Using the above discharge records, flood frequency curves were computed for the five stations with discharge records.

The flood frequency curves were developed utilizing the conventional method of selecting the peak discharges for each year, ranking these discharges in descending order of magnitude and then determining the return period (Yevjevich, 1972). The resulting frequency curves are slightly different compared to those determined by the U.S. Army Corps of Engineers, which are based on the Log-Pearson type 3 analysis as specified by the Water Resources Council, 1976. The difference is due to the 11 years of additional data utilized in this analysis (1966 to 1977) and adjustments made in the Corps data considering the effect of reservoirs. Flood control projects were assumed removed from the watershed system in the Corps' analysis to represent preconstruction conditions ("Comprehensive Water and Related Land Resources Investigation, Connecticut River Basin," Vol. II, Appendix C). Data supplied by the USGS that were used in this report have not been adjusted in the above manner. A comparison of the variation in frequency values for two stations follows.

		Peak Discharges (cfs)	
Station Location	Return Period (yrs)	Adjusted Data (Corps)	Unadjusted Data
White River Junction, Vermont	50	110,000	92,000
Vernon, Vermont	20	128,000	97,000

The determined frequency curves are presented in Figures 13-17. In addition, flow duration curves were prepared for the five reported gaging stations having discharge records.

These determined curves included in Figures 18-22 serve as an indicator of the magnitude and duration of the velocities and tractive forces that occur.

The effect of reservoirs on flood frequency in the Connecticut River system is significant, since there are 16 in the system. The flood frequency curves at White River Junction and at Vernon under natural and reservoir conditions are given in Figures 23 and 24. The exceedence probability is the probability at which the peak discharge is greater than or equal to the selected value. In addition, Figures 25 and 26 show the effects along the river for the March 1936 and June and July, 1973 floods. These figures show the general effect is increasingly significant along the downstream direction. Furthermore, the effects are relatively close for the 1936 and 1973 floods.

CLIMATOLOGICAL DATA

Climatological data within this report are limited to precipitation and air temperature. Due to the extensive discharge data available for this study, limited analysis of precipitation was conducted to determine potential watershed yields within and above the study area. However, extensive rainfall monitoring on a long-term basis is available and could be used if required for the determination of lateral sediment and water inflow from the tributaries and adjacent watersheds.

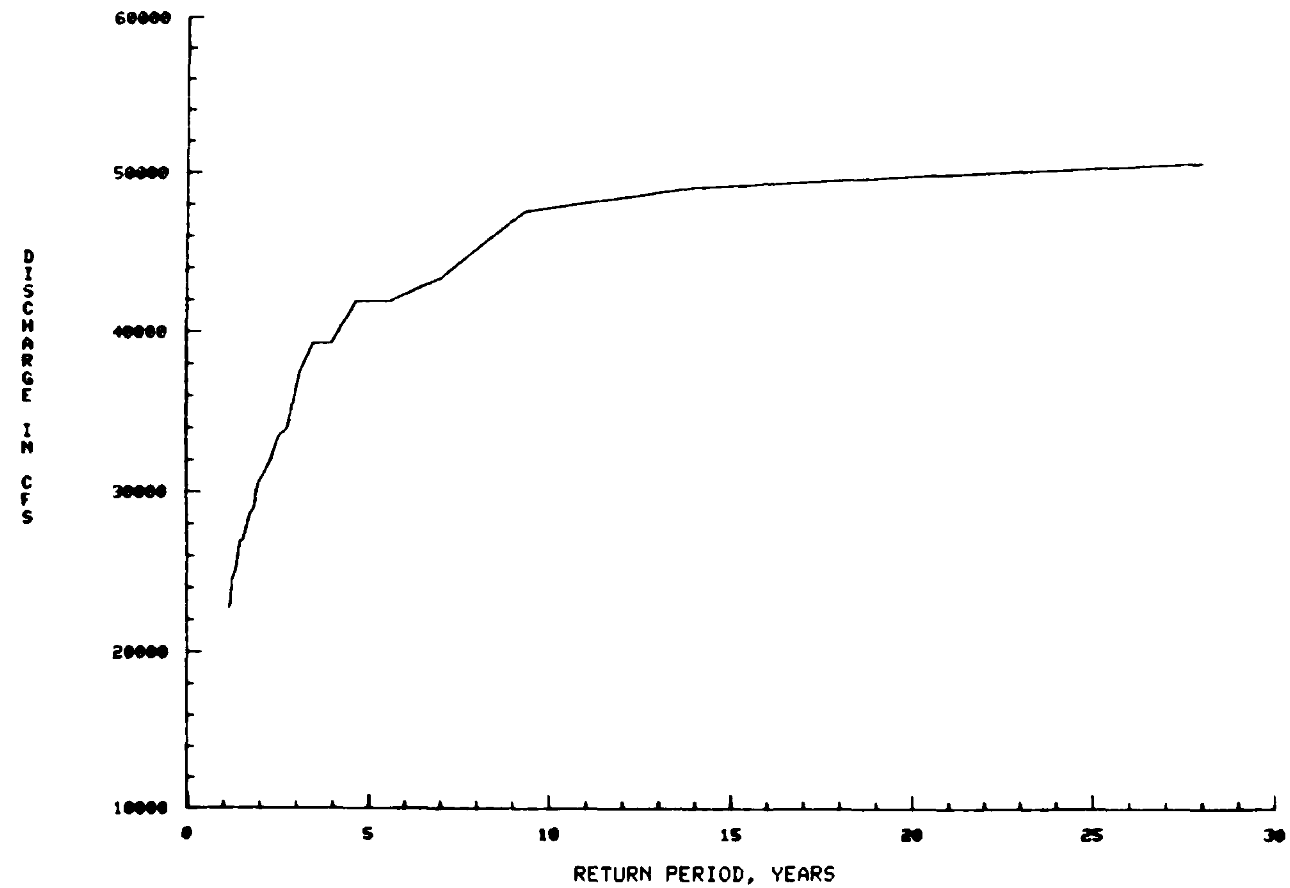


Figure 13. Flood frequency curve for Wells River, Vermont.

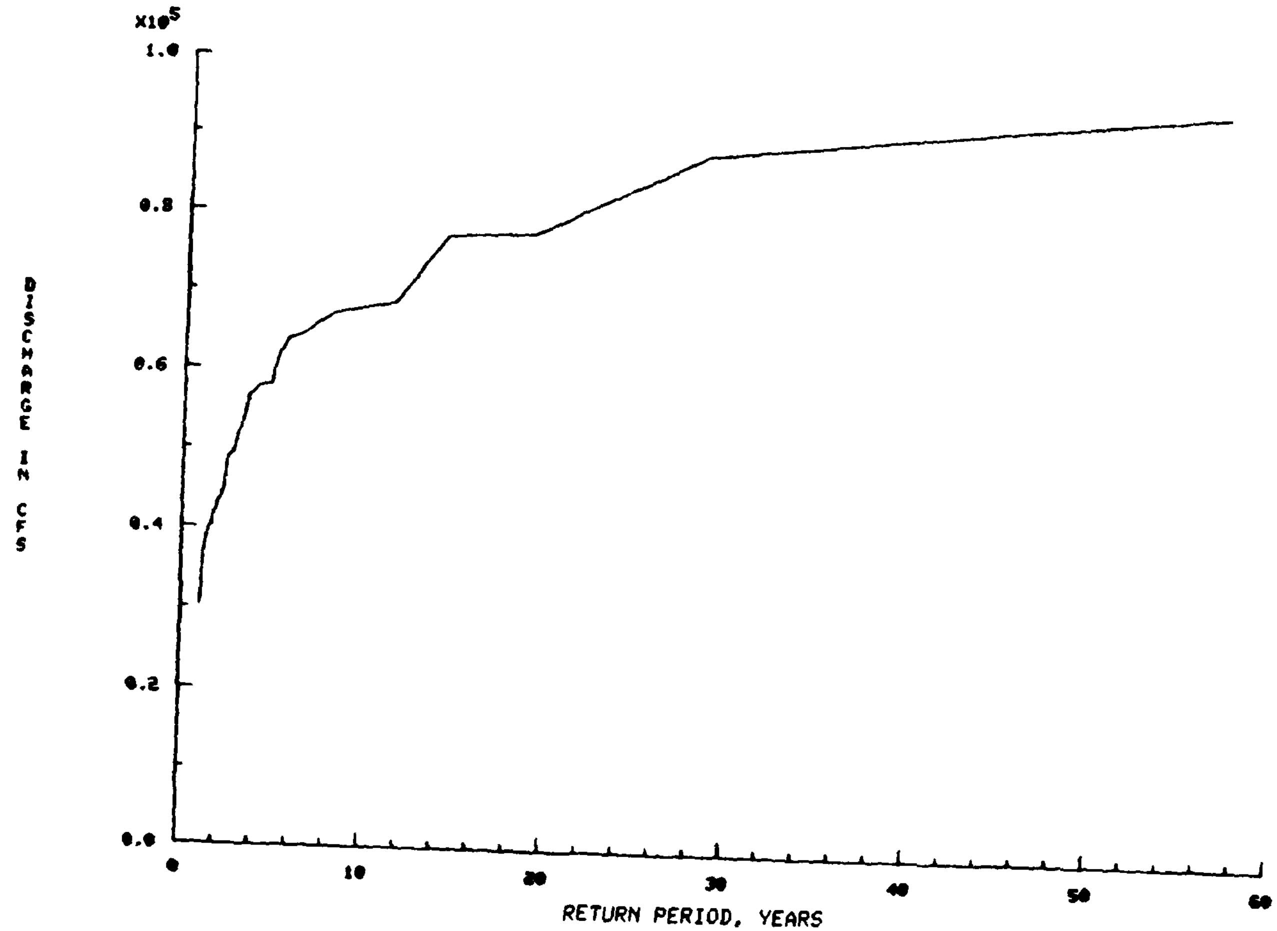


Figure 14. Flood frequency curve for White River Junction, Vermont.

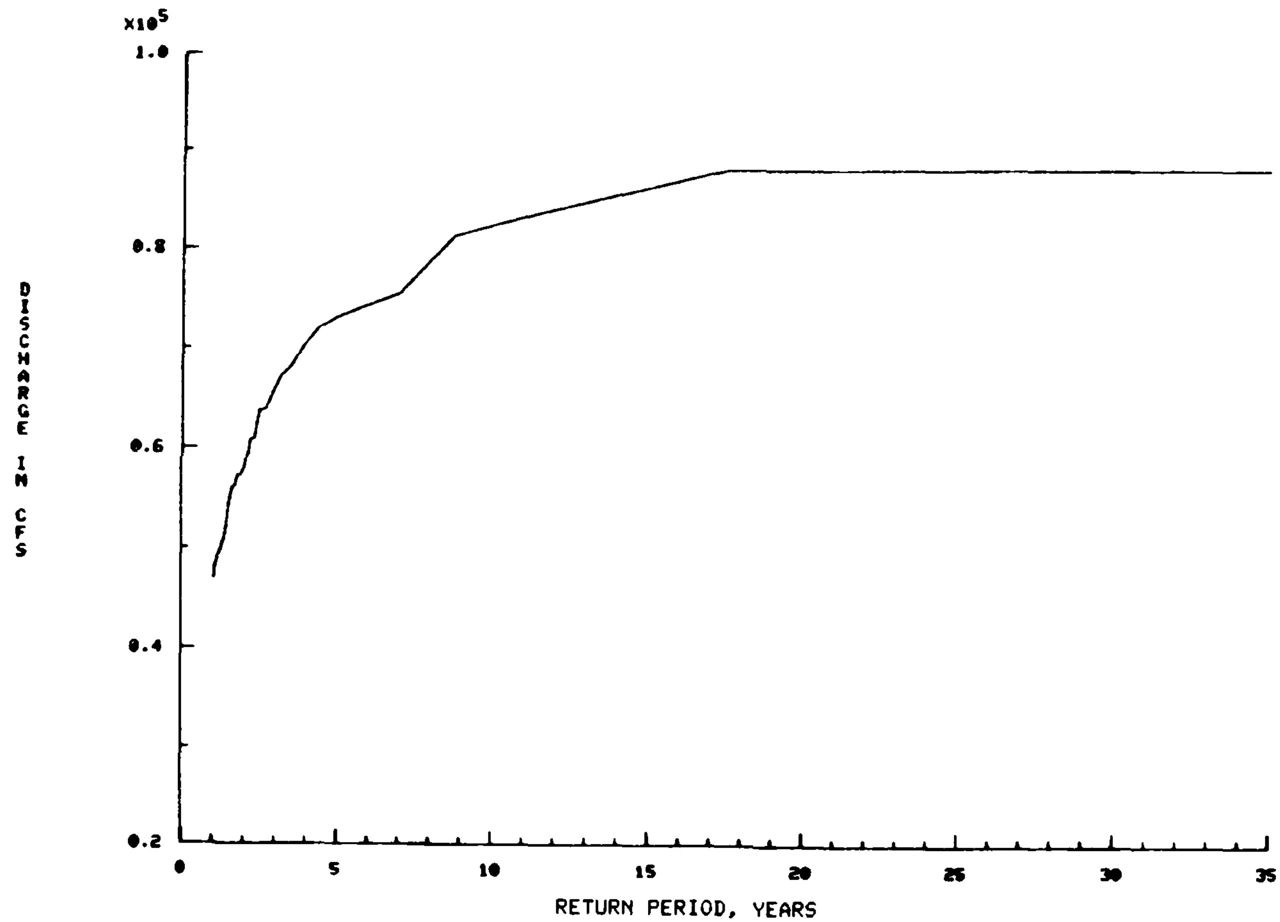


Figure 15. Flood frequency curve for North Walpole, New Hampshire.

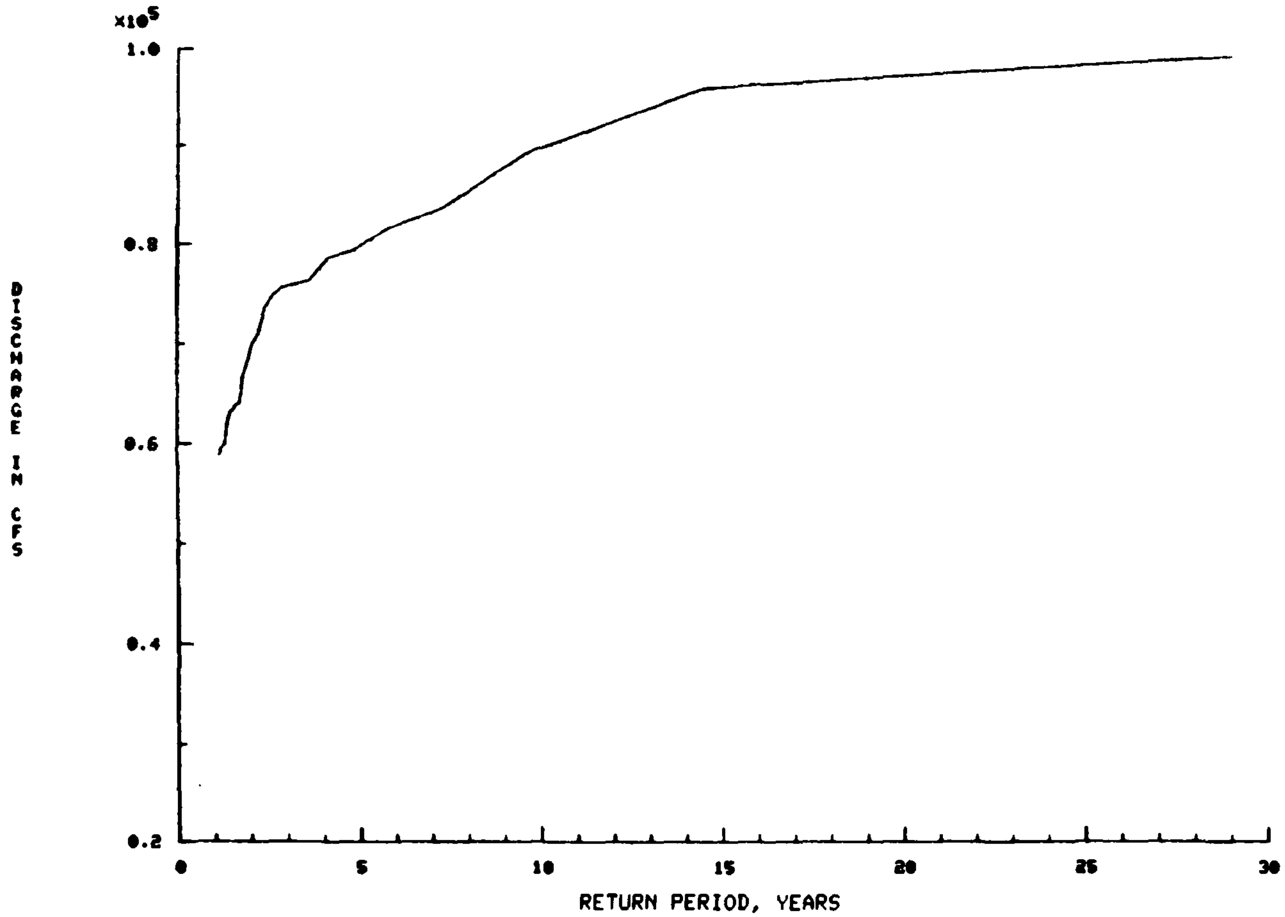


Figure 16. Flood frequency curve for Vernon, Vermont.

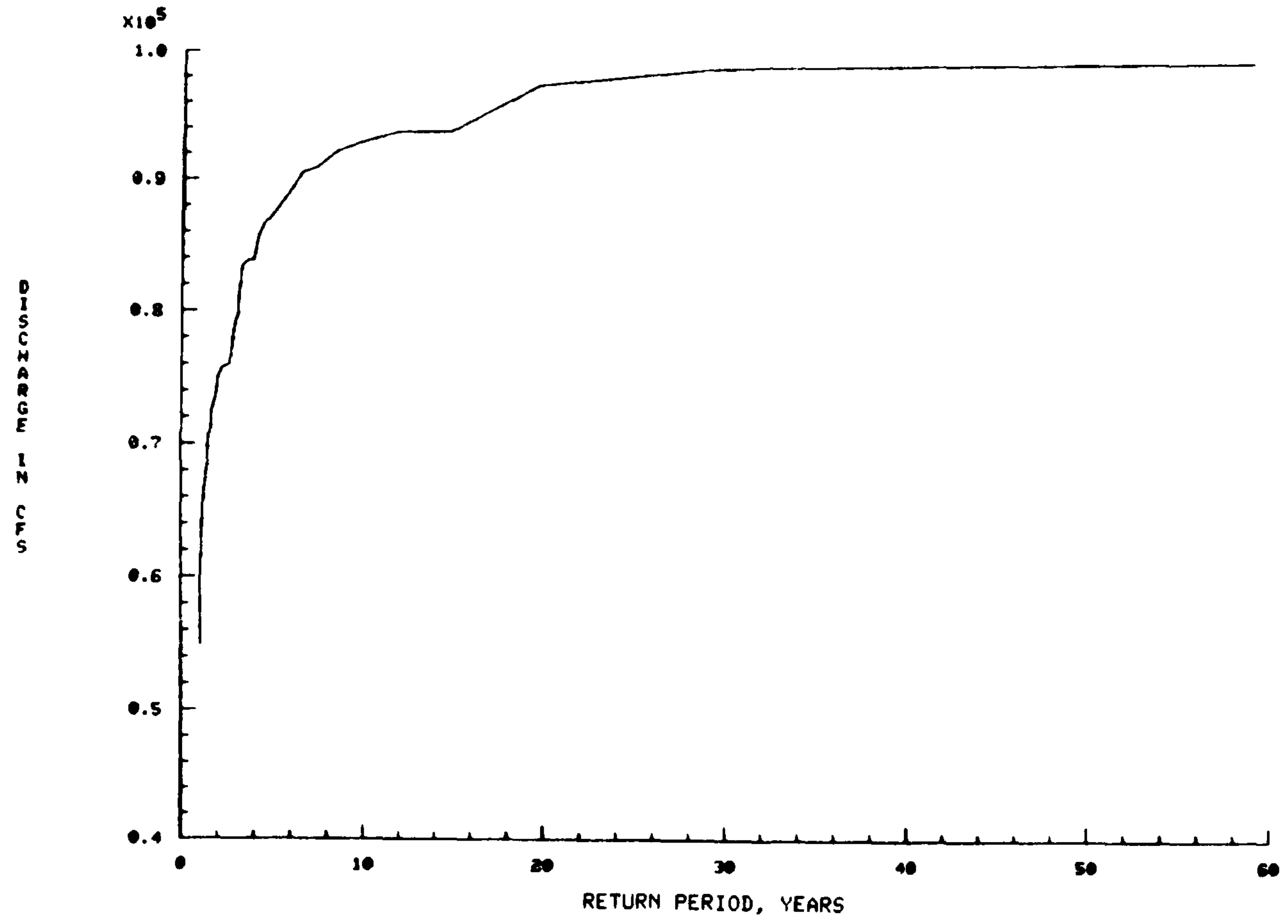


Figure 17. Flood frequency curve for Turners Falls Dam.

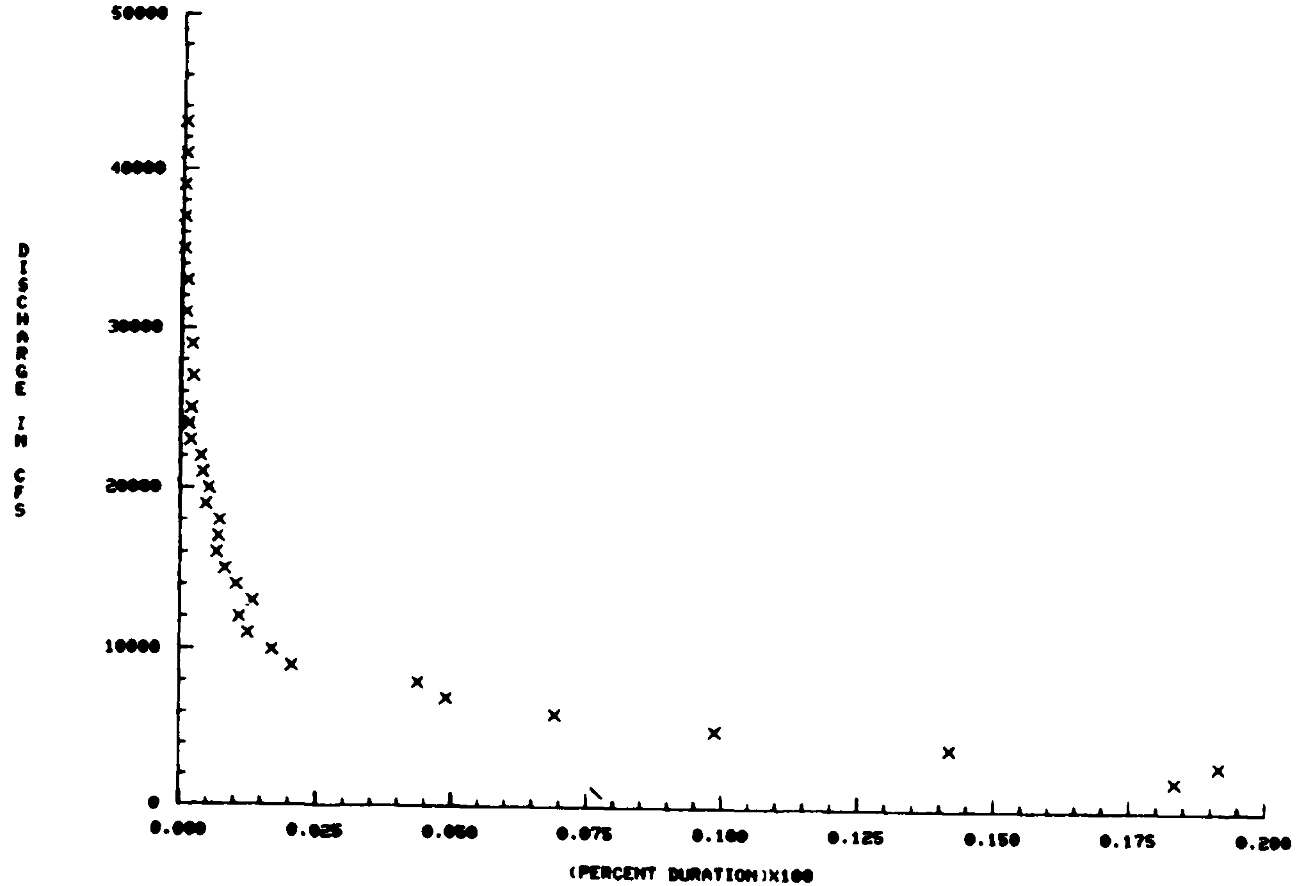


Figure 18. Flow duration analysis for Wells River, Vermont.

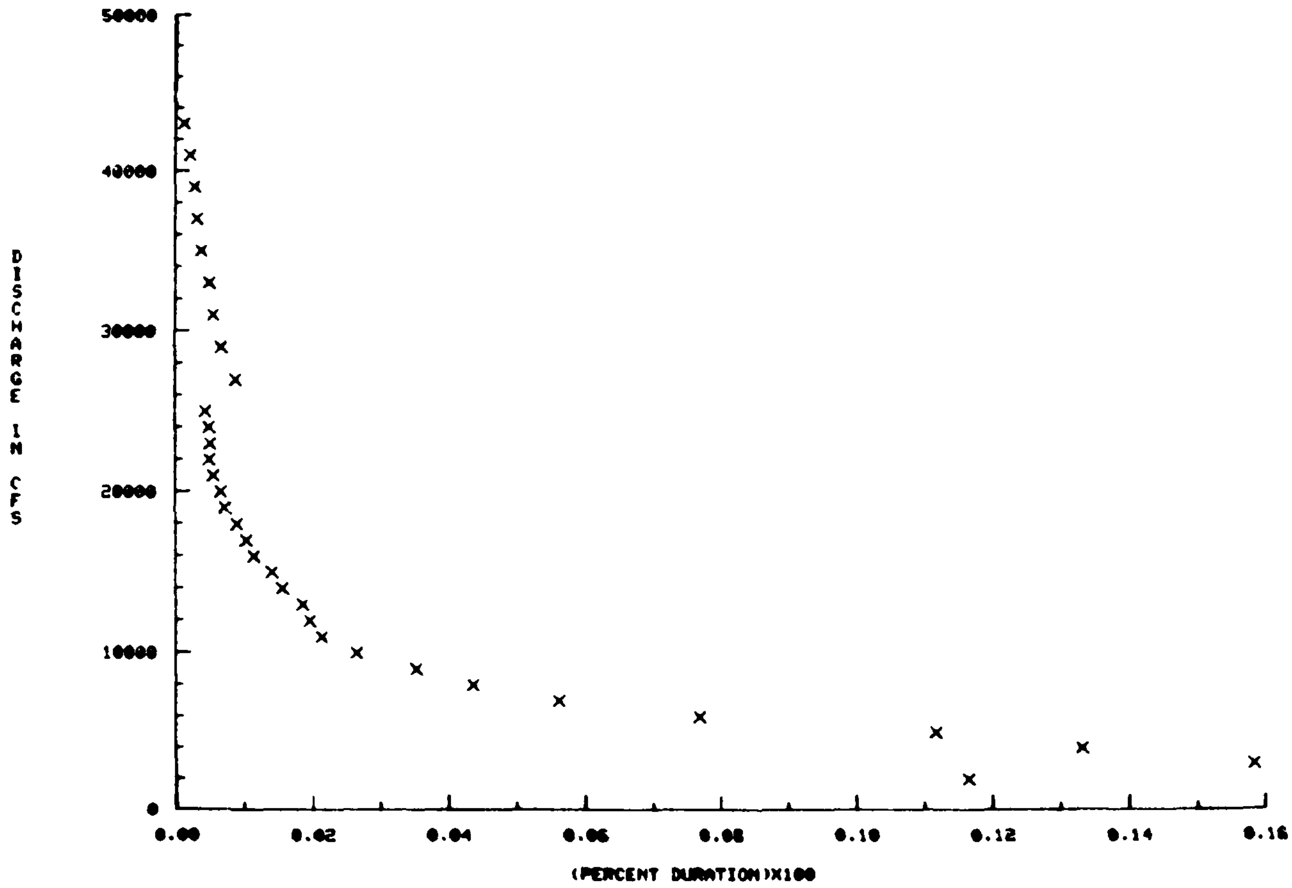


Figure 19. Flow duration analysis for White River Junction, Vermont.

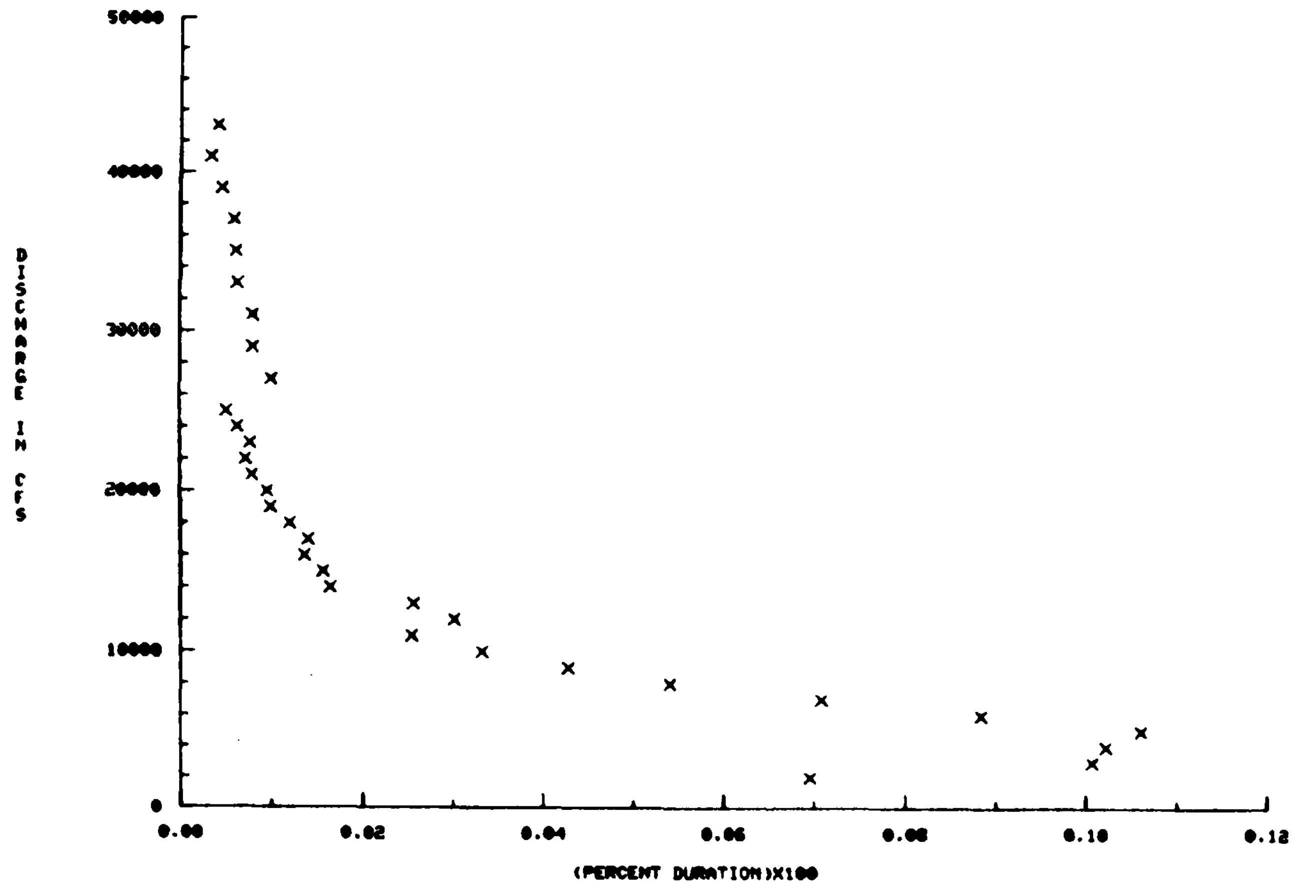


Figure 20. Flow duration analysis for North Walpole, New Hampshire.

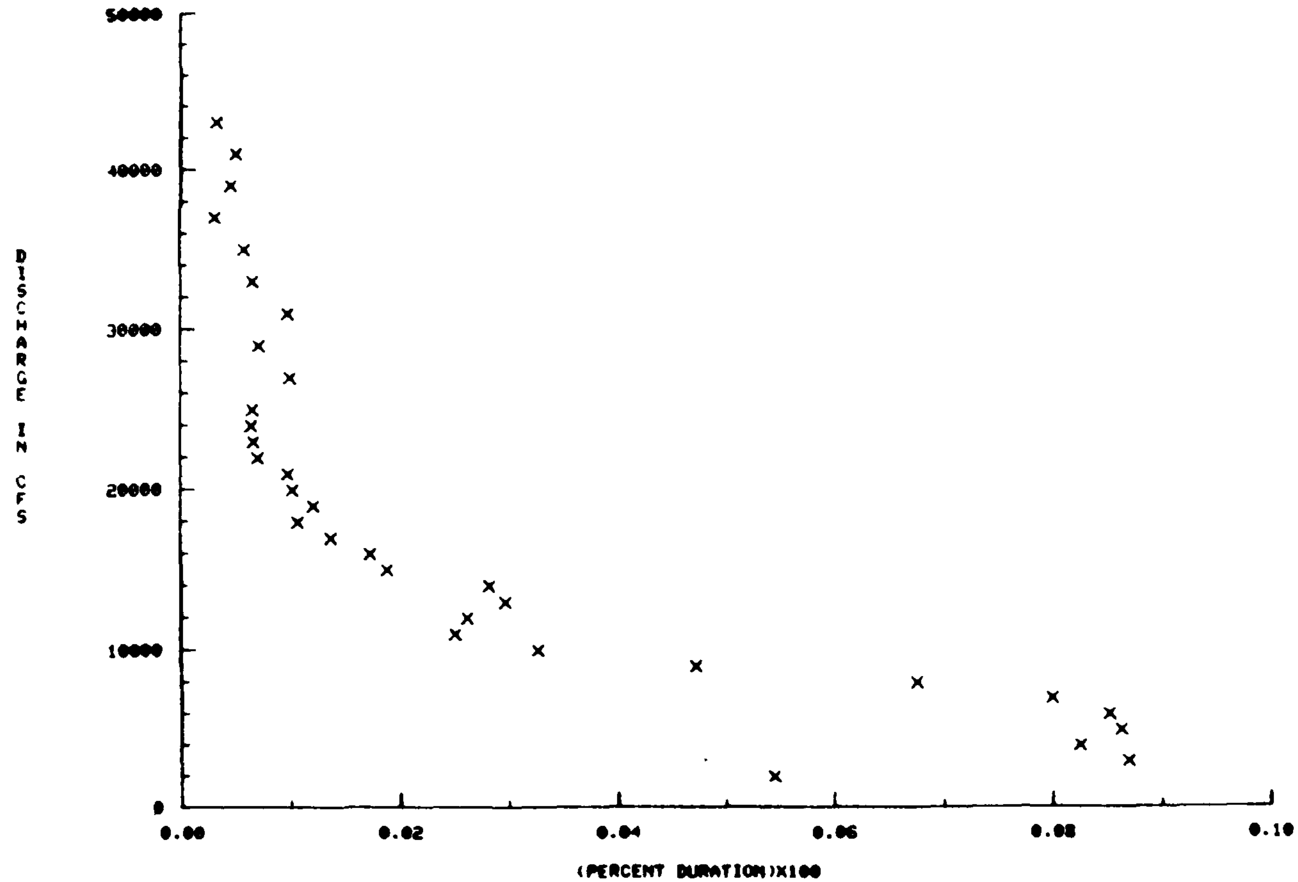


Figure 21. Flow duration analysis for Vernon, Vermont.

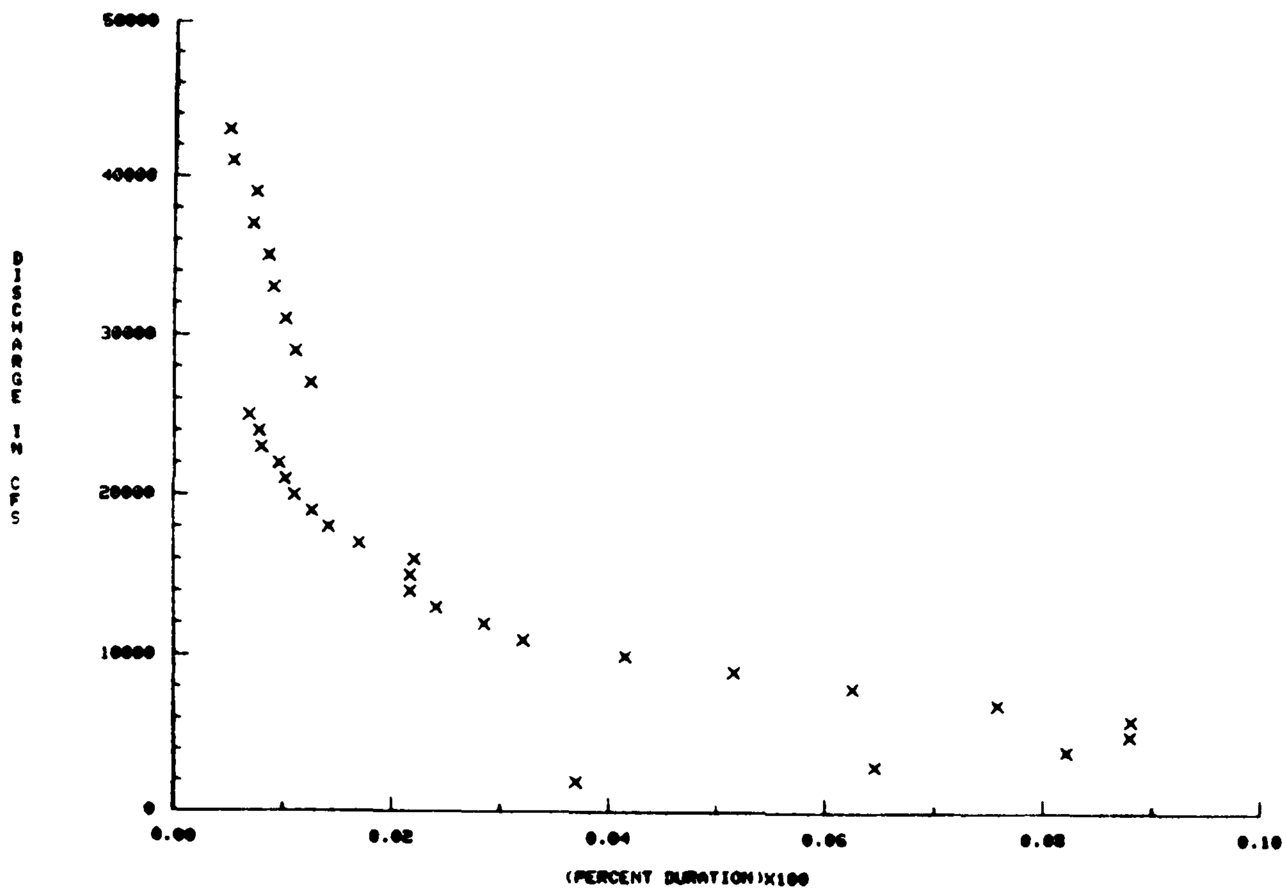


Figure 22. Flow duration analysis for Turners Falls Dam.

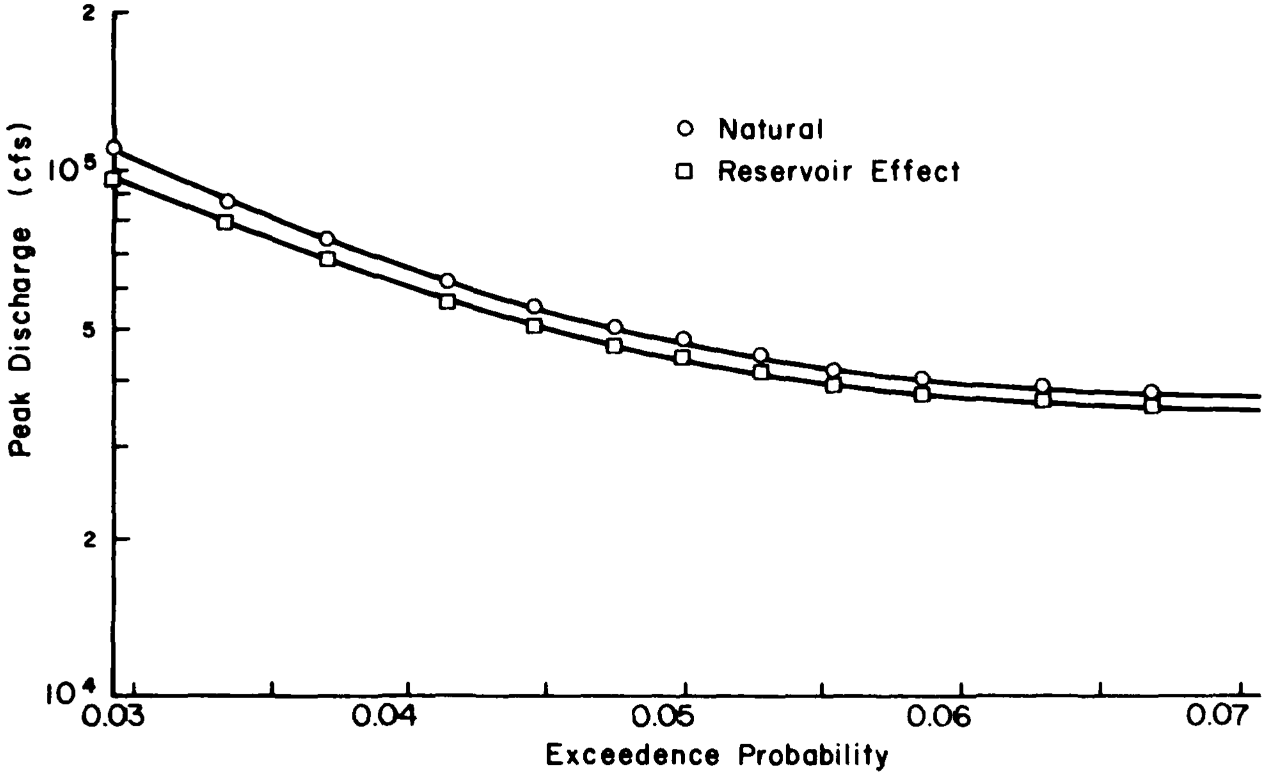


Figure 23. Flood frequency analysis at White River Junction under natural conditions and effect of reservoirs.

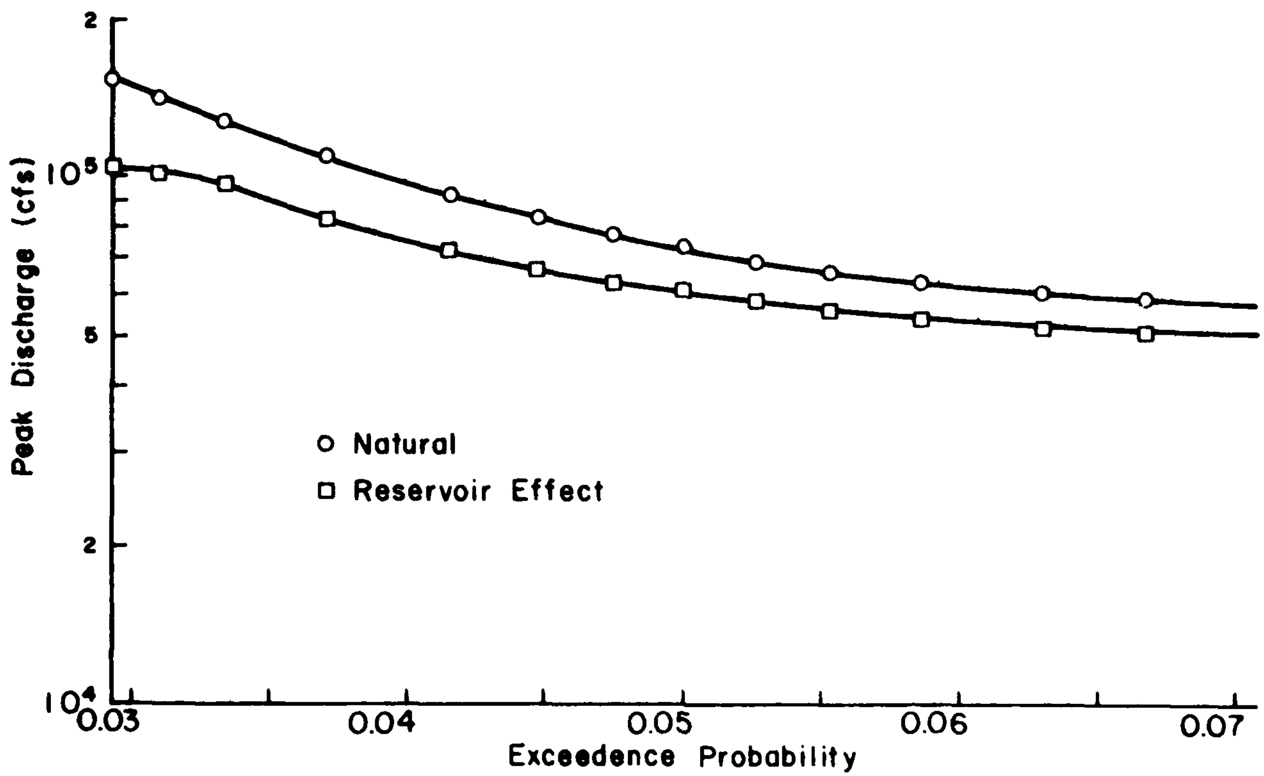


Figure 24. Flood frequency analysis at Vernon under natural conditions and effect of reservoirs.

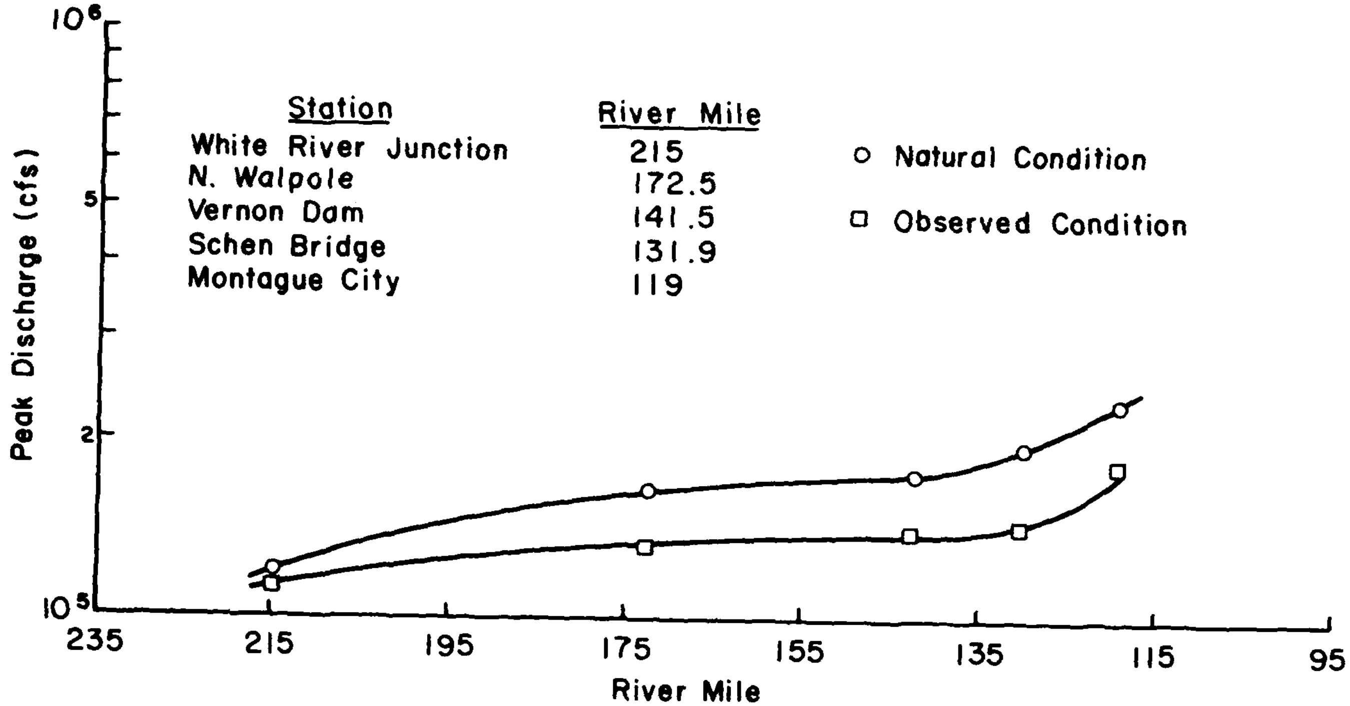


Figure 25. Effect of reservoirs on March 1936 flood.

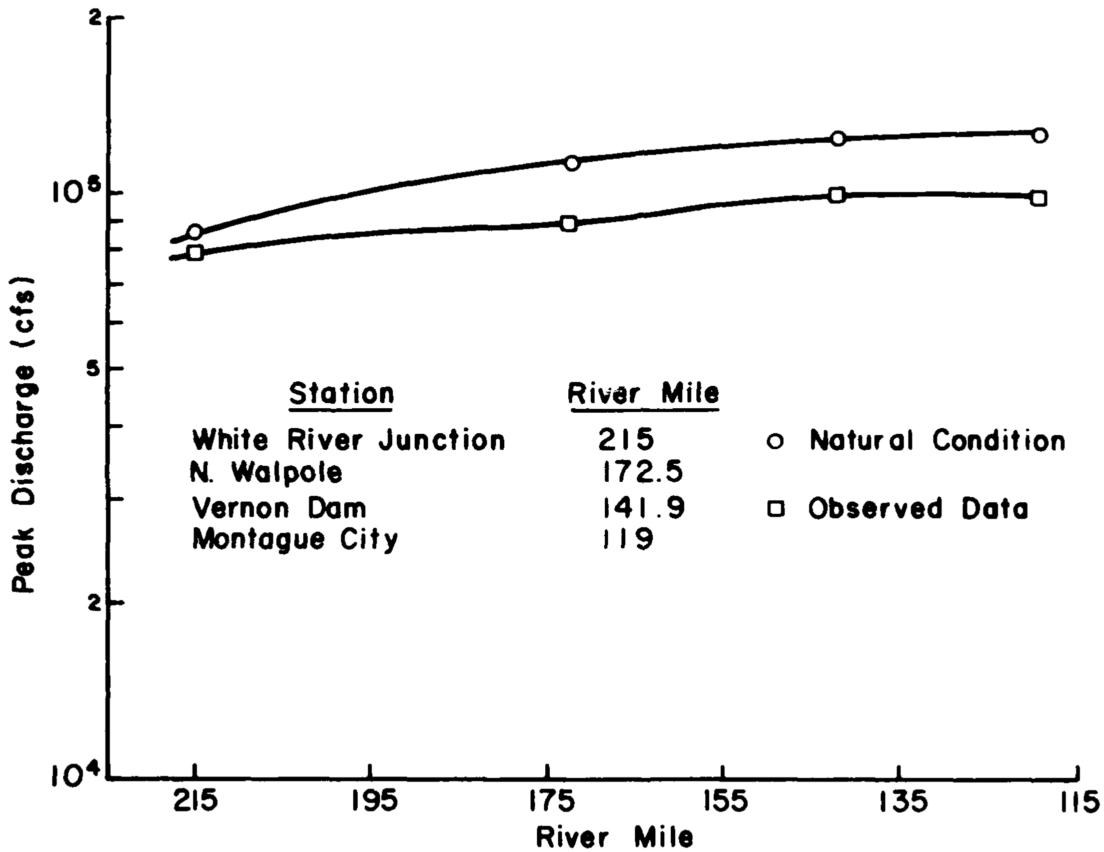


Figure 26. Effect of reservoirs on June and July 1973 flood.

Figure 27 shows monthly total precipitation at Turners Falls for 1964 and 1975. These two years correspond to relatively dry and wet years. In general, the wet period is from June to September. For comparison, the monthly mean discharges at Turners Falls in the corresponding years (1964 and 1975) are also given (Figure 28). Comparison between Figures 27 and 28 shows that the Connecticut River is highly regulated by the system of reservoirs, indicated by noting that the monthly mean discharges are nearly the same despite a significant difference in the total monthly precipitation.

The annual total precipitation is a good indication of water availability in the system. Plots of annual total precipitation at Hanover, Bellows Falls, Vernon, and Turners Falls Dam are given in Figures 29 through 32. These figures demonstrate a general trend over the last 12 years toward increased water availability. According to the study by the U.S. Army Corps of Engineers (1976), the short-term groundwater measurement gages located throughout the study reach also show a steady rise in groundwater level since 1970.

In addition to precipitation data, wind velocities and directions were also investigated in a cursory manner to determine the significant effects, if any, of wind-generated waves within the Connecticut River. A majority of wave action within the study reach is man-induced through recreational boating and wind-generated waves; however, neither of these were considered as a significant cause of bank erosion within this analysis.

HYDRAULIC AND SEDIMENT DATA

Channel Geometry

The locations of main channels, side channels, islands, and floodplains were determined from the available topographic maps and updated by in-field inspection. Channel geometric parameters of top width, depth, bed slope, cross-sectional area, and bank height and angle are significant to the types of erosion observed and the extent of the forces acting upon these types within the system. The location of bars and the sinuosity of the channel were also determined from the topographic maps and in-field surveys. The sinuosity of a river may be expressed numerically as the total river length in miles divided by the straight line distance between the upper and lower ends of the study area. The sinuosity of the study reach is approximately 1.38, indicating that the Connecticut River has low to medium sinuosity. Generally, the meandering pattern is very restricted and the river predominantly flows as a straight channel.

Within the study reach there are approximately 36 locations at which divided flow occurs; where islands or bars are prevalent in the main channel. However, in most of these cases, these islands and bars are small in comparative length and do not have any significant influence on the system as a whole. Of more significance to the flow patterns of rivers are man-made structures, canals, and power station diversions found predominantly at the locations of the four power stations sites.

The bed slopes of the river between the four power stations were determined from available data supplied by the power company and also from previous environmental impact statement reports that documented the thalweg levels extensively (Figure 33). However, no data were available pertaining to

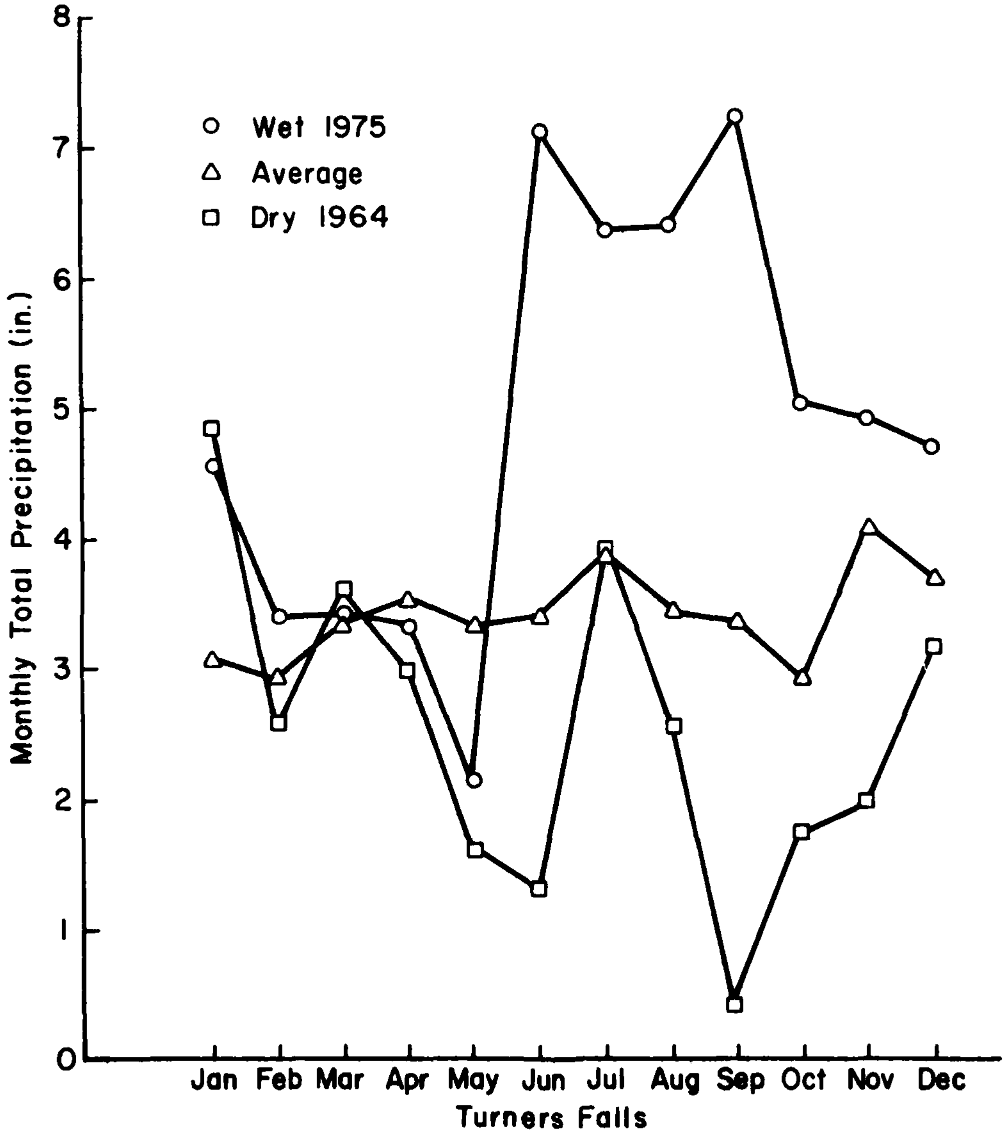


Figure 27. Monthly total precipitation at Turners Falls in 1964 (dry year) and 1975 (wet year).

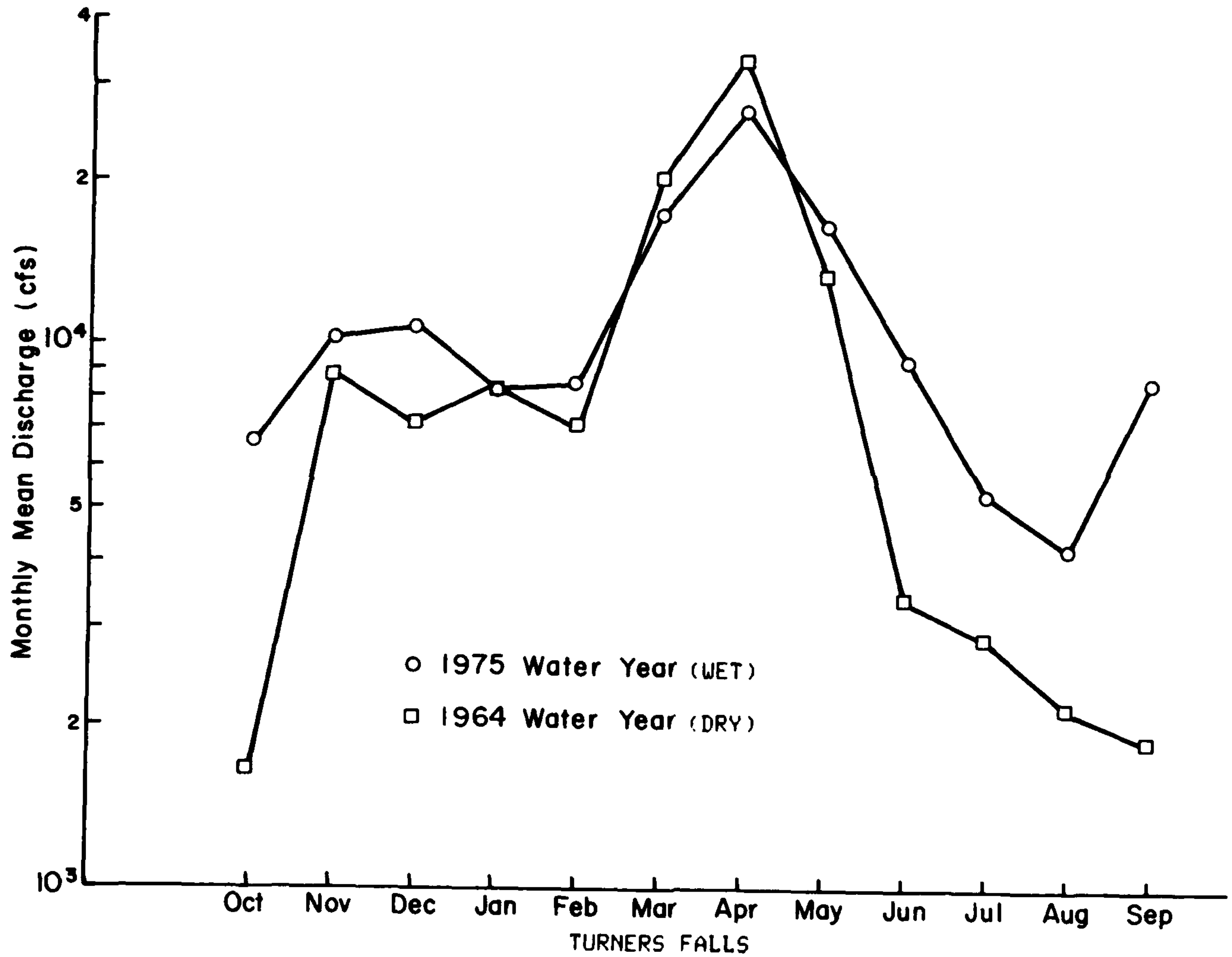


Figure 28. Monthly mean discharge at Turners Falls for 1964 (dry year) and 1975 (wet year).

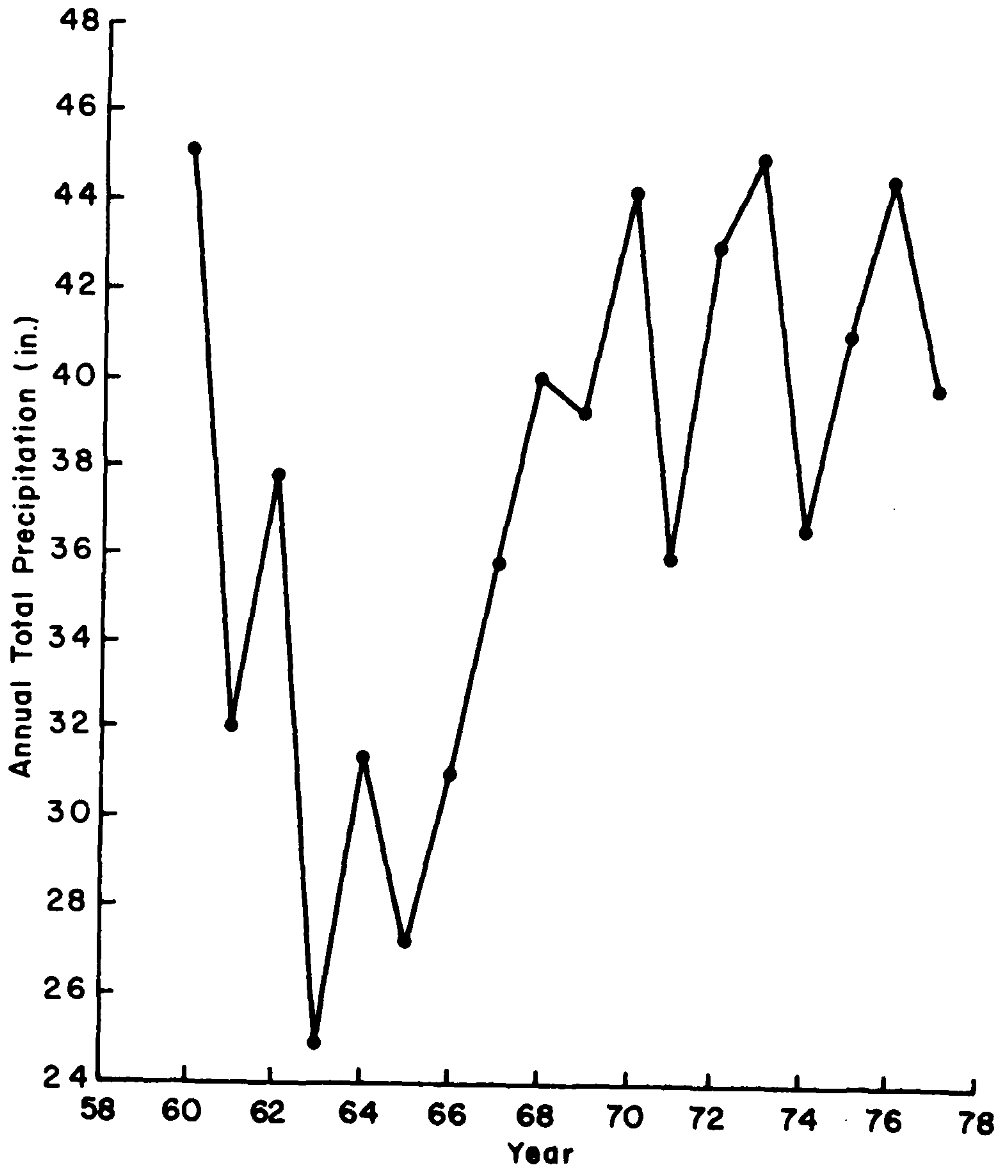


Figure 29. Annual total precipitation for Hanover.

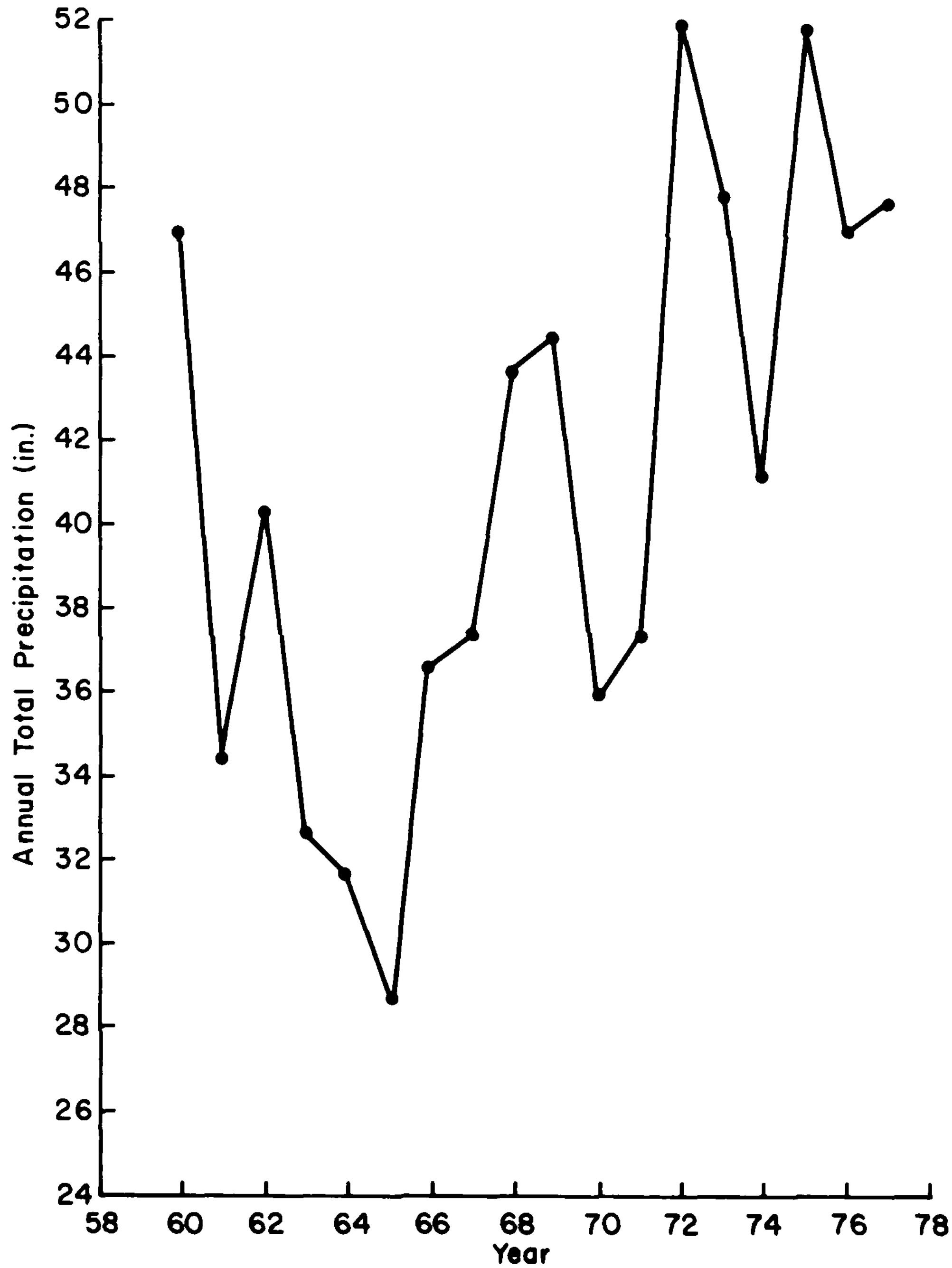


Figure 30. Annual total precipitation for Bellows Falls.

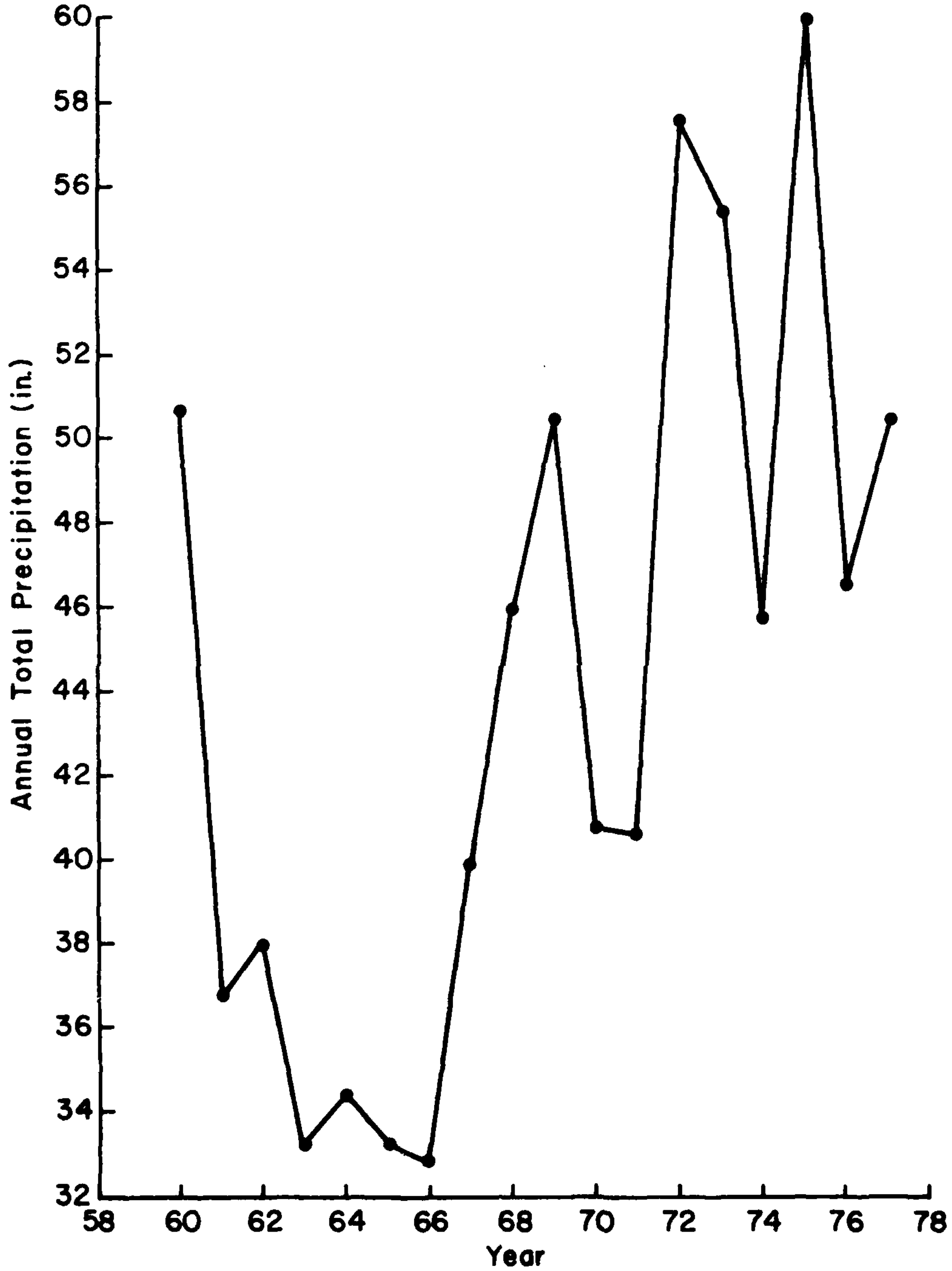


Figure 31. Annual total precipitation for Vernon.

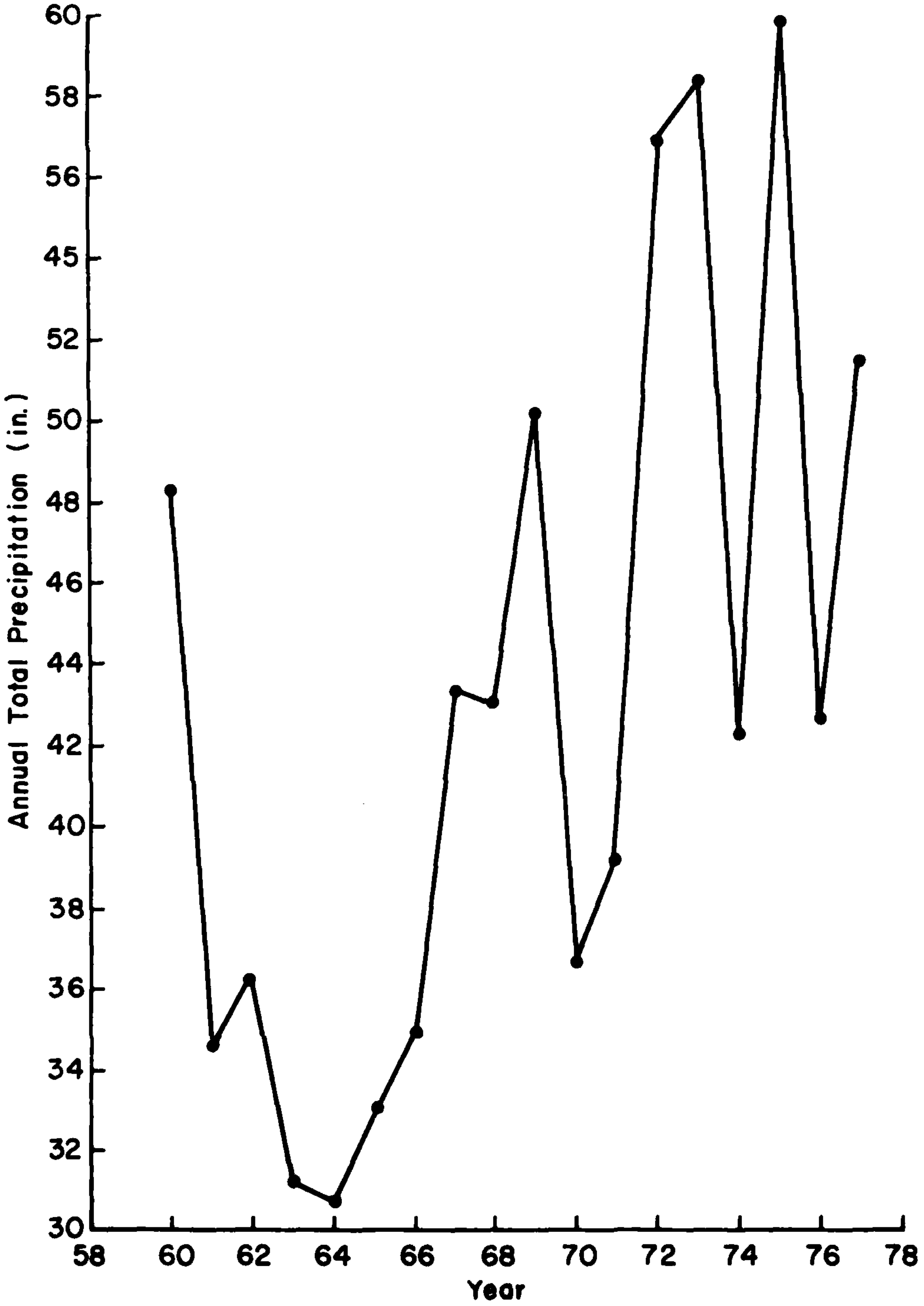


Figure 32. Annual total precipitation for Turners Falls Dam.

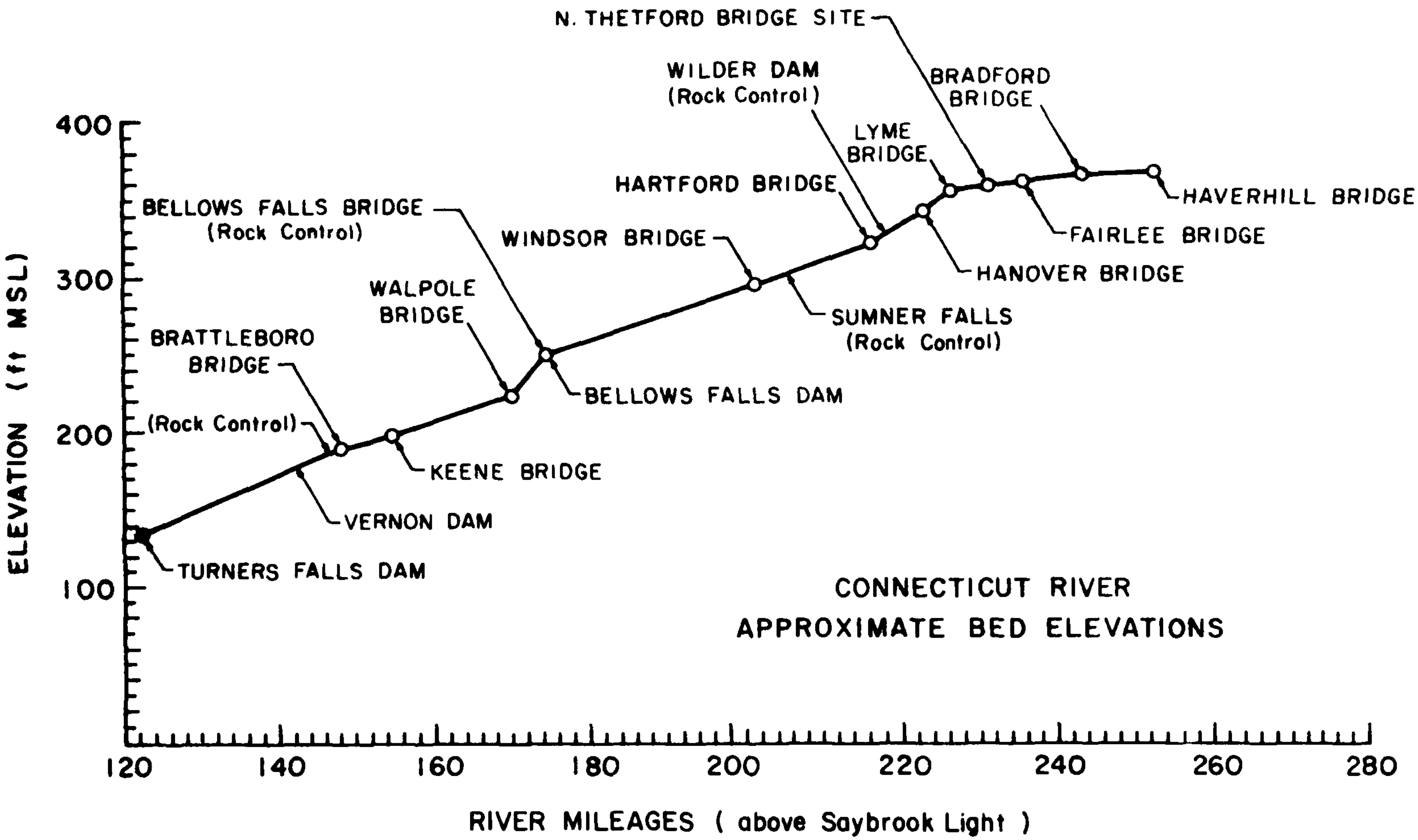


Figure 33. Thalweg bed elevation for the Connecticut River.

the energy slope for various flow regimes. Above Wilder Dam, energy slope gradients were prepared for discharge ranges up to 90,000 cfs (Figure 34). Unfortunately, this was the only area within the study reach where data of this nature were available.

Throughout the entire study reach there are numerous controls such as falls, rapids, restrictions, and rock outcroppings that significantly influence the bed slope of the channel along with the backwater profile irrespective of man-made structures. Rock outcroppings are extremely important in controlling degradation and aggradation of the bed, and consequently determine the type of erosion in the reaches below these outcroppings.

Channel cross-sectional data are essential for hydraulic and sediment analysis. Unfortunately, this important data item was insufficient in quantity and quality. Typical cross sections utilized by the U.S. Army Corps of Engineers to conduct the tractive force analysis are given in Figure 2. These cross sections are incomplete considering the whole cross section. Limited useful cross-section data are available near Bellows Falls (river mile 174.0 to 190.3). A representative cross section based on these data was compiled and is shown in Figure 35.

Sediment Data

Very few data pertaining to sediment exist within the study reach. Size distribution and material types observed on the floodplains following large floods were documented extensively by the U.S. Geological Survey and the Soil Conservation Service. Unfortunately, very few data describing bed material and bank material exist other than that collected in-field during this study. As a consequence of the lack of data, very little computation pertaining to total bed material load, suspended load, or wash load has been conducted on the river. Some sediment discharge records are available at lower reaches of the river, but no total sediment discharge data were available within the study reach. In the computations presented in Section 4, the analysis of bed material samples collected in the field in conjunction with the U.S. Geological Survey and the Soil Conservation Service data were used to determine estimates of the typical total sediment load in different reaches of the river. Discussions pertaining to recommendations for continuing sediment monitoring of the system are presented in Section 8. The distribution curve for the bed material sample taken one mile upstream of Wilder Dam is given in Figure 36. The sampling location is in the lower pool area. The bed material sampled is probably material transported in the natural river reach and deposited in the pool area.

Scour and Aggradation Areas

Due to the lack of long-term cross-sectional data throughout the study reach, very few records pertaining to bed scour and aggradation are available. As can be expected downstream of rock outcroppings and falls, the bed is predominantly granitic rock. Very little aggradation was observed during the in-field data collection at the time of low pool levels throughout the study reach. However, it is possible that significant aggradation of fine materials may be occurring within the system, especially at the back of the power station dams. Unfortunately, no data are available to confirm this. Recommendations regarding the establishment of cross-sectional studies within the river are made in Section 8.

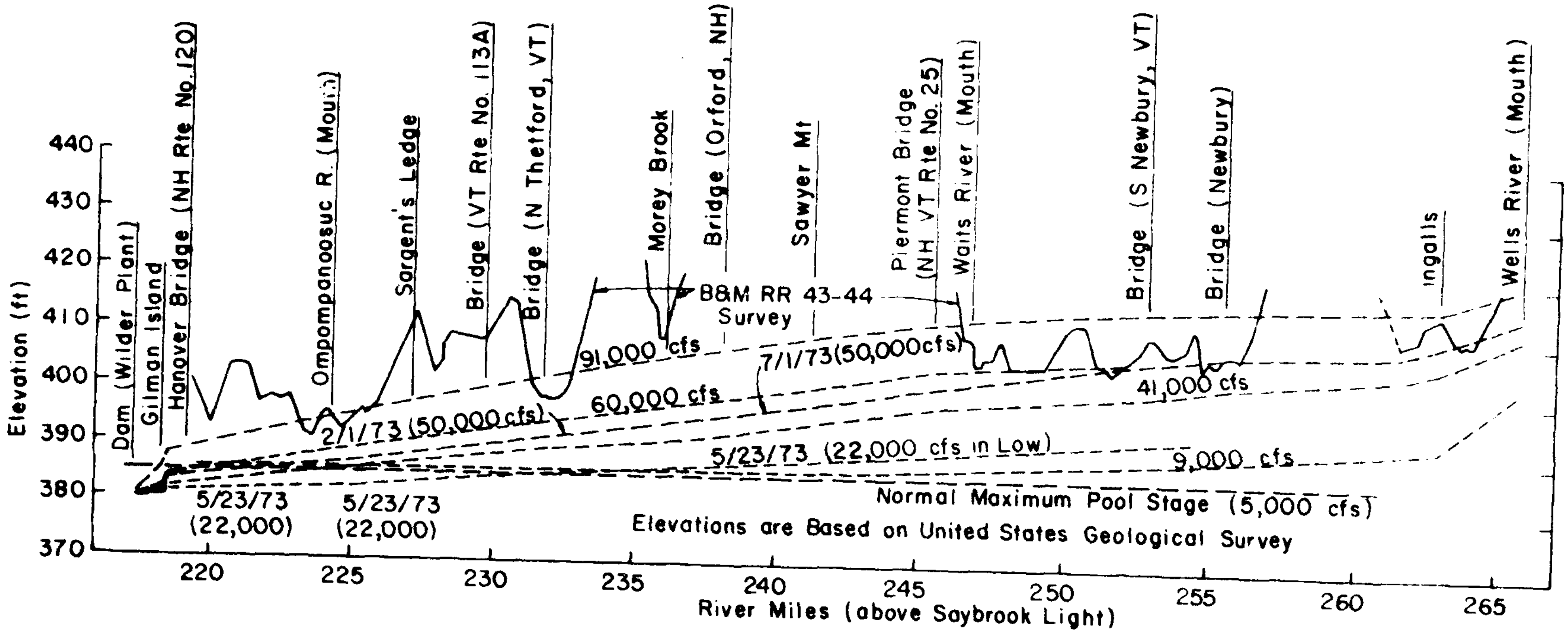


Figure 34. Wilder pool water surface profile.

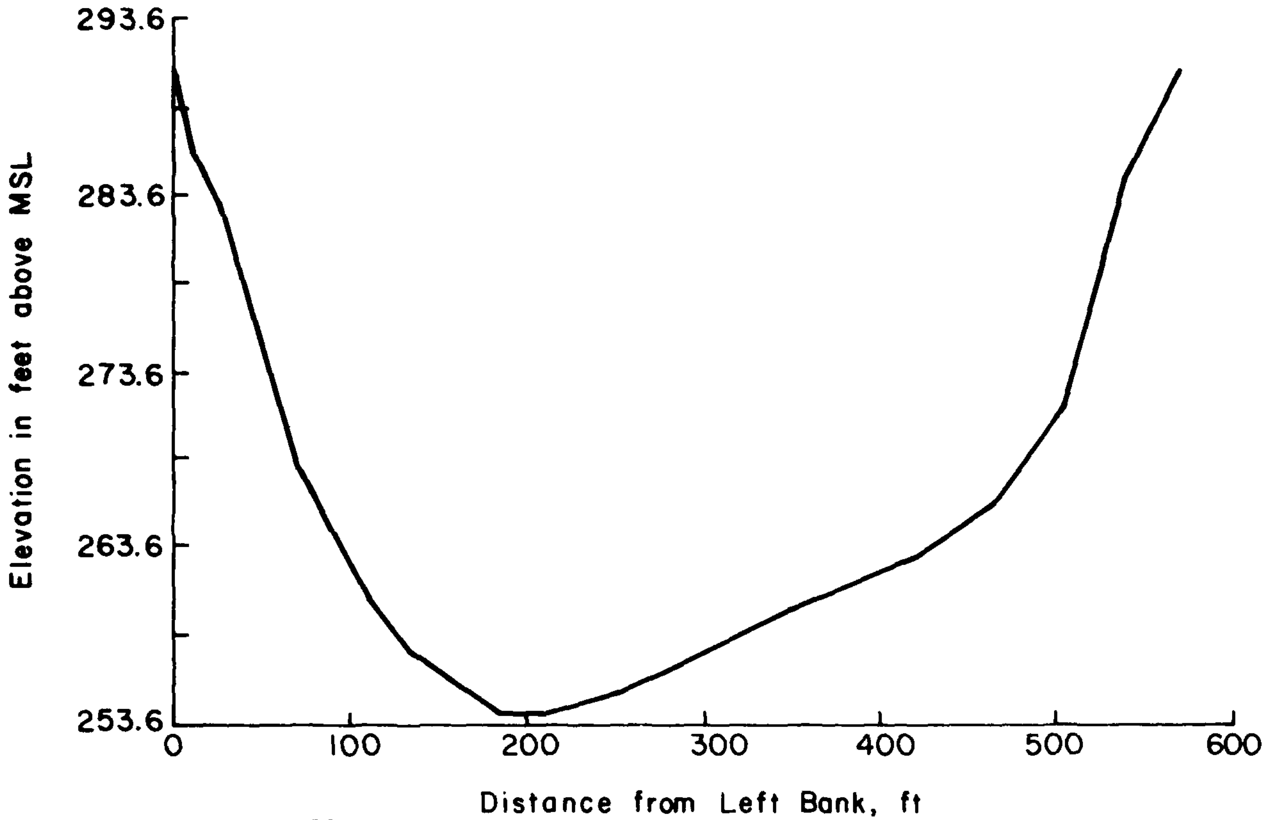


Figure 35. Representative cross section based on Bellows Falls data.

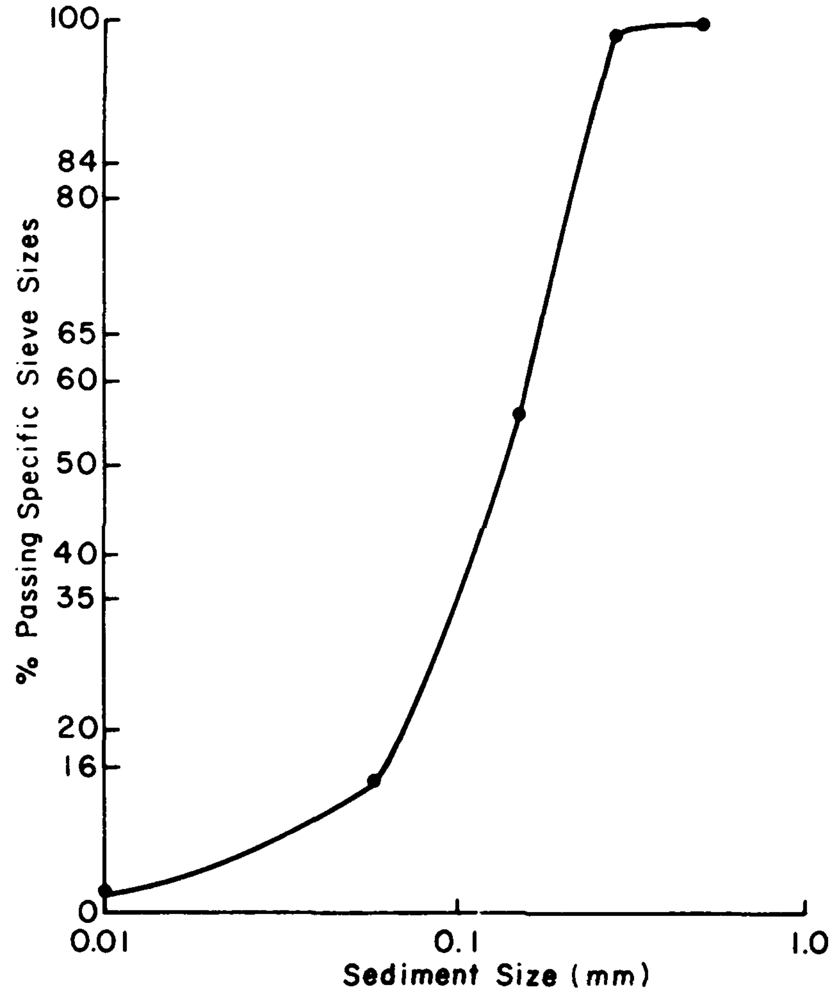


Figure 36. Bed material distribution for Connecticut River at Wilder, Vermont (1 mile upstream of dam).

Groundwater Data

The USGS has mapped some of the groundwater table within the study area and the monitoring of specific locations is continuing. In addition, the Corps of Engineers has monitored the groundwater table at six index sites.

At area 51 two continuous recording piezometers were installed to monitor groundwater fluctuations in relation to river stage. A cross-sectional schematic of this installation is given in Figure 37.

Some of the resulting piezometric data have been plotted in relation to river stage, and the resulting graphs over time reveal a near-zero lag time between river stage peaks or troughs and piezometer head peaks or troughs. This is primarily due to the nature of the bank material and the proximity of the recorders to the river bank. In general, piezometer readings are closely related to the variation in river stages and represent the inflow-outflow water movement within the banks. Very little difference in the groundwater level is observed between the two piezometer records.

Ice Occurrence and Relevant Data

As with sediment discharge data, very few specific ice formation data are available for the river. There have been some local observations of the period when ice starts to form on the river, when it begins to melt, and the thicknesses at various locations within the study reach. Unfortunately, no specific data on a long-term basis were collected or estimated for the area. Recommendations regarding the collection of data on ice occurrence, thickness, and volume are made in Section 8.

ENVIRONMENTAL DATA

Inspection of all available data pertaining to forest land, vegetation types, wildlife, fish habitat, turbidity of the water related to fish life, water quality, and water temperature was made during the study of the Connecticut River. Extensive discussions were conducted to investigate the effects of vegetation on bank stability and the possible problems that would result upon the removal of this vegetation. Most of the data pertaining to the environment were collected from previous environmental impact statements prepared by the power companies and other agencies, and during in-field inspections and associated discussions with people owning property bordering the river. Water temperature data were also available for inspection at some locations within the river, but most of these data were randomly sampled and no long-term water temperature data were available except at the Yankee Nuclear Power Plant.

SUMMARY

Characteristics of the Connecticut River are summarized below.

1. The Connecticut River is highly regulated by a system of reservoirs that reduce flood peaks, resulting in significant effects downstream.

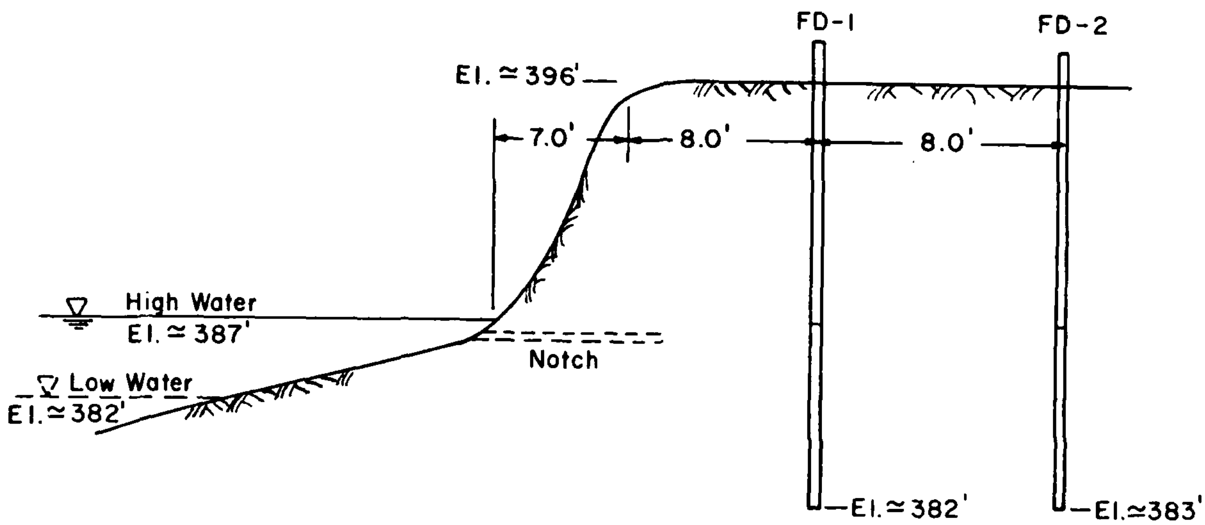


Figure 37. Schemmatic of piezometer installations at Site #51. (after R. McGraw CRREL, August, 1978).

2. There is a general tendency for pool fluctuations to increase with time due to increasing power demands.

3. Pool fluctuations are significant and continuous which adversely affects bank stability.

4. Pool fluctuations are small compared to natural changes in stage.

5. The past few years have been relatively wet, and consequently the supply of water has increased. This increase in water supply and the storage effect of the reservoirs has caused the groundwater level to rise. Additionally, increases in soil saturation may increase mass wasting of the banks.

6. The wind-wave-induced erosion is not significant. Boat waves are perhaps more important. The current data base is not adequate to quantify erosion caused by boat waves.

7. Observed erosion sites are less frequent in the pools than in the natural river indicating that the natural river reaches are more susceptible to erosion than pool reaches.

8. Hydraulic properties such as depth, velocity, energy slope, width, and sediment transport capacity are different for each reach and are dependent upon location within the reach.

9. Sediment and cross-sectional data are the two most important data gaps preventing a quantitative analysis of the Connecticut River.

SECTION 3

CAUSES OF BANK EROSION

REVIEW OF LITERATURE

A comprehensive literature review on the causes of bank erosion and sediment transportation and deposition along the Connecticut River was conducted. Based upon this review, a comprehensive annotated bibliography was developed by Simons, Andrew, Li and Alawady (1979) and presented as a separate report. This bibliography consists of annotated references, unannotated references, and a listing of reports that are indirectly important in the analyses of river bank erosion problems. It should be noted that the annotations are selected statements from the authors' abstracts. The reports and publications reviewed include those that analyze erosion based upon laboratory studies, field studies, physical models, mathematical models, and empirical and mathematical investigations of forces causing erosion, and of erosion problems in general.

It should be further noted that the references cited are not limited to the Connecticut River. Most of the physical processes governing the magnitude of bank erosion are active under most river conditions. Hence, even though the literature search was oriented to determine the extent of knowledge pertinent to erosional problems for the Connecticut River, because of the commonality of many forces causing erosion in all climates, the reference material presented is all-inclusive. With respect to the completeness of the literature search, it should be stressed that even using computer search techniques, a complete listing is not possible. Undoubtedly, many open file reports, reports published abroad, theses, dissertations, and consultant reports have not been identified because they are not available in libraries or otherwise accessible. Nevertheless, it is believed that most of the important references have been identified.

An important reference cited in this study is "Interim Report to Congress, September 30, 1978--Section 32 Program--Streambank Erosion Control Evaluation and Demonstration Act of 1974." This report documents that the United States contains nearly 3.5 million miles of channel and that erosion is occurring on over half a million miles of bank lines. In recognition of the serious economic losses due to bank erosion, the U.S. Congress passed the Streambank Erosion Control Evaluation and Demonstration Act of 1974 (Appendix C). This act authorizes a 5-year program that will cost the taxpayer many millions of dollars. The study consists of an updated analysis of the extent and seriousness of streambank erosion.

The scope of the program includes the following work units.

1. Evaluation of extent of streambank erosion, nationwide.

2. Literature survey and evaluation of bank protection methods.

3. Hydraulic research on effectiveness of bank protection methods.

4. Research on soil stability and identification of causes of streambank erosion.

5. Ohio River demonstration projects.

6. Missouri River demonstration projects.

7. Yazoo River Basin demonstration projects.

8. Demonstration projects on other streams, nationwide.

9. Reconstruction at demonstration projects.

10. Reports to Congress.

Note that Item 4 deals with identification of causes of streambank erosion. As a part of the review of literature for this study and other related work the writers visited many of the demonstration sites and have participated in meetings, workshops and field trips dealing with bank erosion along many rivers throughout the United States, including the Connecticut River, the Mississippi River, the Missouri River, the Des Moines River, the Ohio River, Goodwin Creek, etc. Little progress has been made thus far regarding quantification of the specific causes of erosion. Hence, it is necessary for this study to proceed with very limited information from other sources regarding details of man-induced and natural erosional processes, their impacts on erosion rates and prediction of erosion, process by process.

VARIABLES AFFECTING RIVER CHANNELS

The large number of variables that affect channel geometry, channel stability, bed forms in sand bed and gravel channels, and velocity are interdependent. Some of the variables change with the flow conditions and alter their roles from dependent to independent variables. It is difficult, especially in field studies, to differentiate between the independent and dependent variables.

Simons and Richardson (1965) reported a comprehensive study of variables affecting resistance to flow, flow characteristics, their dependence and independence, and the conditions in which a dependent variable becomes independent, or the reverse.

The principle variables involved in the analysis of flow in alluvial channels are:

$$f_1\ (U,\ d,\ S_E,\ \rho,\ \mu,\ g,\ D,\ \sigma,\ \rho_s,\ S_p,\ S_r,\ S_c,\ f_s) = 0 \qquad (3\text{-}1)$$

where

U = average velocity of flow,

d = average depth,

S_E = slope of the energy grade line,

ρ = density of water,

μ = dynamic viscosity of the water,

g = gravitational acceleration,

D = representative fall diameter of the bed material,

σ = measure of the size distribution of the bed material,

ρ_s = density of sediment,

S_p = shape factor of the particles,

S_r = shape factor for the reach of the stream,

S_c = shape factor for the cross section of the stream, and

f_s = seepage force in the bed of the stream.

In Equation 3-1, fluid viscosity μ and particle shape factor S_p can be eliminated if the fall velocity of the bed material w is included in the list of variables, changing Equation 3-1 to

$$U = f_2 \ (d, S_E, \rho, \rho_s, g, D, \sigma, w, S_r, S_c, f_s) \quad . \qquad (3\text{-}2)$$

The role of each variable and its relative importance are discussed in the following sections.

Depth

In the natural river system depth is an important indicator of: (1) the size of the channel, (2) the stability of the banks (width-to-depth ratio), and (3) the shear stress exerted on the channel boundary by the flow. For the natural system the width-to-depth ratio was relatively small, indicating greater than average channel stability. With the construction and operation of the hydropower dams, the usefulness of depth as an indicator of shear stress and channel stability is limited. It can be concluded, then, that in the present system the depth is artificially increased, the velocity of the flow is reduced, deposition of sediment is encouraged during periods of low flow and with greater wetted bank height the zone of erosion is shifted landward. Also, the forces causing erosion act on new bank materials that were previously only subjected to erosive forces during periods of flooding. Hence, the usefulness of depth as an indicator of channel behavior is limited and must be used with caution when interpreting bank stability and designing bank protection works.

Slope

The slope of the energy gradient plays an extremely important role in the hydraulics of river channels. Slope is utilized in velocity equations such as the Manning equation to estimate average velocity, and it is utilized in the tractive force equation, $\tau = \rho g d S_E$, to estimate the tractive force exerted on the bed and banks of open channels. The magnitude of the energy gradient has been altered by the low head hydropower dams. Hence, to obtain a meaningful value of slope of energy gradient, it is necessary to define it as the slope of the imaginary line located a distance equal to the velocity head $\frac{V^2}{2g}$ above the water surface. In summary, the analysis of the stability of the system must consider the changes imposed on the slope of energy gradient by the systems of dams. The system no longer operates as a free-flowing alluvial channel. Its energy gradient and the velocity have been reduced except for those reaches above the influence of the pools. Hence, as with depth one must proceed with caution when utilizing hydraulic and similar relations to interpret the stability of the channel.

Density

The density of a sediment-water mixture can be computed from the relation (Simons and Sentürk, 1977)

$$\rho_s = \frac{\gamma \, \gamma_s}{\gamma_s - C_s(\gamma_s - \gamma)} \tag{3-3}$$

where ρ_s is the density of a sediment-water mixture and C_s is the concentration of the suspended sediment by weight expressed as a percentage.

The density of the water-sediment mixture increases with increasing concentrations of suspended sediment. Hence, during periods of flooding the specific weight, γ, can be increased by the suspension of sediment, and this specific weight should be used in the tractive force equation to estimate the boundary shear stress.

Size of Bed Material

Effects of the physical size of the bed material on bed configurations and resistance to flow include: 1) its influence on the fall velocity that is a measure of the interaction of the fluid and the particle in the formation of the bed configurations, 2) its effect as grain roughness, and 3) its effect on the turbulent structure and the velocity field of the flow. The first effect is of great importance in combination with the characteristics of flow and fluid. These factors largely determine the different bed forms.

Gradation of Bed Material

The gradation of the bed material has a significant effect on bed form. Daranandana (1962) reported that bed forms of uniform sand are more angular and have considerably larger resistance to flow than graded sands with identical median fall diameters. Sentürk (1976) showed that an increase in D_{35}/D_{65} increases the resistance for a dune bed and decreases resistance for a ripple bed.

Fall Velocity

Fall velocity is the primary variable that determines the interaction between the bed material and the fluid. For a given depth and slope, it significantly affects the bed form that will occur, the actual dimensions of the bed form, and except for the contribution of the grain roughness, the resistance to flow. With an increase in fall velocity there must be an increase in the product of depth times slope (that is, shear stress) for the bed to change from a static plane to ripples or dunes, or from dunes to a plane bed and antidunes.

The parameter f given by Sentürk (1976) includes the fall velocity and can be written as

$$f = \frac{R}{D_{65}} \cdot \frac{D_{65}}{D_{35}} \cdot \frac{\gamma}{\gamma_s'} \cdot \frac{S}{\left(\frac{w_{65} D_{65}}{\nu}\right)^2} \tag{3-4}$$

If only w is a variable, an increase in its value causes: 1) an increase in the resistance for a dune bed, 2) a decrease in the resistance for a ripple bed, and 3) a change from ripples to dunes. If the bottom is a plane, an increase in wD/ν first decreases the resistance to motion and slightly increases it afterwards.

Shape Factor for the Reach and Cross Section

The shape factor for the reach and cross section enters into the analysis because of the nonuniformity of a reach and cross section in a natural stream. The shape factor for a reach of channel affects the energy losses. This is due to bends and banks and the effect of the shape factor on the velocity distribution, boundary shear stress, and secondary circulation. The shape factor for the cross section has a similar effect across a channel section causing variations in velocity, width, depth, and boundary shear stress. Variations of these variables and combinations of variables cause multiple bed roughness in the channel section. Detailed knowledge of these factors in natural channels is quite limited

Bed and Bank Material

Resistance of a river bank to erosion is closely related to the characteristics of the bank material. The characteristics of the material forming the banks of rivers are highly variable. Bank material deposited in the river can be broadly classified as cohesive, noncohesive, and stratified. The cohesive material is more resistant to surface erosion and has low permeability that reduces the effects of seepage, piping, frost heaving, and subsurface flow on the stability of the banks. However, such banks when undercut and/or saturated are more likely to fail due to mass wasting processes such as sliding.

The noncohesive bank material tends to be removed grain by grain from the bank line. The rate of particle removal, and hence the rate of bank erosion, is affected by factors such as the direction and magnitude of the velocity adjacent to the bank, the turbulent fluctuations, the magnitude and fluctuations in the shear stress exerted on the banks, seepage force, and piping and wave forces, many of which may act concurrently.

The stratified banks are very common on alluvial rivers and generally are the product of past transport and deposition of sediment by the river. More specifically, these types of banks consist of layers of materials of various sizes, permeability, and cohesion. The layers of noncohesive material are subject to surface erosion, but may be partly protected by adjacent layers of cohesive material. This type of bank is also vulnerable to erosion and sliding as a consequence of subsurface flows and piping. To better understand the erodibility of stratified banks in general, a physical model of such a bank was constructed in the Hydraulics Laboratory at Colorado State University and subjected to both short- and long-term changes in stage and wave action to simulate activity by wind and/or boats. The model consisted of a tank within which bank erosion caused by changes in pool level, water waves and piping could be observed and evaluated. The testing procedure was conducted in the system illustrated in Figure 38. The tests conducted were as follows:

1. The pool level was raised and lowered frequently to simulate pool level changes.

2. The water in the tank was displaced to form a wave that attacked the bank. The effects of a sequence of many waves were observed.

The response of the bank to changing stage was removal of some of the permeable material in Zone 1. This material was carried riverward as the pool level was lowered. In time sufficient granular material was eroded that the block of bank material above Zone 1 tilted downward, opening a vertical tension crack (Figure 39). When the bank line was subjected to wave wash after the initial failure signified by the settling down of the block of bank material and the development of the tension crack, the bank line geometry developed as shown in Figure 40. After achieving this geometry, the erosion rates became small, indicating that the channel was achieving some level of stability.

It was not possible with the system to subject the bank to erosion by flowing water. However, if it had been possible, one would expect the flowing water to erode the lower and upper bank landward setting the system up for a second cycle of erosion unless the lower bank was stable, preventing landward erosion. The observed bank erosion resulting from these laboratory actions was compared with bank erosion observed in the field where bank lines were subjected to varying forces including boundary shear stresses. By this means it was possible to gain insight into the relation between the erosion caused by the tractive force and erosion caused by changing stage and wave action.

The bed material affects bank erosion indirectly. For example, if the bed is eroded, lowering the bed level adjacent to the banks, a higher unsupported bank results that is more susceptible to undercutting and failure. Also, the bank is more susceptible to seepage and piping because additional lenses of erodible, noncohesive material may be exposed in the bank. Inflow and outflow of water through the more permeable layers causes piping. This action weakens the bank, allowing partial slumping, sliding, and consequently, an increase in the rate of bank erosion. Conversely, in an aggrading reach of river, the accumulation of bed sediments can increase the local energy gradient, increase the velocity of the flow, and subject the bank line to large shear stress and larger velocities that complement accelerated bank erosion. In addition, the movement of bed material is related to the bed forms and bars that form on the beds of rivers. These roughness elements affect the resistance to flow, velocity and velocity distribution, and hence have an important effect on bank erosion. As cited before, the accumulation of bed sediment in the form of bars causes a displacement of the flow. For example, the forming of an alternate bar adjacent to one bank forces the water towards the opposite bank, increasing the velocity of the flow and shear stress attacking the bank (Figure 41). These alternate bars form in all straight and relatively straight reaches of alluvial rivers and canals where transport of bed material occurs. The formation, movement and effect of these bars have been documented by Simons and Sentürk (1977) and others. The amplitude of these alternate bars decreases with decreasing width-depth ratios.

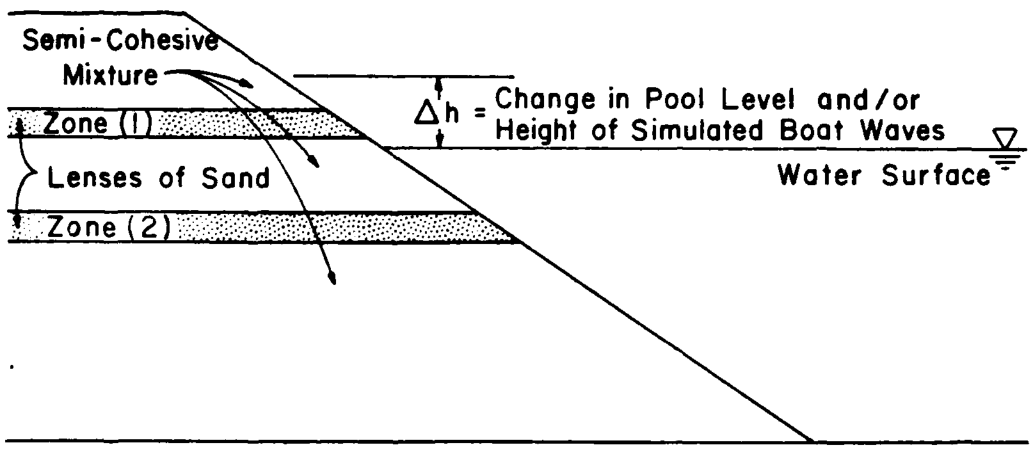


Figure 38. Sketch of physical model study.

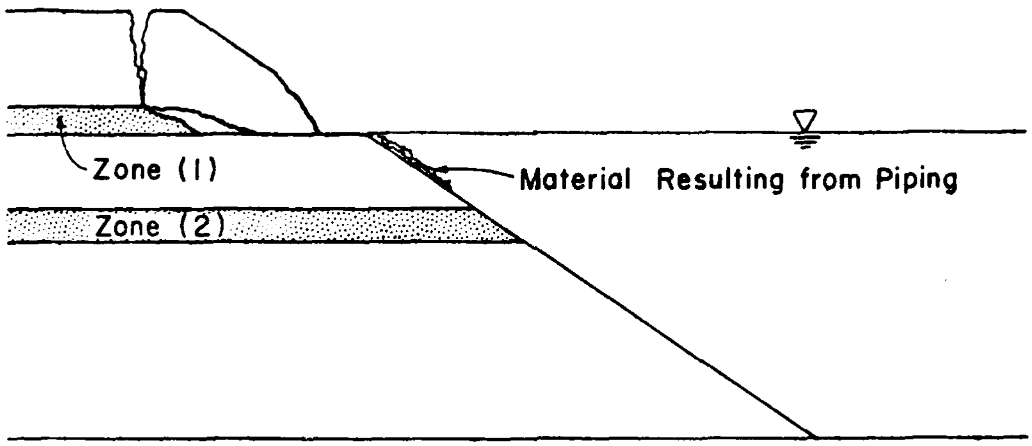


Figure 39. Development of tension crack.

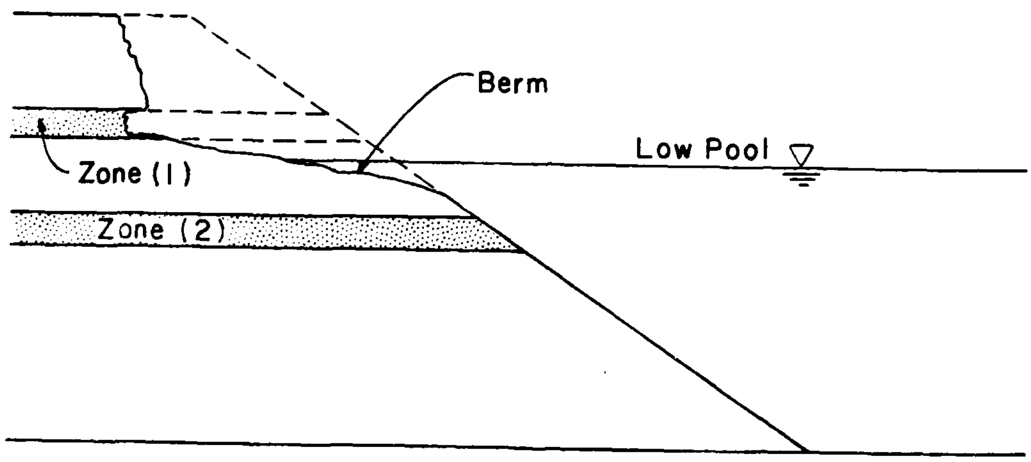


Figure 40. Development of berm.

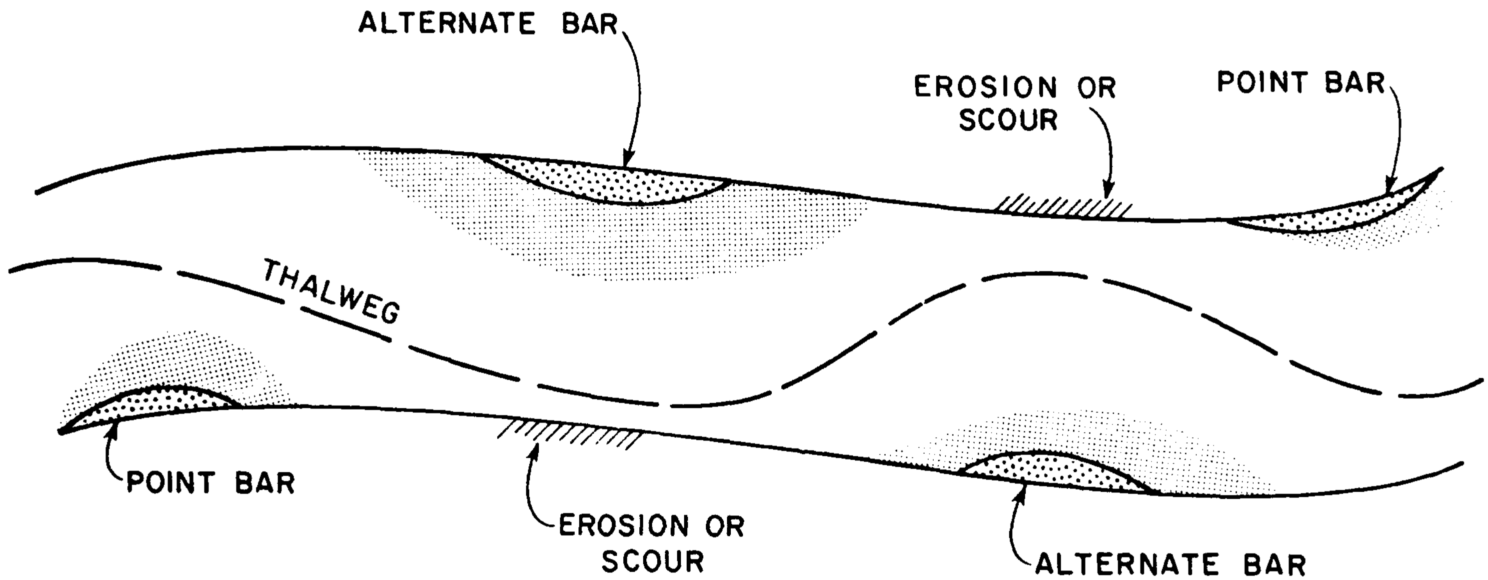


Figure 41. Typical shoaling and scour patterns in a relatively straight reach of alluvial channel.

Subsurface Flow

The banks of alluvial rivers experience varying degrees of flow. Forces that cause the movement of water through the bank material may be generated by several factors:

1. On the rising stage a gradient develops, sloping from the river channel into the bank material. On the falling stage the energy gradient reverses direction and water moves through the banks toward the river channel decreasing the stability of the bank.

2. If the water table is higher than river stage, flow will be from the banks into the river. The high water table may result from many conditions: a) a wet period during which water draining from adjacent watersheds saturates the floodplain to a higher level, b) poor drainage conditions resulting from deterioration or failure of drainage systems, c) increased infiltration resulting from changes in land use causing an increase in water level, and d) development of the adjacent floodplain for homes and businesses that utilize septic tanks and leach fields to dispose of waste water and sewage.

3. In general, the storage and release of water for hydropower generation causes numerous fluctuations in river stage. These changes in stage, even though relatively small, cause flow conditions in the banks as described in the first paragraph.

4. Wind-waves cause local variations in stage that introduce inflow and outflow of water from the banks. However, because the duration of the change in stage is small, the inflow and outflow phenomena are usually concentrated locally in the surface of the banks.

5. Boat-generated waves have an effect similar to wind-waves, but the characteristics of the waves generated are different. This difference must be considered when comparing bank erosion caused by wind- and boat-generated waves.

6. The formation and loss of backwater caused by ice flows and ice jams lead to seepage into and out of the banks.

The presence of water in the banks of rivers and its movement toward or away from the river affect bank stability and bank erosion in various ways. The related erosion of banks results as a consequence of seepage forces, piping, and mass wasting.

With flow of water from the river into the adjacent banks, a stabilizing seepage force is generated. Rivers that continuously seep water into the banks tend to have smaller widths and larger depths for a particular discharge. The reverse is true of the rivers that continuously gain water by an inflow through their banks. The inflowing water creates a seepage force that makes the banks less stable. This condition was verified by laboratory tests at Colorado State University (Karaki, 1968).

Piping of River Banks

Piping is another phenomenon common to the alluvial banks of rivers. With banks that are stratified, i.e., with lenses of sand and coarser material sandwiched between a layer of finer cohesive materials, flow is induced in more permeable layers by changes in river stage and, to some degrees, by wind- and boat-generated waves. With a rise in river stage, a gradient is developed that induces flow into the more permeable lenses of the banks. As the stage drops, the energy gradient is reversed and significant flow occurs toward the river in the more permeable lenses. If the flow through the permeable lenses is capable of dislodging and transporting particles from the permeable lenses, the material is slowly removed, undermining portions of the bank. Without this foundation material to support the overlying layers, a block of bank material drops down and results in the development of tension cracks that may allow surface flows to enter, further reducing the stability of the affected block of bank material. Bank erosion may continue on a grain-by-grain basis or the block of bank material may ultimately slide downward and outward into the channel causing bank failure as a result of a combination of seepage forces, piping, and mass wasting.

Bank Stability with Respect to Mass Wasting

An alternate form of bank erosion is caused by local mass wasting. If the bank becomes saturated and possibly undercut by the flowing water, blocks of the bank may slump or slide into the channel. A graphic illustration of typical mass wasting is shown in Figure 42. Mass wasting may be further aggravated by construction of homes on river banks, operation of equipment on the floodplain adjacent to the banks, added gravitational force resulting from trees, location of roads that may cause unfavorable drainage conditions, saturation of banks by leach fields from septic tanks, and increased infiltration of water into the floodplain as a result of changing land use practices.

Landslides, the downslope movement of earth and organic materials, result from an imbalance of forces. Various forces are involved in mass wasting. These forces are associated with the downslope gravity component of the slope mass. Resisting these downslope forces are the shear strength of the earth's materials and any additional contributions from vegetation via root strength or man's slope reinforcement activities. When a slope is acted upon by a stream or river, an additional set of forces is added. These forces are associated with removal of material from the toe of the slope, fluctuations in groundwater levels, and vibration of the slope. A slope may fail it stable material is removed from the toe. When the toe of a slope is removed, the slope loses more resistance by buttressing than it does by downslope gravitational forces. The slope materials may then tend to move downward into the void in order to establish a new balance of forces or equilibrium. Oftentimes, this equilibrium is a slope configuration with less than original surface gradient. The toe of the failed mass can provide a new buttress against further movements. However, if this buttress is removed by stream erosion, the force equilibrium may again be upset. For slope toes acted upon by erosive stream water, the continual removal of toe material can upset the force balance.

A characteristic of streams and bank slopes is the stream's influence on groundwater levels in the slope. During high flow the stream is often influent into the banks. During low flows or when the stream level drops,

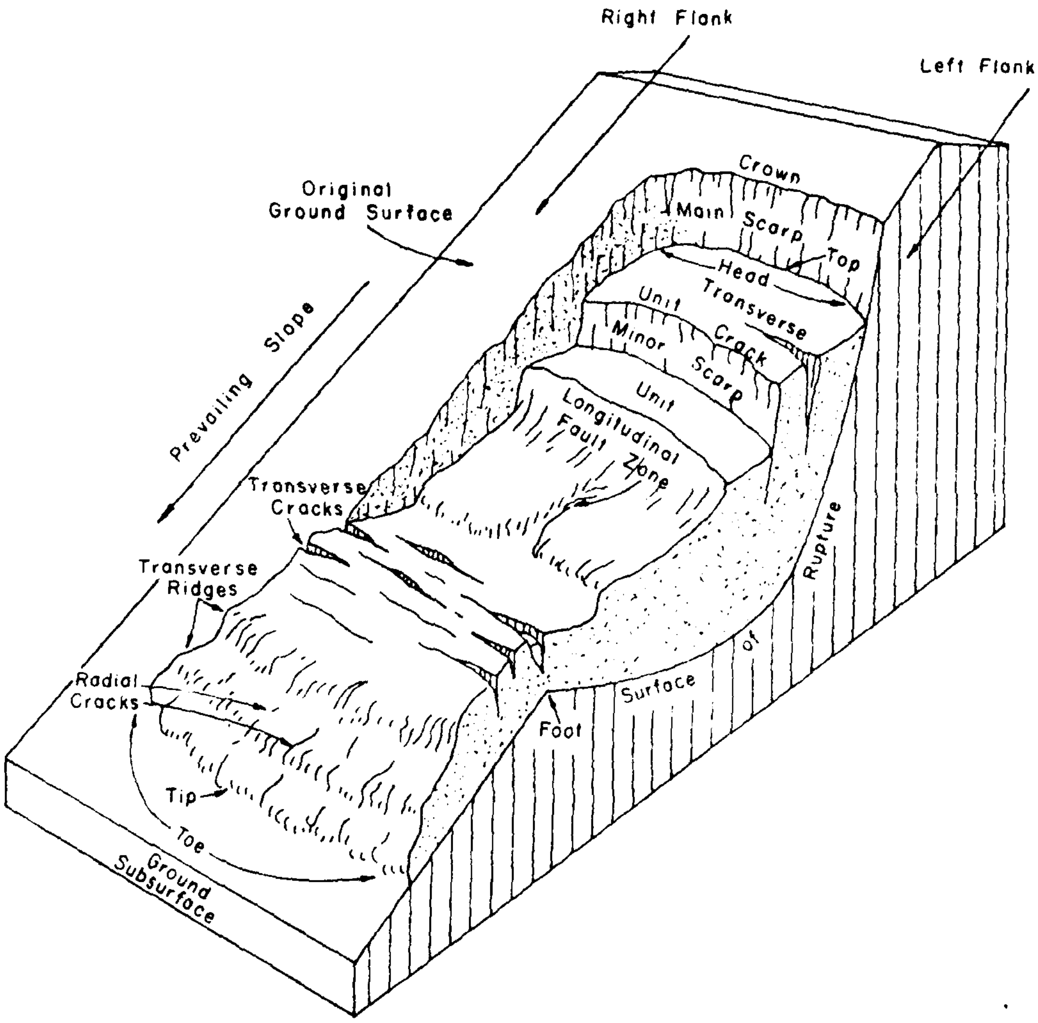


Figure 42. Rotational slide type landslide (after Varnes, 1958; Rogers, Ladwid, Hornbacker, Schwochow, Hart, Shelton, Scroggs, Soule, 1974).

water flows from the bank into the stream. In the former case, the inflowing water acts as a positive force in providing groundwater flow into the bank. In the latter case, when groundwater flow is away from the bank into the rivers, there is a reduction in bank stability. Another influence of stream activity on slope stability is through mechanical vibration of the slope mass through wave flow impingement or by striking with floating objects such as trees or ice. Although probably a minor influence, continual vibration may alter slope properties to enable other forces to produce failure. The above effects indicate why bank landslides often occur following a drop in river level. The high river level may erode the slope toe causing unfavorable groundwater conditions. In addition, vibrations during high water may cause small but important changes in the slope.

FORCES CAUSING BANK EROSION

Erosion of river banks occurs when the net result of all forces acting on the erodible material exceeds the net result of all forces tending to hold the material in place. The principal factors causing erosion of river banks are outlined below.

<u>Variables Causing Bank Erosion</u>

I. Hydraulic Parameters

   A. Fluid Properties

      1. Specific Weight
      2. Temperature/Viscosity

   B. Flow Characteristics

      1. Discharge
      2. Duration
      3. Frequency
      4. Velocity
      5. Velocity Distribution
      6. Turbulence
      7. Shear Stress
      8. Drag Force
      9. Lift Force
      10. Momentum Force

II. Characteristics of Bed and Bank Material

   A. Bed Material

      1. Size
      2. Gradation
      3. Shape
      4. Specific Weight

B. Bank Material

1. Size
2. Gradation
3. Shape
4. Specific Weight

III. Characteristics of the Banks

A. Noncohesive
B. Cohesive
C. Stratified
D. Rock
E. Height

IV. Subsurface Flows

A. Wave Forces
B. Seepage Forces
C. Piping

V. Wind Waves

A. Wave Forces
B. Surface Erosion
C. Piping

VI. Boat Waves

A. Wave Forces
B. Surface Erosion
C. Piping

VII. Climatic Factors

A. Freezing

1. Ice Thickness
2. Duration
3. Frequency and Duration

B. Thawing
C. Permafrost

VIII. Biological Factors

A. Vegetation

1. Trees
2. Shrubs
3. Grass

B. Animal Life

IX. Man-Induced Factors

A. Pool Fluctuations Caused by Power Generation
B. Agricultural Activities
C. Mining
D. Transportation
E. Urbanization
F. Drainage
G. Floodplain Development
H. Recreational Boating

EVALUATION OF FACTORS AND FORCES CAUSING EROSION

Hydraulic Factors

There are several hydraulic factors that affect the stability of banks. Water has specific properties that relate to hydraulic forces acting on the banks of rivers, including the specific weight and temperature or viscosity of the fluid. Both of these properties are affected by suspended sediment. The presence of suspended sediment in the flow increases the specific weight of the water-sediment mixture and increases its apparent viscosity. These characteristics of the flow directly effect the velocity, velocity distribution, shear stress, and consequently, the rate of erosion of channels.

Considering the water discharge, several aspects are important in the evaluation of bank stability. In general, a channel achieves a pseudo-equilibrium over time so that during periods of low flow there is little erosion, and the channel and segments of the banks may even experience accretion. During periods of intermediate river flows some bank erosion and some deposition occur. In this range of flows the dynamic processes that form rivers are apparent. With major flood events, major bank erosion occurs. Comprehensive literature surveys reveal that numerous experienced engineers and geologists have concluded that 90 to 99 percent of all significant bank erosion occurs during major flood events. These observations are not based upon concept or theory, but on field observation. Examples regarding the importance of water discharge and duration of major flows can be found in publications by Schumm, 1977 and Simons and Sentürk, 1977. In analyzing flow conditions in channels, it is obvious that the magnitude of the discharge in a given river system is an important factor affecting bank stability.

The duration of a particular discharge is even more important than the magnitude, except for very large floods such as those that occur infrequently during periods of intense rainfall and/or snowmelt, and flows associated with the failure of both natural and artificial obstruction, such as dams and ice jams. For example, the 1973 flood on the lower Mississippi caused significant channel modifications including several new divided flow reaches. This flood had a 30-year frequency considering magnitude and about a 90-year frequency considering flow duration.

Channel Geometry

Channel geometry affects many of the forces causing bank erosion. Also, the geometry of the cross section of a river is an excellent indicator of its erodibility and stability. In general, the Connecticut River is relatively stable as shown by Figures 43, 44, and 45 which indicate the width, depth, and width-depth ratio with discharge for several alluvial rivers. For the Connecticut River, when the width is less, depth is greater, and the width-depth ratio at flood stage is larger than for most other alluvial rivers.

The reasons for the greater stability are: 1) relatively uniform flow, 2) presence of geologic controls, 3) bank line vegetation, and 4) bank materials which are relatively resistant to erosion. Nevertheless, the erosional processes are active. About 20 percent of the bank line is experiencing some form of erosion. Figure 46 shows a natural cutoff that occurred on the Connecticut River. In rivers of this type, geomorphologists and engineers have documented that the outside banks will annually erode landward a distance about equal to the depth of flow. This is a general rule of thumb based upon averaging observed rates of bank erosion on many alluvial rivers. There are large deviations from the approximation when one considers specific sites.

Velocity

Velocity, velocity distribution in the cross section, and velocity fluctuations are closely related to discharge and are major variables causing river bank erosion. The average channel velocity can be estimated using relations such as those proposed by Manning (1891), Simons and Sentürk (1977), and others. Referring to the Manning equation,

$$U = \frac{1.486}{n} R^{2/3} S^{1/2} \quad . \tag{3-5}$$

From this equation, it can be noted that velocity is significantly affected by the resistance to flow n that varies with stage, the hydraulic radius R, and the slope of energy gradient S.

The velocity is not uniform across the river channel. In long straight reaches the thalweg meanders from side to side and is stronger near one bank than the other causing local erosion depending on the characteristics of the bank material and the position, strength, and duration of the thalweg. The thalweg position changes with discharge, time, sediment load, bank characteristics and location, and as a consequence of past erosion and deposition.

In bends of rivers, the flow impinges strongly on the outside bank subjecting it to erosion, the magnitude of which depends on the velocity, velocity distribution, shape of the channel, and characteristics of the bank material.

Tractive Force

The tractive force of concern is the drag force exerted by the impingement of flowing water and sediment on the banks. The tractive force is related to the velocity. Either velocity effects or tractive force effects can be considered, but it is not necessary to consider both. To illustrate the relation between velocity U and tractive force τ,

$$U = \frac{1.486}{n} R^{1/6} \sqrt{RS} \tag{3-6}$$

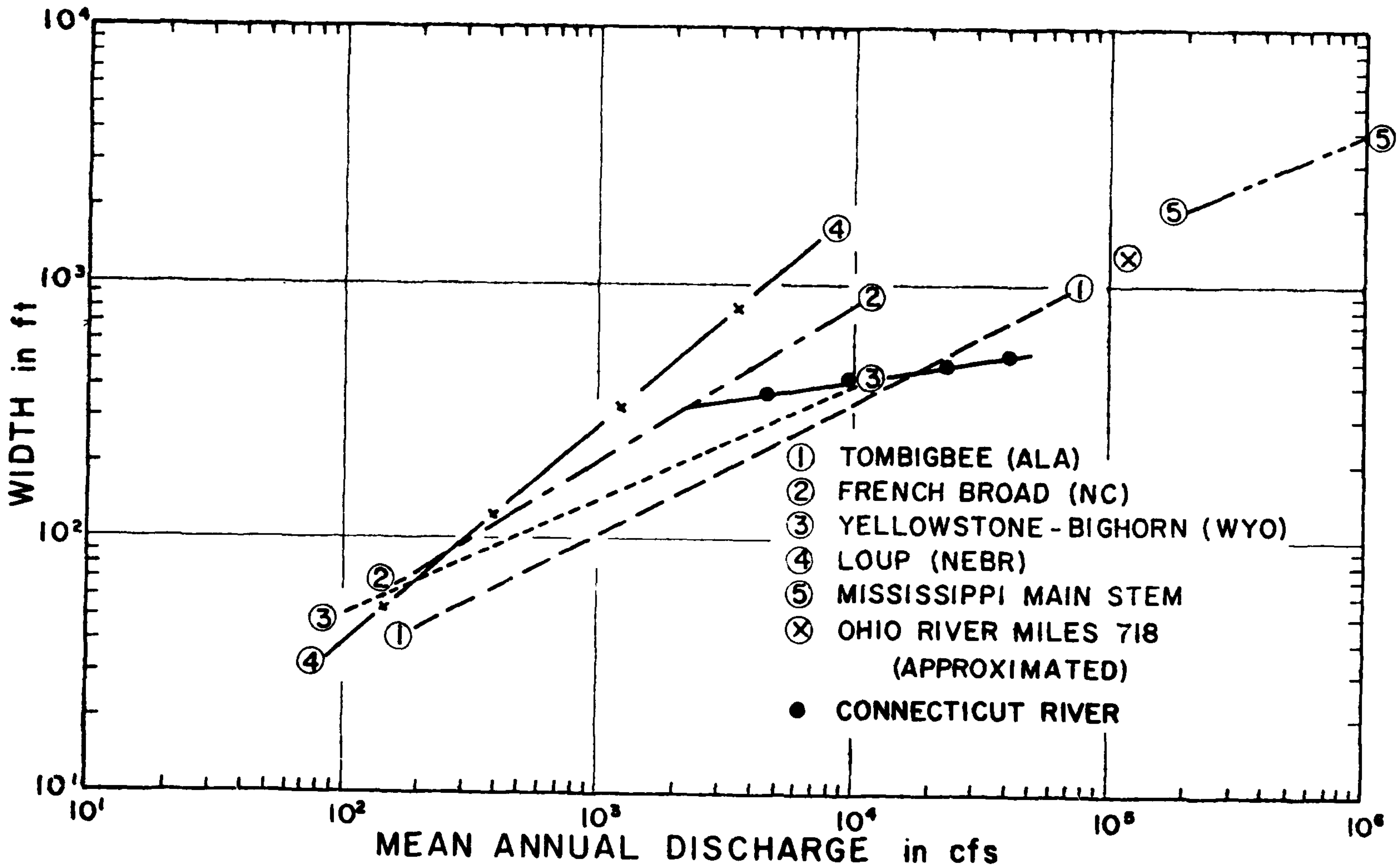


Figure 43. Width as compared to mean annual discharge curve.

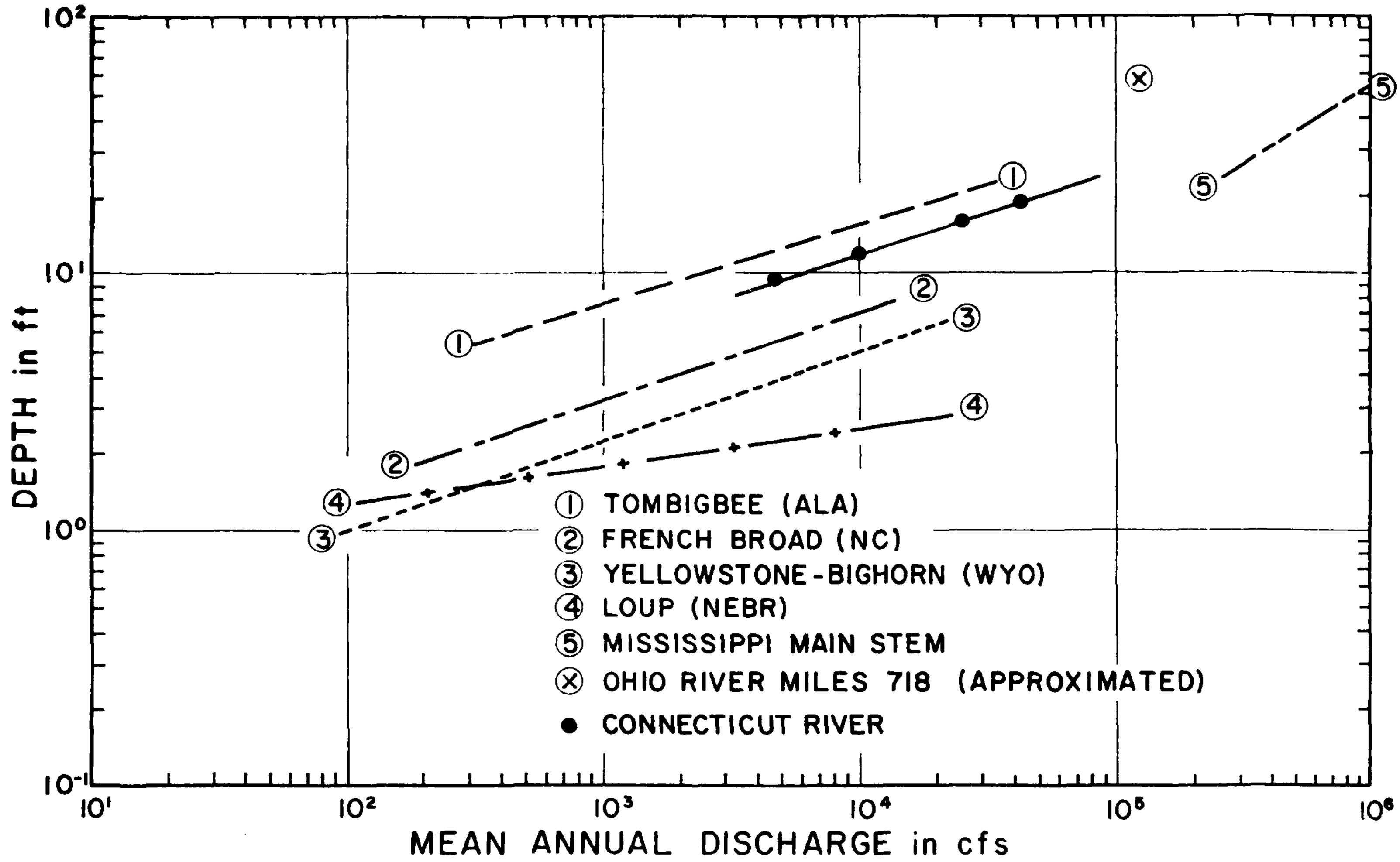


Figure 44. Depth as compared to mean annual discharge curve.

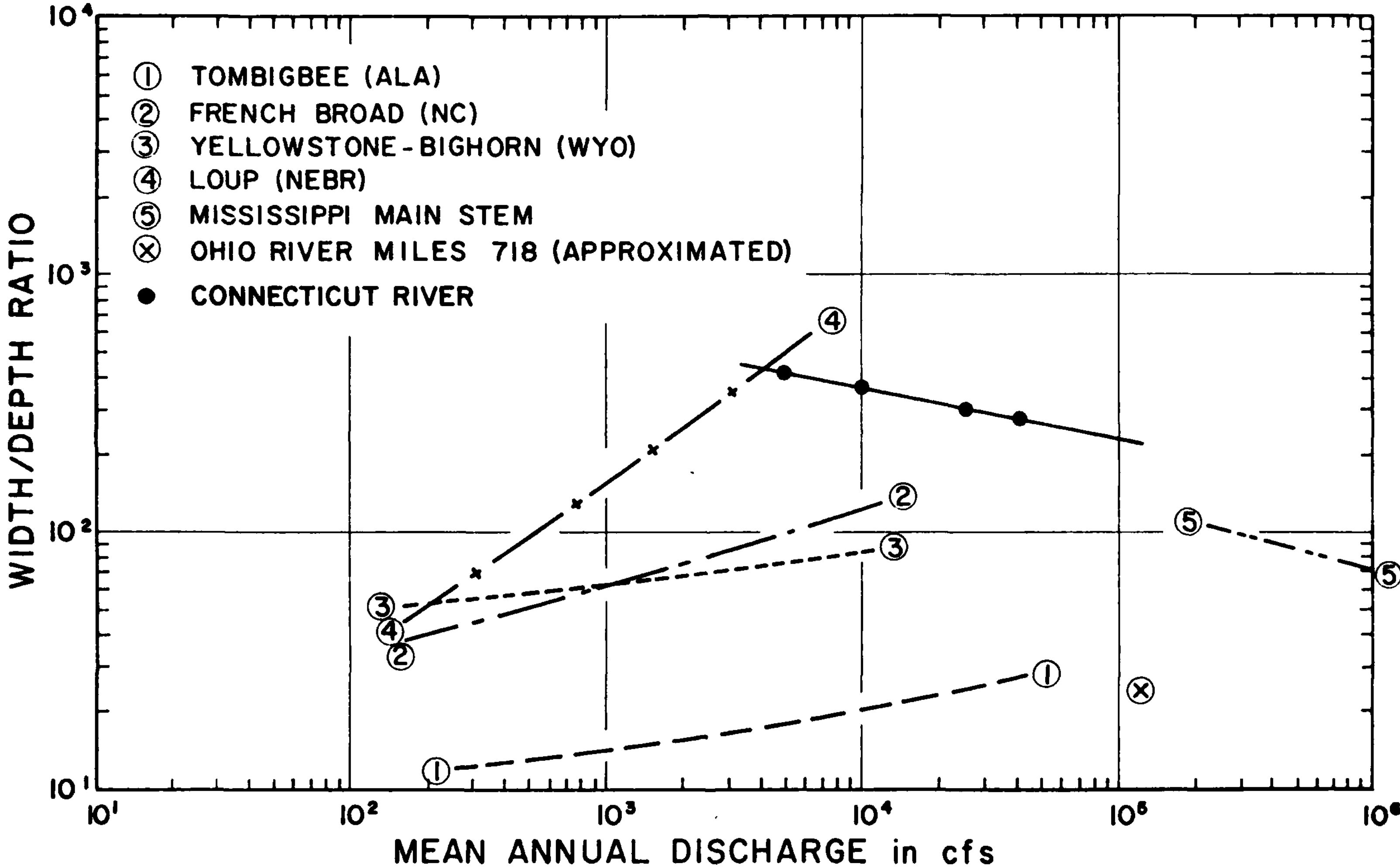


Figure 45. Width and depth ratio as compared to mean annual discharge curve.

Figure 46. Photograph showing natural cutoff.

$$U = \frac{1.486}{n\ \gamma^{1/2}}\ R^{1/6} \sqrt{\gamma RS} \qquad (3\text{-}7)$$

since

$$\tau = \gamma RS$$

Therefore,

$$U = \frac{1.486}{n\ \gamma^{1/2}}\ R^{1/6} \sqrt{\tau} \qquad (3\text{-}8)$$

and

$$\underline{\tau}\ \alpha\ \underline{U}^2 \qquad (3\text{-}9)$$

Hence, τ is a more sensitive indicator of bank erosion than velocity. The tractive force was used by the U.S Army Corps of Engineers in their analysis of bank erosion along the Ohio River.

Drag and Lift Forces

The basic equations utilized to estimate fluid lift and drag acting on a particle or an object are

$$F_D = C_D\ \rho A \frac{U^2}{2g} \qquad (3\text{-}10)$$

$$F_L = C_L\ \rho A \frac{U^2}{2g} \qquad (3\text{-}11)$$

Because of the similarity of the two relations, only one (usually the expression for fluid drag) is utilized to evaluate particle stability. However, recent studies show that to evaluate particle stability, both forces should be considered (Samad, 1978). This is particularly true when designing riprap protection for river banks.

Momentum

As water and sediment, ice and other moving objects are stopped or deflected by the river bank, bridge piers, dams, revetment works, and dikes, the mass in motion exerts a force on the object, stopping or altering its course. This force is equal to the product of the mass of the flowing or moving object multiplied by its change in velocity. Consequently, water and ice can exert significant forces on river banks and hydraulic structures, often causing partial or even total failure. For example, if a cake of ice weighing 3220 pounds moves at a velocity of 6 fps and is stopped by the bank or another structure within one second, it exerts a momentum equal to $W/g\ (\Delta v) = 3220/32.2\ (6) = 600$ lbs. With ice movement, debris, and large flows, forces resulting from their interaction with the bank line can cause accelerated erosion, damage to vegetation, and possible loss of bank protection works. It is particularly difficult to design bank protection works that will economically withstand the ravages of ice and ice flows. The riprap can actually be rafted off the banks by ice that has become attached to it as a consequence of the freeze-thaw-flow process.

Wind Waves

The wind exerts drag or stress on the water surface and this force generates waves. The magnitude and frequency of the wind generated waves are dependent on wind velocity, duration of the wind, fetch distance that is a function of the direction of the wind, orientation and surface area of the exposed water surface, depth of the water, and other minor variables. Relatively narrow channels with trees on the banks, more or less incised in the floodplain or located between nearby bluffs or hills, are normally insignificantly affected by bank erosion caused by wind-generated waves.

Wind-waves are generated by the interaction between the air and the water where they interface. The amplitude of these surface waves are a function of wind velocity, fetch distance, depth of flow, and boundary effects. More specifically, fetch distance is the distance over which the wind can interact with the open water to produce waves.

Wave heights on lakes can be predicted using Equation 3-12 (U.S. Army Coastal Engineering Research Center, 1973) as

$$H = 0283 \frac{U^2}{g} \tanh[0.578 \left(\frac{gd}{U^2}\right)^{0.75}] \tanh \frac{0.0125\left(\frac{gF}{U^2}\right)^{0.42}}{\tanh [0.578\left(\frac{gd}{U^2}\right)^{0.75}]} \tag{3-12}$$

where H = height of wave from crest to trough,

U = wind speed,

g = gravitation and acceleration,

F = fetch length, and

d = the water depth.

Figures 47, 48, 49 and 50 were developed from Equation 3-12 to determine wave height. It is interesting to note the importance of fetch distance in these charts. Assuming that wind speed is equal to 20 mph = 29.3 fps, fetch is equal to 1 mile = 5280 feet, and depth is equal to 20 feet, the wave height can be calculated from Equation 3-12 as

$$H = 0.283 \frac{(29.3)^2}{32.2} \tanh \quad 0.578 \quad \frac{32.2 \times 20}{(29.3)^2}^{0.75}$$

$$\tanh \frac{0.0125 \quad \frac{32.2 \times 5280}{29.3}^{0.42}}{\tanh \quad 0.578 \quad \frac{32.2 \times 20}{29.3^2}^{0.75}}$$

$$= 0.283 \times 26.7 \times \tanh [0.466]$$

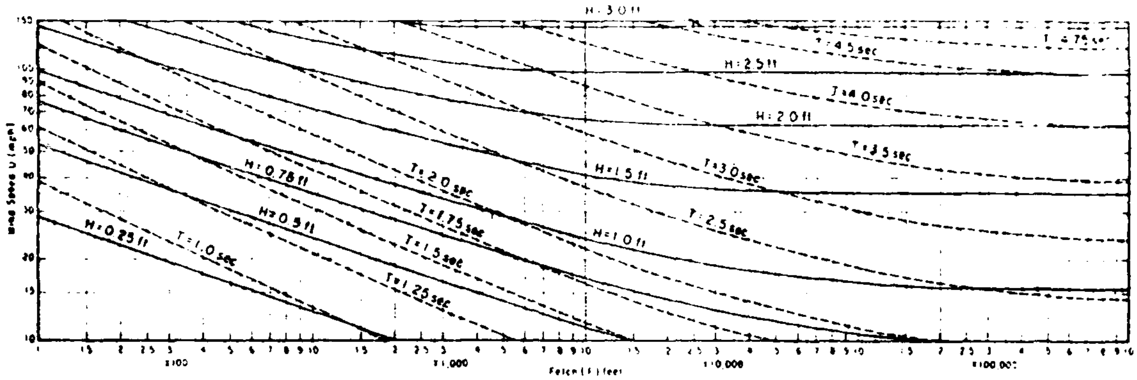


Figure 47. Forecasting curves for shallow-water waves--constant depth = 5 feet (after U.S. Army Coastal Engineering Research Center, 1973).

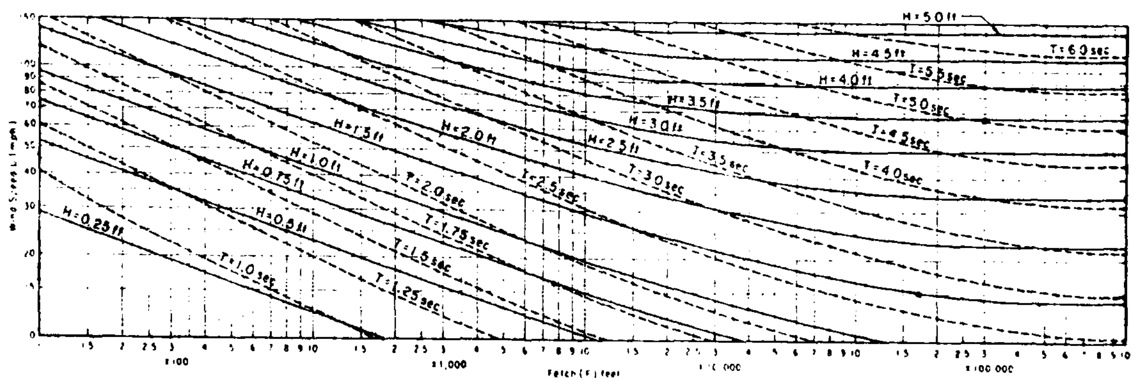


Figure 48. Forecasting curves for shallow-water waves--constant depth = 10 feet (after U.S. Army Coastal Engineering Research Center, 1973).

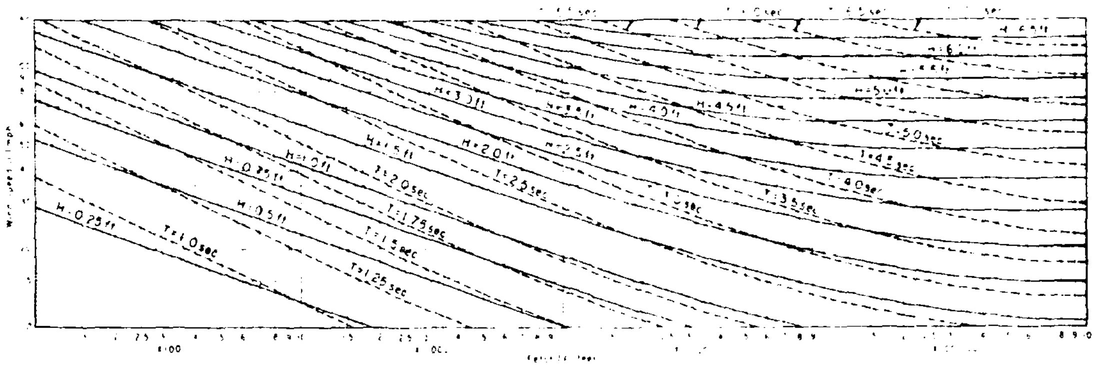


Figure 49. Forecasting curves for shallow-water waves--constant depth = 15 feet (after U.S. Army Coastal Engineering Research Center, 1973).

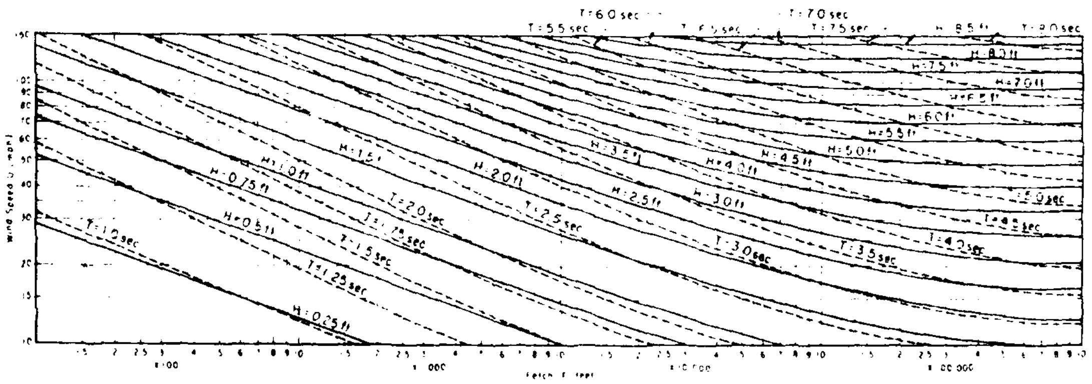


Figure 50. Forecasting curves for shallow-water waves--constant depth = 20 feet (after U.S. Army Coastal Engineering Research Center, 1973).

$$\tanh \frac{0.115}{\tanh [0.466]}$$

$$= 7.56 \times 0.435 \times \tanh \frac{0.115}{0.435}$$

$$= 7.56 \times 0.435 \times 0.258 = 0.849 \text{ feet} \tag{3-13}$$

Approximately the same wave height can be estimated from Figure 49, i.e., H is equal to 0.94 feet.

In general, wind-generated waves are small on the Connecticut River, but during periods of strong winds the waves have sufficient energy to erode exposed bank lines downstream of long fetch distances, resuspend fine sediment in the shallow areas, and possibly adversely affect bank vegetation. Wind-waves are, however, a relatively insignificant cause of bank erosion on the Connecticut River in the reach under consideration.

Boat Waves

The surface waves generated by boats are quite different from those generated by the wind. A review of the literature has identified the existence of considerable data pertinent to the evaluation of effects of boating in rivers. The waves thus generated can significantly affect bank stability depending on the size, shape, and speed of the boat, the frequency of boating, and the location or position of the speeding boats relative to the channel banks. For example, Bhowmik (1968) stated that in order to reduce wave damage to river banks and bank protection works, a no-boating zone at least 100 feet wide should be established adjacent to each bank.

As with wind-generated waves, relations are available from which the characteristics of boat-generated waves can be estimated for a wide range of conditions. In fact, detailed measurements of waves generated by boats were gathered during 1976 and 1977 on the Ohio River. The boats utilized ranged from pleasure crafts to large barges; the latter is not relevant to the impact of boating on the Connecticut River.

Erosional processes caused by boat waves are similar to those caused by wind-waves. The principle causes of erosion are:

1. The impact of the wave on the bank,

2. The wave wash on the bank caused by the wave riding up the bank and then running back down the bank line into the channel, and

3. The rise and fall of the water surface causing an increase and then decrease in water surface elevation. Even though this change in elevation occurs rather quickly, the wave phenomenon causes a measurable inflow and outflow of water in the permeable zones sandwiched between less permeable layers of bank material. This surging flow can cause piping to occur that weakens the bank, therefore increasing failure, as described previously.

Climatic Factors

The climatological region encompassing the Connecticut River experiences a wide temperature range during the year. During the winter months the banks of the river are subjected to freezing, subsequent thawing, and ice effects. During the freeze-thaw cycles, portions of unprotected banks may be subjected to frost heaving. This is a phenomenon involving the exposed surface layer of the banks. The uppermost layer of the soil freezes including the water in the pores. Below the surface layer the bank materials may be coarser or finer. If finer, the pore spaces are smaller. The water in the smaller pores freezes at a lower temperature and water moves to the freezing line where a layer of ice is formed. This ice layer grows with time, pushing the overlaying layer of soil upward. As the soil thaws it settles back toward its original position in a loosened state that is easily eroded. On an inclined bank, the formation of ice layers thrust the overlaying bank material outward. When thawing occurs, the loosened and displaced material can slump, slide, or fall down the bank. Theoretically, one freeze-thaw cycle can cause displacement of the soil. However, Heidmann (1974) utilized time-lapse photography to show that heaving of soils is usually the result of a series of freeze-thaw cycles.

Heaving of pure kaolinite clay is shown in Figure 51. Heaving of soils is a function of:

1. The rate at which water flows to the freezing front in the soil depending on pressure differences resulting from supercooling of the soil water,

2. Permeability of the soil that determines the rate of flow of water in the bank material (soils most susceptible to heaving are in the silt range),

3. Soil particle size which influences permeability and pore size, and

4. The nature of the particles (colloidal-sized clay materials segregate very little during freezing and thawing cycles).

If there is a justifiable need, frost action can be controlled to some degree by utilizing dispersing agents, waterproofing agents, cementing agents, salts, and nucleating agents. In general, mitigating frost heaving involves reducing the freezing point of the soil water, reducing flow of water through the soils, and cementing of soil particles to avoid supercooling of the soil water.

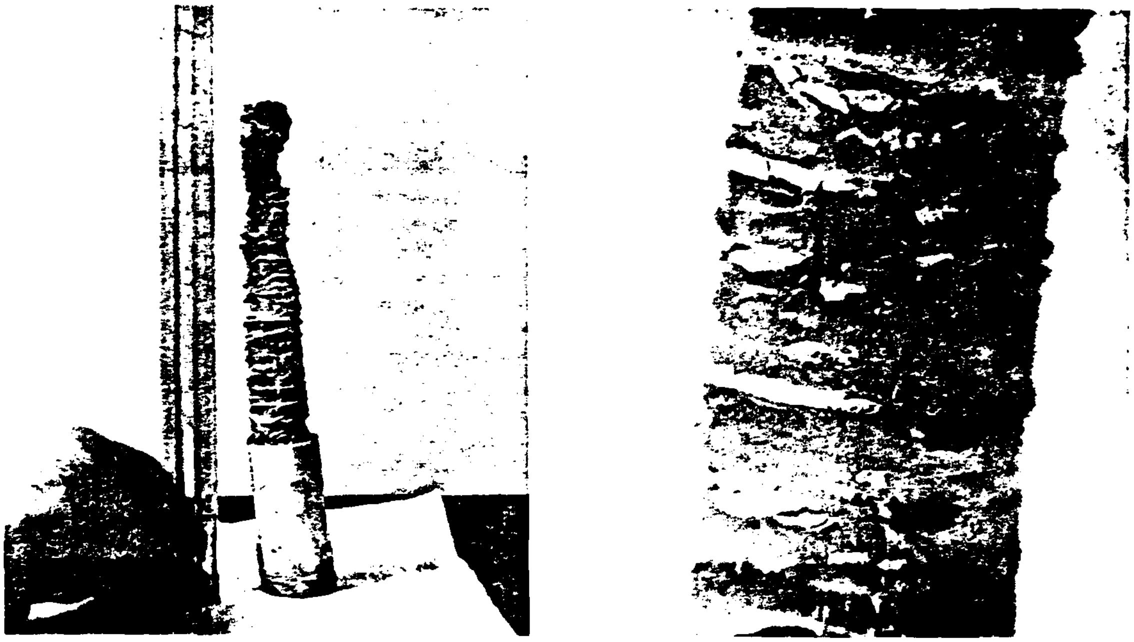


Figure 51. Example of ice lens formation in pure kaolinite clay. The sample heaved 15 cm (200 percent) in 7 days.

SECTION 4

EVALUATION OF THE MAGNITUDE OF THE FORCES CAUSING BANK EROSION

EVALUATION OF FORCES

The tractive force method described by Chow (1959) was utilized by the New England Division of the U.S. Army Corps of Engineers to evaluate bank erosion on the Connecticut River. This method is basically sound and has been widely used to design and evaluate the stability of alluvial channels. The method as proposed and utilized by the U.S. Army Corps of Engineers is limited in its application to straight reaches of channel that are principally affected by the shear stress exerted on the bed and banks by the flowing water. Considering the Connecticut River environment, the banks of the channel are subjected to additional forces as identified in Section 3 and Table 2. The analysis made in this study considers all significant factors including the tractive force and effective durations based upon the physical system. This is different from the tractive force method, which considers the tractive force alone. Typical average depth, average velocity and average sediment transport rates for the natural river, upper pools, and lower pools are given in Figures 52, 53, and 54 respectively. The definition of a natural river is one that is usually unaffected by backwater curves caused by the downstream dam. The computed depth is roughly within ten percent of the normal depth of the river. Figure 54 indicates that the sediment transporting capacity is significantly reduced in the pools. These figures were determined by assuming a dam height of 30 feet, a channel bed slope of 0.00029, and a typical channel cross section as given in Figure 35 (page 47).

In Table 2 magnitudes of the forces acting on the river banks are expressed as a percentage of the tractive force exerted on the bed of the channel by the flowing water.

In this table, the relative magnitude (M_B) and relative duration (D_B) of the forces causing bank erosion for non-cohesive and stratified bank materials have been assessed qualitatively and rated from 1 to 9 in ascending order of estimated effect. This qualitative assessment was accomplished through examination of available data, review of current theory, personal experience, and sound professional judgment. The shear stress acting on a non-cohesive bank within a reach of natural river is considered the most significant force exerted upon that bank and as such is rated as 9. The least significant effect under a similar river condition is the freeze-thaw action, and it is ranked as 1. In relation to this scale, other factors causing bank erosion in the Connecticut River have been rated accordingly. A similar rating scale has then been established with regard to the relative duration of these forces. The shear stress is acting as long as the water is flowing and is rated as 9 in the relative duration. Freezing and thawing effect is usually active for a short period of time and is rated as 1.

From these two rating scales, the relative magnitude of bank erosion (R_B) has been defined as $R_B = (M_B)(D_B)$ for both the non-cohesive and stratified bank conditions. The resulting values have then been standardized to the shear stress acting on a non-cohesive bank within a natural river reach. As an example of this table, to determine the relative effect of pool fluctuations upon low, stratified banks within the operation limits of a pool, the M_B

Table 2. Evaluation of the causes of bank erosion.

Factors & Variables Causing Bank Erosion (1)	River Condition (2)	Relative Magnitude of forces: Noncohesive (3)	Relative Magnitude of forces: Stratified (4)	Relative Duration of Forces (5)	Relative Magnitude of Bank Erosion: Noncohesive* (6)	Noncohesive* (7)	Relative Magnitude of Bank Erosion: Stratified** (8)	Stratified** (9)
Shear Stress or Velocity	Natural river with low banks	9	8	9	81	1.0	72	1.0
	Natural river with high banks	9	8	9	81	1.0	72	1.0
	Pools; low banks	9	8	7	63	.78	56	.78
	Pools; low banks with vegetation	7	6	6	42	.52	36	.50
	Pools; high banks	8	7	7	56	.69	49	.68
	Pools; high banks with vegetation	6	5	6	36	.44	30	.42
Flood Variation	Natural river with low banks	2	2	1	2	.02	2	.03
	Natural river with high banks	2	2	1	2	.02	2	.03
	Pools; low banks	2	2	1	2	.02	2	.03
	Pools; low banks with vegetation	1	1	1	1	.01	1	.01
	Pools; high banks	2	2	1	2	.02	2	.03
	Pools; high banks with vegetation	1	1	1	1	.01	1	.01
Stage Variation	Natural river with low banks	3	3	2	6	.07	6	.08
	Natural river with high banks	4	3	2	8	.10	6	.08
	Pools; low banks	3	3	1	3	.04	3	.04
	Pools; low banks with vegetation	2	2	1	2	.02	2	.03
	Pools; high banks	5	4	1	5	.06	4	.06
	Pools; high banks with vegetation	3	3	1	3	.04	3	.04
Pool Fluctuations	Natural river with low banks	3	3	2	6	.07	6	.08
	Natural river with high banks	4	3	2	9	.11	6	.08
	Pools; low banks	4	3	3	12	.15	9	.13
	Pools; low banks with vegetation	3	3	3	9	.11	9	.13
	Pools; high banks	5	4	3	15	.19	12	.17
	Pools; high banks with vegetation	4	4	3	12	.15	12	.17

*Standardized values based on the shear stress on noncohesive bank and natural river. For example, for condition of pool, low bank, 63/81 = 0.78.

**Standardized values based on the shear stress on stratified bank and natural river. For example, for condition of pools, low banks, 56/72 = 0.78.

Table 2. (continued).

Factors or Variables Causing Bank Erosion	River Condition	Relative Magnitude of Forces		Relative Duration of Forces	Relative Magnitude of Bank Erosion			
		Noncohesive	Stratified		Noncohesive*		Stratified**	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Wind waves surface erosion & piping	Natural river with low banks	2	2	1	2	.02	2	.03
	Natural river with high banks	1	1	1	1	.01	1	.01
	Pools; low banks	3	3	2	6	.07	6	.08
	Pools; low banks with vegetation	2	2	1	2	.02	2	.03
	Pools; high banks	2	2	1	2	.02	2	.03
	Pools; high banks with vegetation	1	1	1	1	.01	1	.01
Boat Waves surface erosion & piping	Natural river with low banks	2	3	2	4	.05	6	.08
	Natural river with high banks	2	2	2	4	.05	4	.06
	Pools; low banks	3	4	2	6	.07	8	.11
	Pools; low banks with vegetation	3	3	2	6	.07	6	.08
	Pools; high banks	4	5	2	8	.10	10	.14
	Pools; high banks with vegetation	3	4	2	6	.07	8	.11
Freeze-Thaw	Natural river with low banks	1	1	1	1	.01	1	.01
	Natural river with high banks	1	1	1	1	.01	1	.01
	Pools; low banks	1	1	1	1	.01	1	.01
	Pools; low banks with vegetation	1	1	1	1	.01	1	.01
	Pools; high banks	1	1	1	1	.01	1	.01
	Pools; high banks with vegetation	1	1	1	1	.01	1	.01
Ice	Natural river with low banks	2	2	1	2	.02	2	.03
	Natural river with high banks	3	2	1	3	.04	2	.03
	Pools; low banks	2	2	1	2	.02	2	.03
	Pools; low banks with vegetation	1	1	1	1	.01	1	.01
	Pools; high banks	2	2	1	2	.02	2	.03
	Pools; high banks with vegetation	1	1	1	1	.01	1	.01
Seepage Forces	Natural river with low banks	2	3	2	4	.05	6	.08
	Natural river with high banks	3	3	2	6	.07	6	.08
	Pools; low banks	2	3	2	4	.05	6	.08
	Pools; low banks with vegetation	2	3	2	4	.05	6	.08
	Pools; high banks	3	4	2	6	.07	8	.11
	Pools; high banks with vegetation	2	3	2	4	.05	6	.08
Gravitational Forces	Natural river with low banks	2	2	2	4	.05	4	.06
	Natural river with high banks	3	4	3	9	.11	12	.17
	Pools; low banks	2	2	2	4	.05	4	.06
	Pools; low banks with vegetation	1	2	1	1	.01	2	.03
	Pools; high banks	3	4	3	9	.11	12	.17
	Pools; high banks with vegetation	2	3	2	4	.05	6	.08

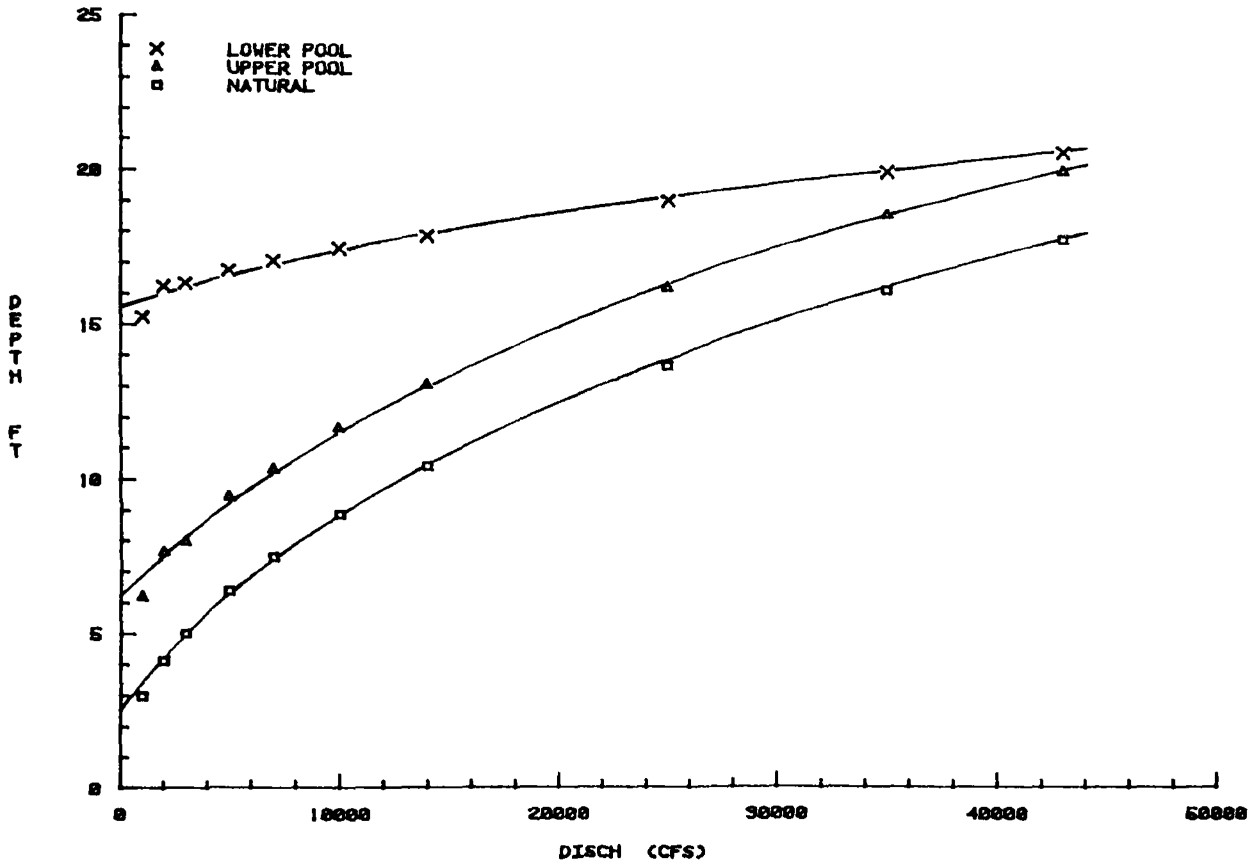


Figure 52. Water discharge as compared to depth for typical reaches.

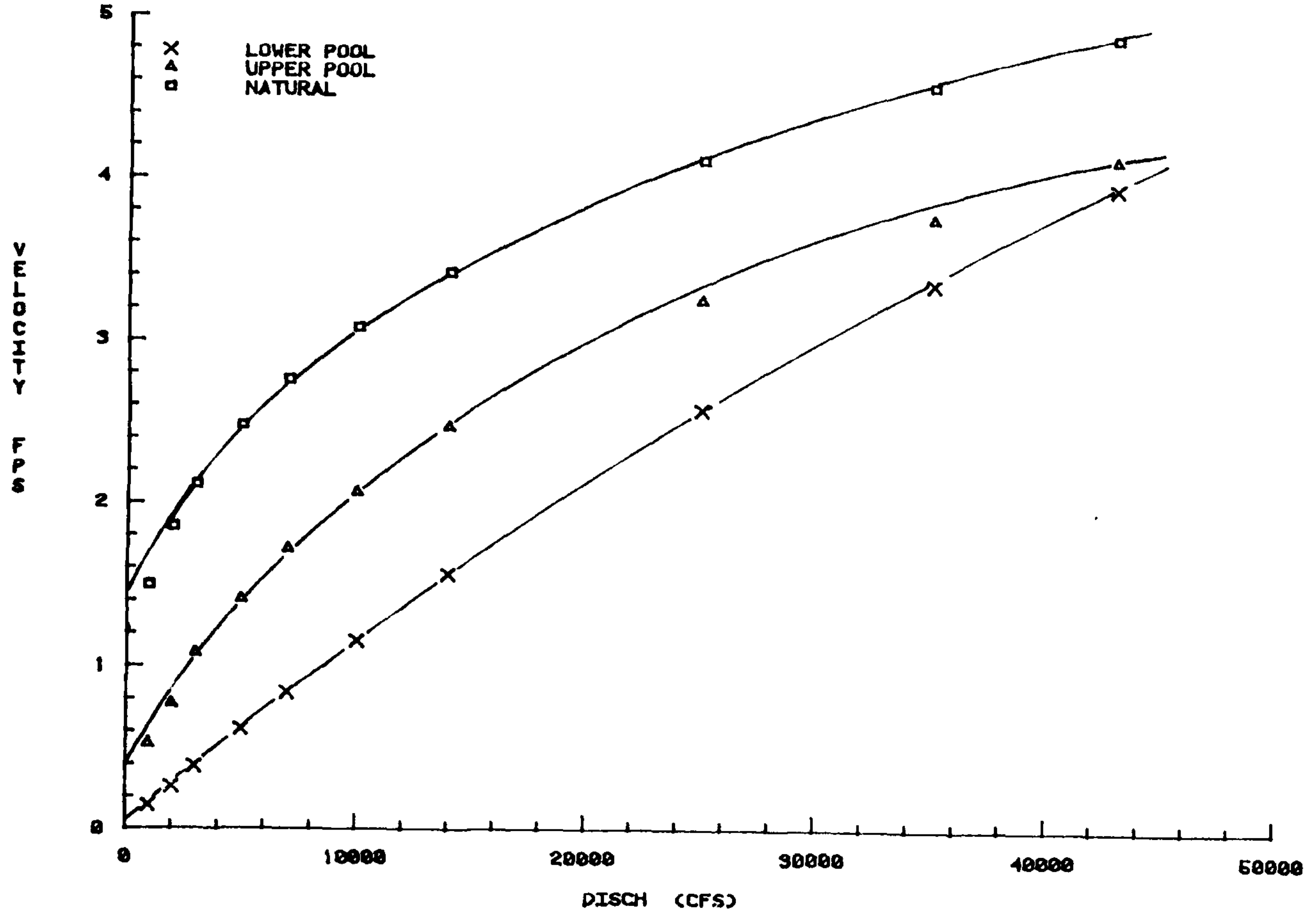


Figure 53. Water discharge as compared to velocity for typical reaches.

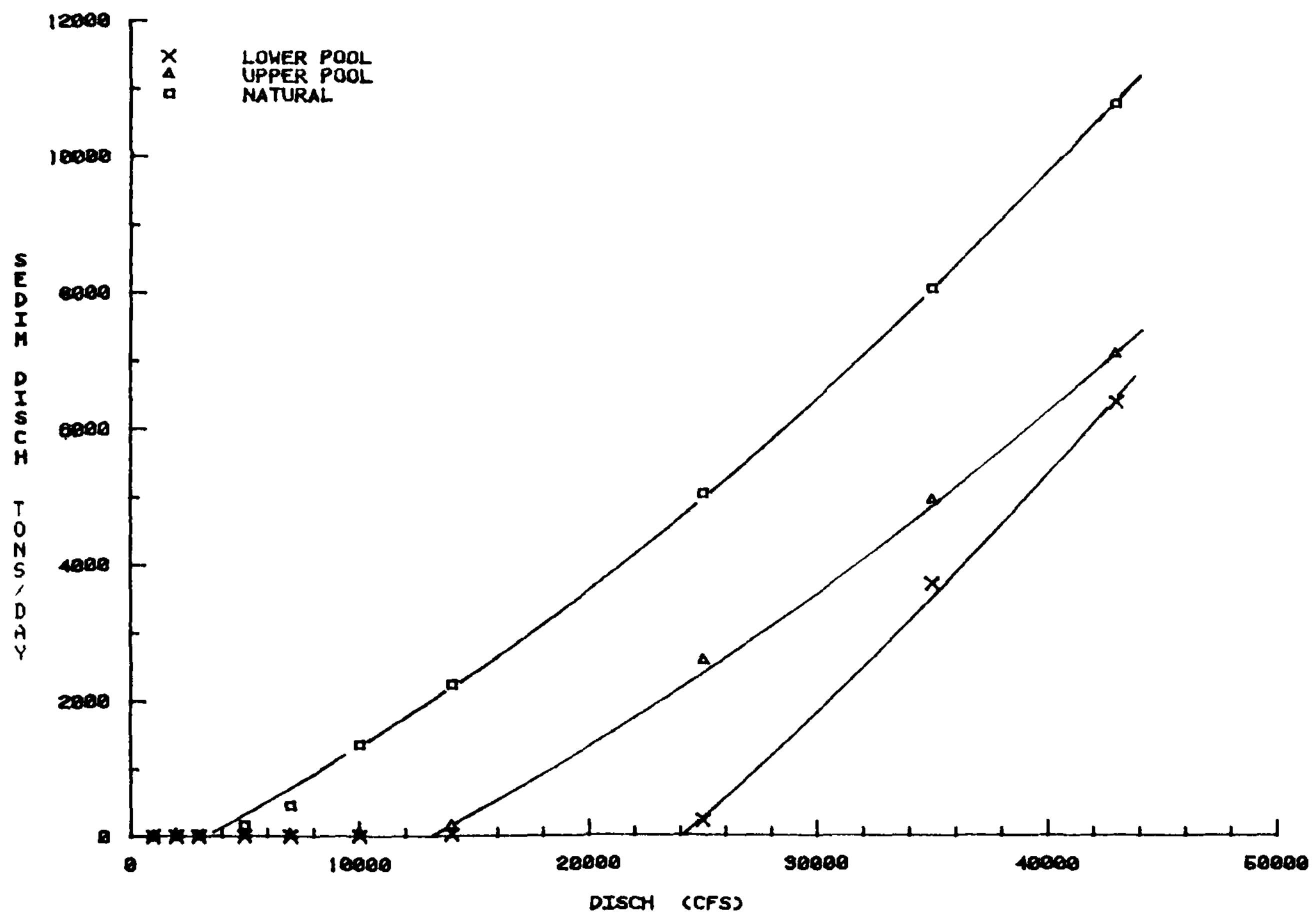


Figure 54. Water discharge as compared to sediment discharge for typical reaches.

value is 3 (Column 4), the D_B value is 3 (Column 5), and the resulting relative magnitude of bank erosion (R_B) is (M_B) x (D_B) = 9 (Column 8). The relative magnitude of the bank erosion for a stratified bank within a natural river reach is 72 and the standardized value is thus $\frac{9}{72} \cong 0.13$ (Column 9). The tractive force can be approximated by the relation

$$\tau = \gamma d S \qquad (4\text{-}1)$$

where τ = stress,

γ = specific weight of the water-sediment mixture,

d = depth of flow at the location where the shear stress is to be estimated, and

S = slope of energy gradient

As an alternative, the shear stress acting on the channel boundary can be estimated by the relation

$$\sqrt{\tau/\rho} = \frac{u_1 - u_2}{5.75 \log \frac{y_1}{y_2}} \qquad (4\text{-}2)$$

where ρ is equal to the density of water and u_1, u_2 are equal to the point velocities measured at distances y_1 and y_2 from the boundary of the channel.

For best results, y_1 and y_2 should be as close as possible to the boundary. This relation provides a better approximation of the boundary shear stress in reaches affected by power dams since shear stress is independent of the channel and is sensitive to the velocity distribution.

To estimate the stability of a river channel the critical shear stress, which is just sufficient to initiate movement of bed material, is first determined. This critical shear stress can be approximated from the Shields Diagram presented in Figure 55.

In Figure 55, D_s is the particle size, U_* is the shear velocity, ν is the kinematic viscosity, $\Delta\gamma$ is the difference between specific weight of sediment and water, and Re is the particle Reynolds number. When Re > 500, the term $\tau/\Delta\gamma D$ has a constant value equal to 0.06. Hence, knowing $\Delta\gamma$ and D, the critical shear stress τ_c can be estimated. If the shear stress acting on the bed of the river, as determined by Equations 4-1 and 4-2, is greater than the critical shear stress τ_c, the bed material will be in motion. If the rate of transport on a segment of the river exceeds the inflow of sediment from upstream, degradation of the river bed will occur, and the converse. Having estimated the shear stress acting on the bed of the river and the critical shear stress, the next step is to estimate the critical shear stress and the actual shear stress acting on the banks of the river. If the shear stress acting on the river bank exceeds the critical shear stress, bank erosion will result. Consequently, the river bank will be unstable unless protected by vegetation, riprap, or some other form of protection. The

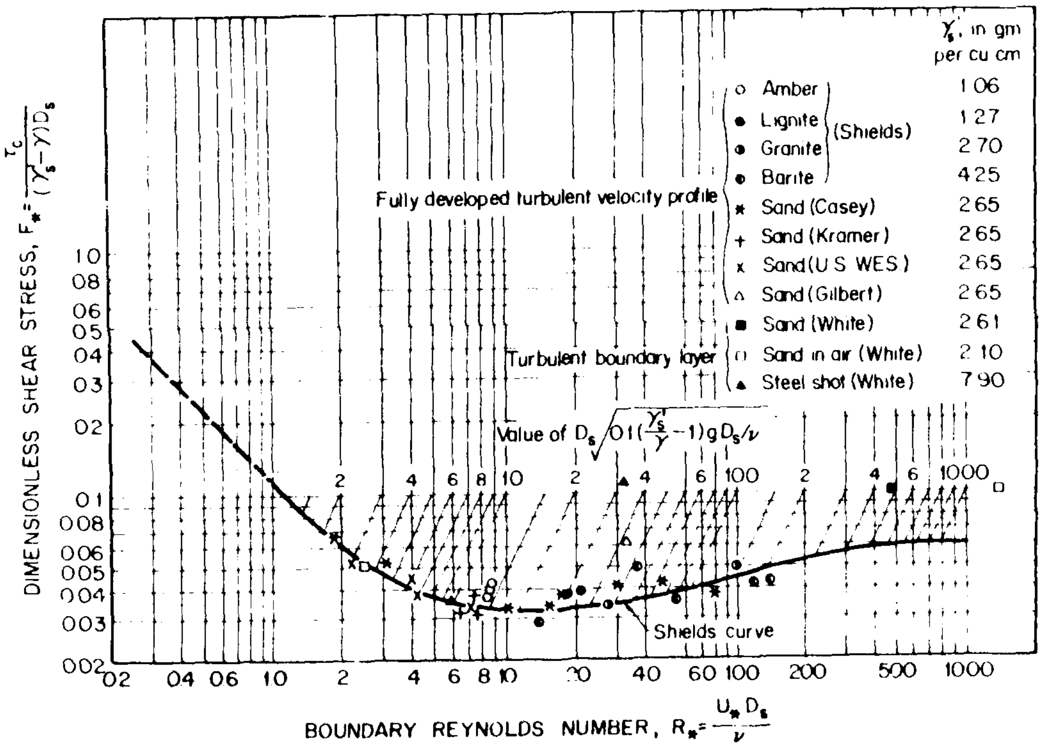


Figure 55. Shields' diagram indicating critical shear stress (after Shields, 1936).

critical shear stress for the material on the river bank can be estimated utilizing Figure 55 to give critical bed shear stress, and then reducing this value by a factor K to allow for the gravitational component of forces on the bank particles:

$$K = \frac{(\tau_s)_c}{(\tau_b)_c} = \cos\theta \sqrt{1 - \frac{\tan^2\theta}{\tan^2\phi}} \tag{4-3}$$

where $(\tau_s)_c$ and $(\tau_b)_c$ are the critical shear stresses on the side and bed respectively, θ is the angle of the side slope and ϕ is the angle of repose of the bank material which can be estimated from Figure 56.

The actual shear stress on the sides of a channel relative to the shear stress on the bed is given by Figure 57. For a wide channel the maximum value of shear stress on the bank is 0.77 times the maximum shear stress on the bed:

$$\tau_s = 0.77\ \tau_b\ . \tag{4-4}$$

As when analyzing the stability of the bed of a channel, the banks will be unstable if the tractive force acting on the banks exceeds the critical tractive force for the bank material.

Thus, only the shear stress exerted on the river banks as caused by flowing water was evaluated. Consequently, it was necessary to add effects of other forces acting on the banks to the tractive force. These additional increments of shear stress can be determined from Table 2. For example, the relative incremental values of shear stress caused by the other forces for noncohesive soils are tabulated in Table 3. The total effective tractive force acting on the bank of the river can be estimated by adding the relative incremental value of shear stress from the preceding tabulations taken from Table 2 to the shear stress exerted by the flowing water on the banks of the channel. For example, if this shear stress is 0.77 γdS and the sum of incremental values of shear stress for a natural river having high banks formed of noncohesive material is 0.52 γdS, then the total effective shear stress will be 1.29 γdS. If this value exceeds the critical shear stress acting on the banks, erosion of the banks will occur unless protected by some means.

If the bank line is being subjected to erosion, the size of riprap required to stabilize the bank line can be estimated from the Shields diagram (Figure 55). In this case the estimated effective shear stress acting on the river bank can be substituted for τ in the relation $\tau/\Delta\gamma D_s$; and if $Re > 500$ then, (Shields, 1936)

$$\frac{1.29\ \gamma dS}{\Delta\gamma D_s} = 0.06 \tag{4-5}$$

The median diameter of the required size of riprap can be approximated by solving for D_s. Considering the numerator term 1.29 γdS is not a constant, but a function of river geometry and the forces acting on the river banks, its value can be approximated for each condition or evaluated as above.

The preceding analysis applies to essentially straight reaches of river channel. This method of analysis can be extended to apply to the outside banks of a river system by considering the lateral velocity distribution

Table 3. Example of the incremental values of shear stress. (Noncohesive Banks)

Factors and Variables Causing Bank Erosion	Equivalent Shear Stress Relative to Shear Stress on Natural River Banks					
	Natural River; low banks	Natural River; high banks	Pools; low banks	Pools; low banks with vegetation	Pools; high banks	Pools; high banks with vegetation
Flood variation	.02 yds	.02 yds	.02 yds	.01 yds	.02 yds	.01 yds
Stage variation	.07	.10	.04	.02	.06	.04
Pool fluctuation	.07	.11	.15	.11	.19	.15
Wind waves	.02	.01	.07	.02	.02	.01
Boat waves	.05	.05	.07	.07	.10	.07
Freeze-thaw	.01	.01	.01	.01	.01	.01
Ice	.02	.04	.02	.01	.02	.01
Seepage forces	.05	.07	.05	.05	.07	.05
Gravitational	.05	.11	.05	.01	.11	.05
Sum	0.36 yds	0.52 yds	0.48 yds	0.31 yds	0.60 yds	0.40 yds

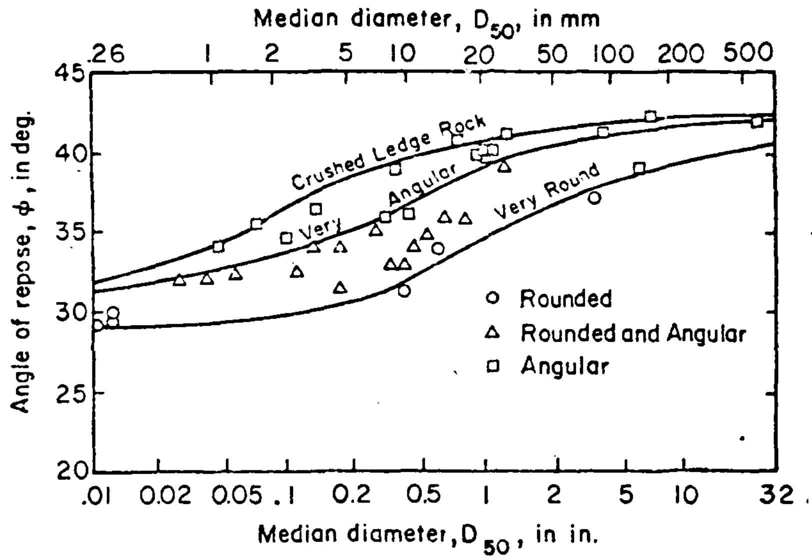


Figure 56. Angle of repose for dumped riprap (after Simons and Sentürk, 1977).

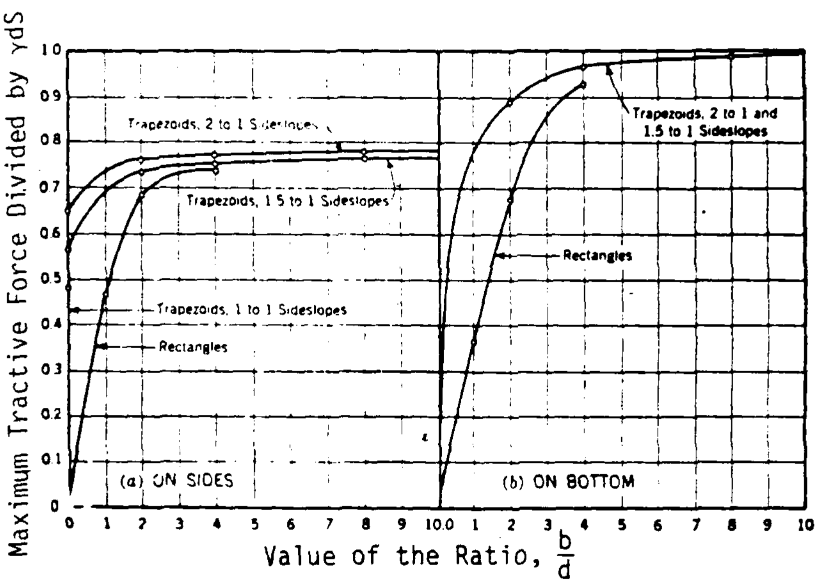


Figure 57. Variation of τ in a trapezoidal cross section (after Lane and Carlson, 1953).

resulting from the properties of the river bend. The procedure requires an estimate of the increase in boundary shear stress acting on the outside bank of a river bend.

For straight channels the flow of water is more or less evenly distributed so the velocity remains basically constant across a cross section of the stream. In a curved channel the velocity of flow is generally higher on the outside of the bend and smaller on the inside. These changes in velocity cause even larger changes in the shear stress acting on the bed and banks of the river. A graph relating the boundary shear stress in a curved reach to that of a straight reach is given in Figure 58. This figure is reproduced from "Design of Open Channels," Soil Conservation Service, 1977. This figure shows the factor by which the mean shear stress is increased for the outside of bends in curved channels. This value changes depending on the curvature of the channel. For sharp bends with a small radius of curvature, the boundary shear stress on the outside of a bend is approximately twice that of a straight channel.

To illustrate, a river bend has a radius of curvature, R_c = 1500 feet and width W = 300 feet. According to Figure 58

$$\frac{\tau_{bend}}{\tau_{straight}} = 1.5$$

For this case, the shear stress in the bend is about 1.5 times that in the straight reach. With this value established, the effect of the other forces acting on the bank can be added to the computed value to determine the total effective shear stress acting on the bank. Then the stability of the bank can be evaluated and if riprap is required, it can be sized as described in the preceding paragraphs.

Considering river bends in general, the shear stress in the bend way can be evaluated for each case. For more severe conditions, the shear stress acting on the outside bank in the bend way may be as much as 1.5 times as large as for the straight reach. Details pertaining to channel stabilization are described in the next section.

CLASSIFICATION OF BANK EROSION ON THE CONNECTICUT RIVER

Within the entire study reach, an evaluation of all the erosion sites was made to classify the erosional type and assist in the classification of the erosional forces pertinent to that particular type. In all, 103 erosion sites (locations were given in Figure 1) were selected as representative of all erosional patterns within the river and have been classified in the following manner. The primary sources of these data were the surveys conducted by the New England U.S. Army Corps of Engineers, including the photo journal that was compiled during the surveys, and in-field data collection conducted during September 1978. The U.S. Army Corps of Engineers' surveys delineated 30 areas along the Connecticut River between Turners Falls Dam (river mile 122.2) and Wells River gaging stations (river mile 266.0). These study areas represent the most severe bank erosion cases along the river. Photographs taken of these areas provide a visual quantitative record of the erosion. Other sources of data were topographic and depositional maps of the study region.

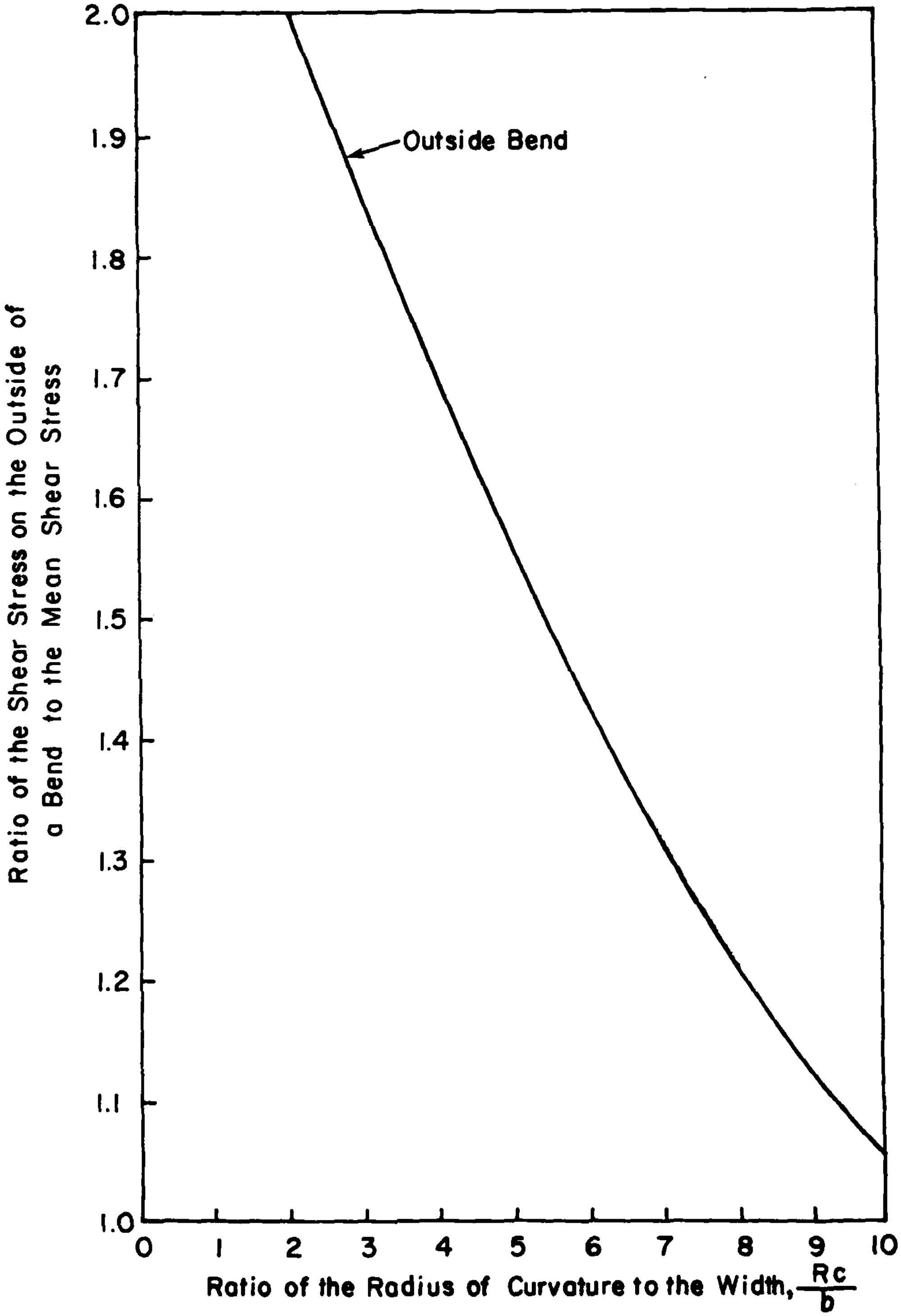


Figure 58. Effect of bend on boundary shear stress.

From these data, each study area was evaluated and classified into six different nomenclatural groups. Within these six nomenclatural groups, characteristics were delineated and subgroups established.

Bank Height

The first nomenclatural group is bank height with the following subgroups: low banks (under 15 feet) and high banks (equal to or greater than 15 feet).

Erosion Type

The second major classification group is erosion type with the following subgroups: mass wasting, head cutting, sloughing, shallow washing, and undercutting. These subgroups are shown in Figures 59 through 63. The photographs corresponding to each subgroup are given in Figures 64 through 68.

Erosion Site Location

The third nomenclatural group is the erosion site location with the following subgroups: upper pool, middle pool, lower pool, and natural reach. The pool divisions were determined on a purely mathematical basis. First, a pool was delineated as a reach of river that acts as a reservoir for water backed up by a dam. The pool was then split into thirds, yielding three pool divisions. The sections of river not delineated as being in a pool were automatically classified as natural reaches.

Bank Location

The fourth major delineation was bank location with the following subgroups: outer bend (single or divided flow), inner bend (single or divided flow), and straight reach (single or divided flow).

Soil Type

The fifth nomenclatural group was soil type, with the following subgroups: cohesive, noncohesive, and stratified. It must be noted here that no study area was composed of totally cohesive soil. Rather, those study areas that are marked as being composed of cohesive soil were comprised of a larger percentage of clays than those labeled stratified.

Vegetation

The final nomenclatural group is vegetation, with the subgroups vegetated and barren. Table 4 provides the classification of bank erosion for all 105 erosion study sites.

Application of Analysis to Index Sites

The Corps of Engineers has established six index sites for detailed study. These six index sites are Areas No. 147, 51, 31, 26, 301 and 255. Locations were given in Section 2. These index sites have been photographed documenting the rate of bank erosion. For example, refer to Figure 69. This figure shows conditions at index site 26 on October 29, 1975, and subsequently on May 17, 1977. The rate of erosion on an annual basis is not large but is

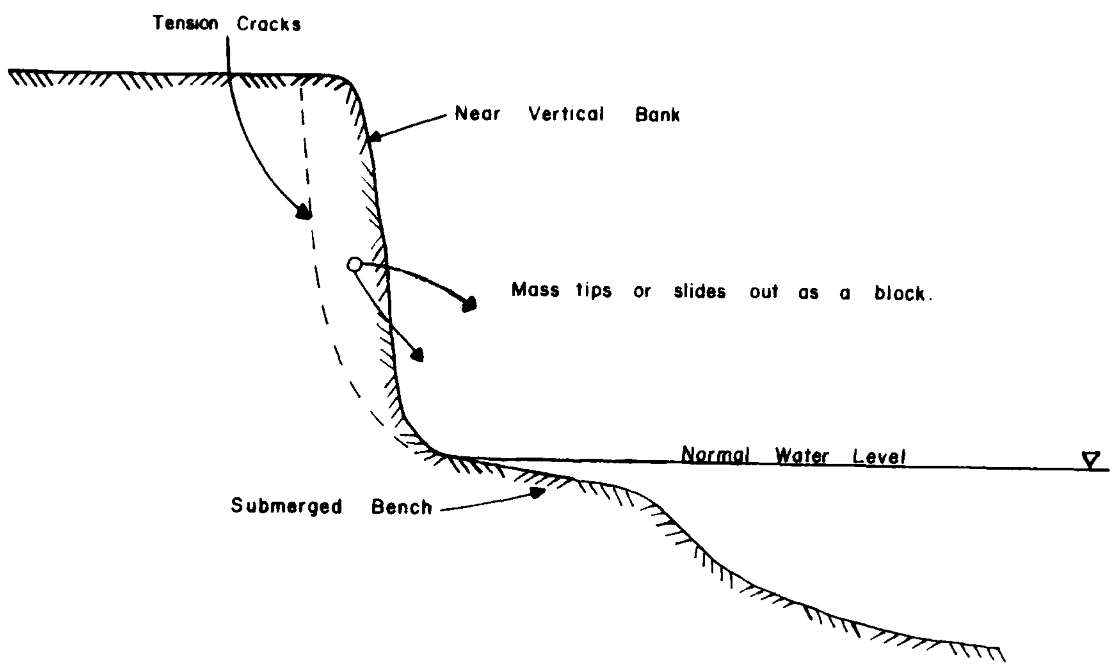


Figure 59. Mass wasting on a vegetated or barren bank.

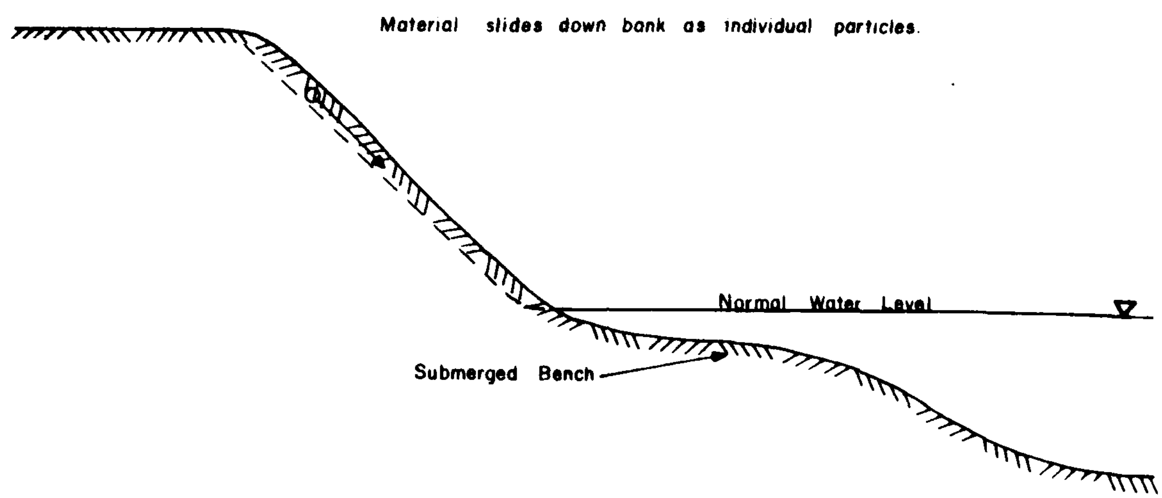


Figure 60. Sloughing on a partially vegetated or barren bank.

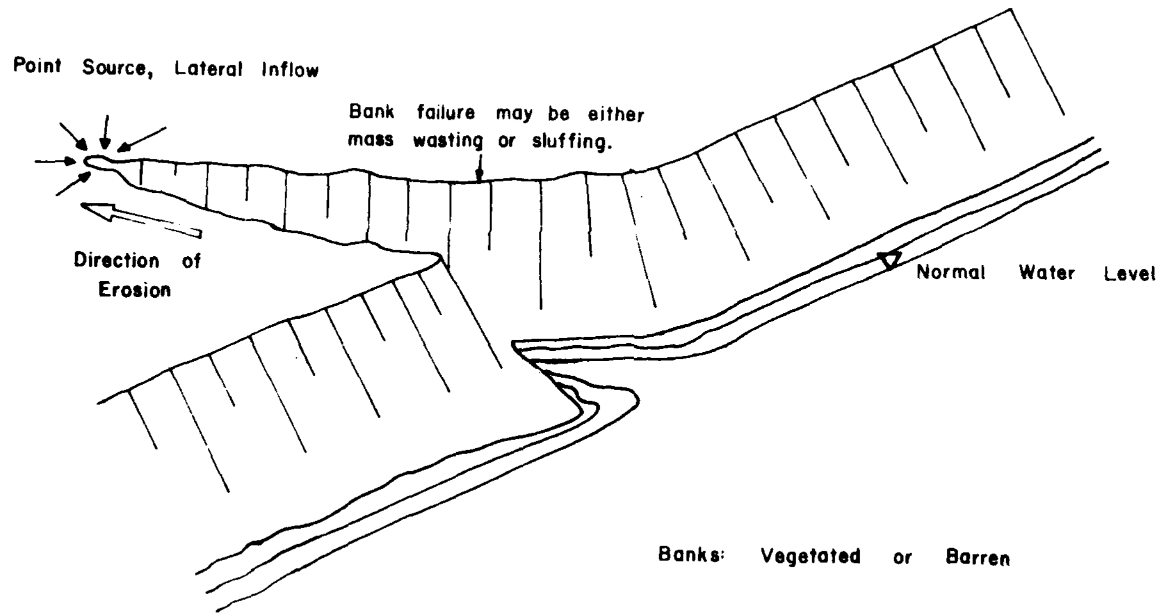


Figure 61. Headcutting on a vegetated or barren bank.

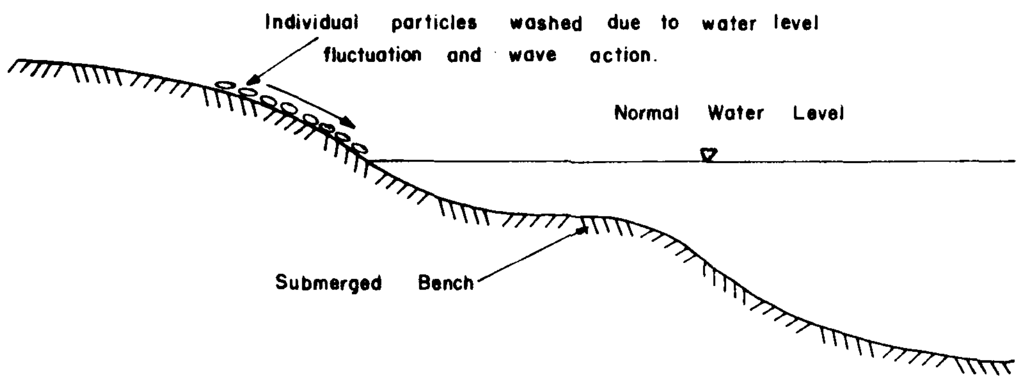


Figure 62. Shallow washing on a barren bank.

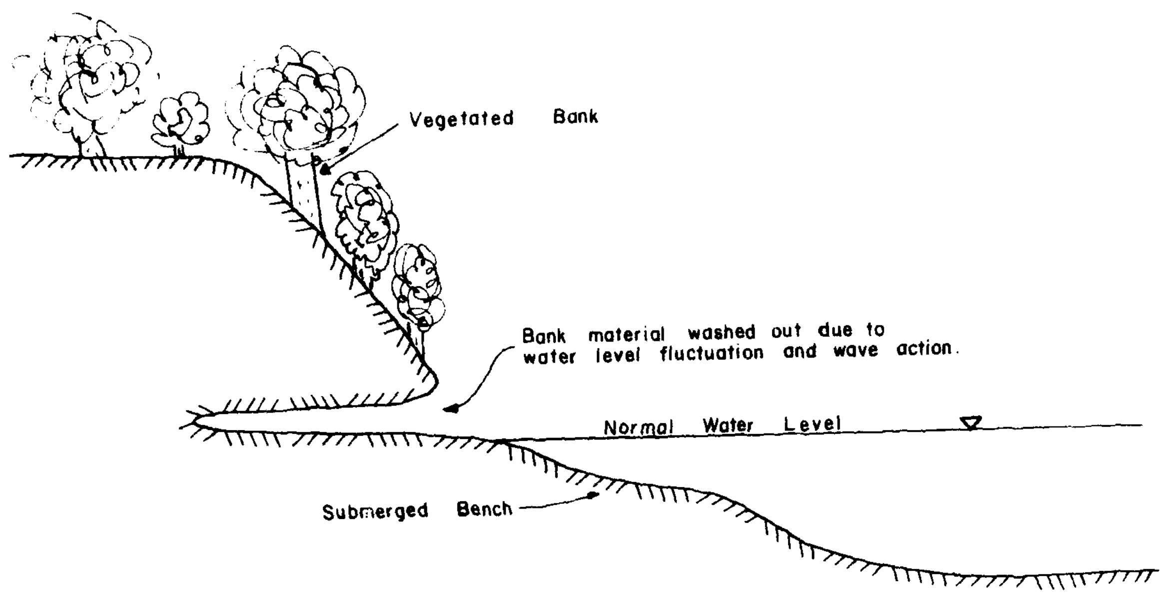


Figure 63. Undercutting on a vegetated bank.

Figure 64. Slumping or block slide mass wasting on a vegetated or barren bank.

Figure 65. Sloughing on a partially vegetated or barren bank.

Figure 66. Headcutting on a vegetated or barren bank.

Figure 67. Shallow washing on a barren bank.

Figure 68. Undercutting on a vegetated bank.

Table 4. Classification of bank erosion based on observed data.

Reach	River Mile	Station Number	Bank Height: Low	Bank Height: High	Erosion Type: Mass Waste	Erosion Type: Head Cut	Erosion Type: Sloughing	Erosion Type: Shallow Wash	Erosion Type: Under Cut	Erosion Site Location: Upper Pool	Erosion Site Location: Mid Pool	Erosion Site Location: Low Pool	Bank Location: Nat Reach	Bank Location: Outer Bend S D	Bank Location: Inner Bend S D	Bank Location: Strat Reach S D	Soil Type: Coh's	Soil Type: Non Coh's	Soil Type: Strat	Vegetated: Veg'd	Vegetated: Barren
Turners	123.5	250		X			X					X				X		X		X	
	126.5	201	X			X						X				X			X	X	
	[illegible]	[illegible]		X			X				X										X
	[illegible]	[illegible]	X				X				X				X					X	
	[illegible]	[illegible]	X				X				X					X			X	X	
	[illegible]	[illegible]	X					X			X			X				X		X	
	[illegible]	203	X		X						X					X		X		X	
	[illegible]	[illegible]		X	X						X					X			X		X
	[illegible]	[illegible]	X					X			X					X			X		
	[illegible]	[illegible]	X				X				X					X		X		X	
	[illegible]	[illegible]		X		X					X				X				X		X
	[illegible]	[illegible]	X				X			X						X		X		X	
	[illegible]	[illegible]	X				X			X				X				X		X	
Vernon	[illegible]	[illegible]		X			X					X		X				X		X	
	[illegible]	[illegible]		X		X						X				X		X		X	
	[illegible]	[illegible]	X				X					X				X		X		X	
	150.[illegible]	10	X		X							X		X					X	X	
	151.[illegible]	301		X	X						X				X				X		X
	152.[illegible]	2	X				X				X			X				X			X
	153.[illegible]	4	X				X				X			X				X		X	
	153.8	5	X				X			X					X				X	X	
	154.[illegible]	6A	X				X			X						X			X	X	
	155.6	7	X					X		X					X				X	X	
	156.3	8	X					X		X					X				X	X	
	157.2	9	X					X		X						X			X	X	
	158.3	11	X					X		X					X				X	X	
	159.[illegible]	12	X					X		X					X				X	X	
Natural	161.[illegible]	202	X					X					X		X		X				X
	161.[illegible]	13	X					X					X			X			X	X	
	163.[illegible]	204	X				X						X			X			X	X	
	165.15	17		X			X						X			X			X	X	
	165.[illegible]	205		X			X						X			X		X		X	
	166.0	17A		X			X						X	X					X	X	
	166.[illegible]	206	X				X						X		X			X		X	
	167.[illegible]	[illegible]		X	X								X			X		X		X	
	168.[illegible]	209	X				X						X			X			X	X	
	169.[illegible]	210	X					X					X		X			X		X	
	169.3	211	X				X						X			X		X		X	
	170.[illegible]	[illegible]	X					X					X			X			X	X	
	171.[illegible]	213		X			X						X	X					X	X	
	171.6	14		X			X						X		X			X		X	
Bellows Falls	174.[illegible]	[illegible]	X				X					X			X			X		X	
	186.2	[illegible]		X			X				X					X		X			X
	186.2	403	X						X	X						X	X				X
Natural	187.[illegible]	404	X				X						X		X			X		X	
	190.[illegible]	[illegible]		X	X								X			X			X	X	
	191.3	[illegible]		X				X					X					X			X
	194.[illegible]	[illegible]		X		X							X	X		X		X		X	
	197.[illegible]	[illegible]		X			X						X			X		X			X
Wilder	217.[illegible]	[illegible]	X						X			X				X			X	X	
	217.55	[illegible]	X						X			X				X			X		
	217.33	42	X				X					X		X				X		X	
	218.[illegible]	[illegible]	X				X					X		X				X			X
	218.[illegible]	[illegible]		X			X					X				X		X			X
	218.[illegible]	[illegible]	X				X					X				X		X		X	
	22[illegible]	[illegible]	X						X			X				X		X		X	
	22[illegible]	[illegible]	X					X				X				X			X		X
	22[illegible]	[illegible]	X				X					X		X				X		X	
	22[illegible]	[illegible]		X			X					X		X					X		X
	22[illegible]	[illegible]	X			X						X		X					X		X
	228.[illegible]	[illegible]	X		X							X				X			X		X
	230.[illegible]	128	X						X			X				X			X	X	
	230.[illegible]	55	X						X			X				X			X	X	
	231.[illegible]	56	X						X			X				X			X	X	
	231.[illegible]	57	X						X			X		X					X	X	
	232.28	129	X						X			X				X			X	X	
	232.[illegible]	58		X			X					X				X		X			X
	232.[illegible]	130		X					X			X				X			X	X	
	233.[illegible]	59	X				X					X				X			X	X	
	233.99	60	X		X							X			X				X	X	
	234.99	131	X		X							X				X			X		X
	234.[illegible]	61	X		X							X			X				X	X	
	235.64	132		X			X				X					X		X		X	
	236.[illegible]	62		X			X				X			X				X		X	
	237.[illegible]	133	X				X				X			X			X			X	
	238.[illegible]	63	X		X						X			X					X	X	
	239.[illegible]	64	X		X						X			X					X	X	
	239.[illegible]	65	X		X						X					X			X	X	
	23[illegible]	66		X					X		X					X		X		X	
	240.[illegible]	134							X		X				X				X	X	
	240.[illegible]	135							X		X			X					X	X	
	241.[illegible]	136	X						X		X			X				X			X
	241.[illegible]	[illegible]	X				X				X				X			X			X
	242.[illegible]	[illegible]		X					X		X					X			X		X
	242.[illegible]	[illegible]	X						X		X				X				X	X	
	243.[illegible]	69	X				X				X					X			X	X	
	244.19	138	X				X				X					X		X		X	
	244.18	70	X						X		X					X			X	X	
	246.[illegible]	139	X			X					X					X		X		X	
	246.[illegible]	[illegible]	X				X				X					X		X		X	
	247.[illegible]	[illegible]	X				X				X					X		X		X	
	247.[illegible]	141		X			X				X			X				X			X
	248.[illegible]	[illegible]		X			X				X					X		X		X	
	250.[illegible]	74		X			X			X						X		X			X
	250.6	[illegible]	X						X	X					X				X		X
	252.7	[illegible]	X				X			X				X				X		X	
	252.[illegible]	[illegible]	X			X				X					X			X		X	
	25[illegible]	[illegible]		X				X		X				X					X	X	
	25[illegible]	[illegible]		X			X			X				X					X		X
	25[illegible]	[illegible]		X		X				X				X					X	X	
	257.[illegible]	81	X			X				X				X					X		X
	258.[illegible]	[illegible]		X			X			X					X			X		X	
	258.[illegible]	83	X					X		X					X			X			X

October 29, 1975

May 17, 1977

Figure 69. Photographs on Area 26.

progressive at this site as determined by observed erosion relative to the tree on the bank observed in both photos. Table 5 presents the physical characteristics for these six index sites. The relative incremental values of shear stress caused by forces other than shear stress for these six index sites can be determined by using Tables 2 and 5. The total effective tractive force acting on each bank was determined and values are tabulated in Table 6. The effect of river bends on the effective tractive force was also considered.

DISCUSSION

The evaluation of forces causing bank erosion established the relative importance of factors causing erosion and the relative magnitude of the bank erosion problems for different river conditions. Utilizing the observed data at 103 erosion sites, a classification of bank erosion in the Connecticut River was made. These two approaches utilized to quantify the bank erosion make the comparison between theoretical evaluation and field observation possible.

Table 2 was developed based on theoretical consideration of causative factors (Section 3) and available data presented in Section 2. Further examinations of Table 2 follow.

A summation of the relative magnitude of bank erosion for different factors causing erosion is determined from Table 2 and is given in Table 7. This table indicates the relative importance of these factors. In decreasing importance they are: shear stress (velocity), pool fluctuation, boat waves, gravitational forces, seepage forces, stage variation, wind waves, ice, flood variation, and freeze-thaw. In general, the stratified soil is slightly less susceptible to bank erosion than the noncohesive banks except when considering boat waves, seepage forces, and gravitational forces. Overall, the predominant force causing bank erosion is the shear stress (or velocity).

Further analysis of Table 2 provides a measure of the relative magnitude of bank erosion for different conditions (Table 8). This table demonstrates that a reach with a high bank is more susceptible to erosion, and vegetation is important in stabilizing the bank. Furthermore, the natural river has a higher potential for bank erosion than do the pools. The average sum of relative magnitude of bank erosion for a natural river is 113.75 and that for pools is 84.75. In other words, the natural river is roughly 1.34 (113.75/84.75 = 1.34) times more susceptible to major bank erosion than pools.

A further examination of Table 4 indicates that the number of erosion sites per mile for the natural river is 0.92 and that for pools is 0.68 (Table 9). Therefore, the measured data indicate the natural river is 1.35 (0.91/0.70 = 1.30) times more susceptible to bank erosion than pools. This is very close to the theoretical evaluation with a value of 1.34. Table 10 summarizes statistics of erosion sites according to the classification shown in Table 4 for the study reach. This table indicates that the predominant bank height of the observed erosion sites is low (less than 15 feet). The most common type of erosion is the "sloughing". In addition, most observed erosion sites are located in pools, straight reach, stratified soil, and vegetated area. The above statistics cannot be directly utilized to interpolate the causes of erosion; an unbiased statistical approach that considers erosion sites per mile for a particular classification.

Table 5. Physical characteristics of index sites.

Station Number	River Mile	Bank Height	Erosion Type	Erosion Site Location	Bank Location	Soil Type	Vegetation
255	130.04	High (20')	Mass Wasting	Middle Pool	Straight Reach	Stratified	Barren
301	151.4	High (20')	Mass Wasting	Middle Pool	Inner Bend	Stratified	Barren
26	186.2	High (20')	Sloughing	Middle Pool	Straight Reach	Non-cohesive	Barren
31	197.5	High (50')	Sloughing	Natural Reach	Straight Reach	Non-cohesive	Barren
51	223.29	Low (9')	Sloughing & Undercutting	Lower Pool	Outer Bend	Non-cohesive	Vegetated
147	254.9	High (15')	Sloughing	Upper Pool	*Outer Bend	Stratified	Barren

* Observed Erosion is more severe because of the bend effect.

Table 6. Total effective shear stress for index sites.

Factors and Variables	Equivalent Shear Stress Relative to Shear Stress on Natural River Banks					
	Station #255 (R.M. 130.04)	Station #301 (R.M. 151.5)	Station #26 (R.M. 186.2)	Station #31 (R.M. 197.5)	Station #51* (R.M. 223.29)	Station #147** (R.M. 254.9)
Shear Stress	.68 γds	.68 γds	.69 γds	1.0 γds	.52 γds	.68 γds
Flood Variation	.03 γdS	.03 γdS	.02 γdS	.02 γdS	.01 γdS	.03 γdS
Stage Variation	.06	.06	.06	.10	.02	.06
Pool Fluctuation	.17	.17	.19	.11	.11	.17
Wind Waves	.03	.03	.02	.01	.02	.03
Boat Waves	.14	.14	.10	.05	.07	.14
Freeze-thaw	.01	.01	.01	.01	.01	.01
Ice	.03	.03	.02	.04	.01	.03
Seepage Forces	.11	.11	.07	.07	.05	.11
Gravitational	.17	.17	.11	.11	.01	.17
Incremental Sum Excluding Shear Stress	0.75 γdS	0.75 γdS	0.60 γdS	0.52 γdS	0.31 γdS	0.75 γdS
Shear Stress Acting on Bank	0.52 γdS	0.52 γdS	0.53 γdS	0.77 γdS	0.40 γdS	0.52 γdS
Added Shear Stress Due to Bend	-	-	-	-	0.02 γdS	0.42 γdS
Total Effective Shear Stress	1.27 γdS	1.27 γdS	1.13 γdS	1.29 γdS	0.73 γdS	1.69 γdS

*Radius of curvature = 4000 ft and width = 400 ft
**Radius of curvature = 1000 ft and width = 300 ft

Table 7 . Sum of relative magnitude of bank erosion for different factors.

VARIABLES CAUSING EROSION	SUM OF RELATIVE MAGNITUDE OF BANK EROSION	
	NONCOHESIVE	STRATIFIED
Shear stress or velocity	359* (1.0)**	315 (0.88)
Flood variation	10 (0.03)	10 (0.03)
Stage variation	27 (0.08)	24 (0.07)
Pool fluctuation	63 (0.18)	54 (0.17)
Wind waves, surface erosion and piping	14 (0.04)	14 (0.04)
Boat waves, surface erosion and piping	34 (0.09)	42 (0.13)
Freeze-thaw	6 (0.02)	6 (0.02)
Ice	11 (0.03)	10 (0.03)
Seepage forces	28 (0.08)	38 (0.12)
Gravational forces	31 (0.09)	40 (0.13)

*Values are obtained from summing the relative magnitude of basic erosion for each variable as extracted from Table 2. For example, shear stress (or velocity) with noncohesive banks, 81 + 81 + 63 + 42 + 56 + 36 = 359.

**Standardized values based on the shear stress (or velocity) in noncohesive banks. For example, for flood variation, 10/359 = 0.03.

Table 8. Sum of relative magnitude of bank erosion for different conditions.

CONDITIONS	SUM OF RELATIVE MAGNITUDE OF BANK EROSION		
	NONCOHESIVE	STRATIFIED	AVERAGE
Natural river	112* (1.00)**	107 (0.96)	109.5 (0.98)
Natural river with high banks	124 (1.11)	112 (1.00)	118.0 (1.05)
Pools: low banks	103 (0.92)	97 (0.87)	100.0 (0.89)
Pools: low banks with vegetation	69 (0.62)	66 (0.59)	67.5 (0.60)
Pools: high banks	106 (0.95)	102 (0.91)	104.0 (0.95)
Pools: high banks with vegetation	69 (0.62)	69 (0.62)	69.0 (0.62)

*Values are obtained from summing the relative magnitude of basic erosion for each river condition as extracted from Table 2. For example, in the natural river with noncohesive banks, 81 + 2 + 6 + 6 + 2 + 4 + 1 + 2 + 4 + 4 = 112.

**Standardized values based on the natural river with noncohesive banks. For example, for pools with low and noncohesive banks, 100/112 = 0.89.

Table 9. Number of erosion sites per mile for different reaches.

	NATURAL RIVER	POOLS
No. of erosion sites	19.0	84
Total river mile	20.8	120.2
No. of erosion sites per mile	0.91	0.70

Table 10. Statistics of erosion sites according to the classification in Table 4.

Area	Stat's	Bank Height		Erosion Type					Erosion Site Location				Bank Location			Soil Type			Vegetation		Total
		Low <15'	High >15'	Mass Wast'g	Head Cut'g	Sloughing	Shoal'g Wash'g	Under-cut'g	Upper Pool	Mid Pool	Low Pool	Nat' Reach	Outer Bend	Inner Bend	Straight Reach	Coh's	Non Coh's	Strat'	Vegt'd	Barren	
Wilder Pool	#	39	15	7	5	22	3	17	10	21	23	0	18	9	27	1	23	30	37	17	54
	%	72	28	13	9	41	6	31	19	39	42	0	33	17	50	2	43	55	69	31	
Bellows Falls Pool	#	2	1	0	0	2	0	1	1	1	1	0	0	1	2	1	2	0	1	2	3
	%	67	33	0	0	67	0	33	33	33	34	0	0	33	67	33	67	0	33	67	
Vernon Pool	#	11	3	2	1	6	5	0	7	3	4	0	4	6	4	0	5	9	12	2	14
	%	79	21	14	7	43	36	0	50	21	29	0	25	42	29	0	36	64	86	14	
Turners Falls Pool	#	9	4	2	2	7	2	0	2	9	2	0	3	2	8	0	7	6	9	4	13
	%	69	31	15	15	55	15	0	15	69	16	0	23	15	62	0	54	46	69	31	
Natural Reaches	#	9	10	2	1	11	5	0	0	0	0	19	3	5	11	1	10	8	16	3	19
	%	47	53	11	5	58	26	0	0	0	0	100	16	26	58	5	53	42	84	16	

For example, the length of vegetated bank line is much longer than the unvegetated zone in these two areas. A direct interpolation of statistics in Table 10 without a proper adjustment on a linear mile basis will indicate that the total number of observed erosion sites with vegetated banks erroneously exceeds those with barren areas.

Based on the above discussions, the theoretical approach, as presented in Table 2, is justified considering physical significance and field observations. However, the causes of bank erosion are not only a function of forces but are also related to the erodibility of banks. Changes in water surface elevation due to impoundment can reduce some of the forces such as shear stress but at the same time can expose more erodible material to the flow and hence may increase bank erosion.

In the preceding subsections, the major causes of bank erosion were first identified and subsequently evaluated utilizing available data, current theory, personal experience and sound professional judgment. A statistical analysis of the sites studied over recent time by the Corps of Engineers along the Connecticut River was also made. The analysis verifies that the major force causing bank erosion, particularly during episodic events, i.e., major floods, is the tractive shear stress exerted on the banks of the channel by high-velocity flow. The magnitude of this shear velocity depends upon the geometry of the channel, the location within the channel, i.e., whether the flow is in a straight reach, or along the outside of a bend or in some other location. Other forces, of course, play an important role in the stability of the banks of the channel. Note that Table 7 indicates that forces caused by such factors as pool fluctuations can cause an increase in instability on the order of 18 percent of the shear stress exerted on the bank by the flowing water. Other causes of upper bank erosion, such as wind-generated waves, boat-generated waves, ice, etc., have a lesser impact on long-term bank stability, but nevertheless can cause significant erosion rates near the water surface-bank interface.

Having submitted the analysis of factors contributing to bank erosion along the Connecticut river, it is obvious that different people with different views assess the causes and the gross long-term effects differently. A general conflict between the opinion of riparian land owners and the contents of the report has arisen based upon their apparent concept that pool fluctuations and related forces are a cause of major erosion leading to a continuous loss of significant acreage of riparian land. As clearly specified earlier, these factors do cause erosion. The magnitude and extent of erosion depend upon many factors identified in this study. However, causes of major shifts in channel alignment must be attributed to high velocity flows of relatively short duration during periods of flooding. In order to more clearly focus on the major causes of bank erosion, it is perhaps worthwhile to subdivide these forces in relationship to where they act. Many geologists, engineers and laymen alike miss the main point when they consider major causes of bank erosion. One must consider that the forces acting on the bank can be broken into two categories: (1) those forces that act at and near the surface of the water associated with pool fluctuations, related piping, groundwater, wind waves, boat waves, ice, lack of or removal of vegetation, and so forth, and (2) those forces acting on the full height of the submerged bank. Actually, in this instance, the major force is velocity or tractive force. The distribution of velocity and shear stress on the banks of channels is an issue that has been studied in some detail over many years. One of the earliest analytical studies describing shear stress on the banks of streams is attributable to E. W. Lane and the details of this force distribution on the banks are clearly delineated in a report prepared by Lane in 1953. Since that time, many other individuals, Ippen and Drinker at MIT (1962), various groups dealing with the stability of canals and rivers in India, and recently Simons et al. (1979) at Colorado State University, have studied and verified Lane's findings. With regard to identifying the magnitude of the boundary shear stress acting on the banks of channels, it is noteworthy that both theory and physical experiments conducted in the laboratory and in the field verify that the maximum tractive shear stress acts upon the banks of the channel approximately two-thirds of the depth below the air-water interface. Hence, the forces causing erosion of banks can be subdivided into the two categories as previously mentioned, those acting near the surface of the flow and those acting with greatest intensity nearer the bottom of the submerged banks.

Assume a channel cross section as indicated in Figure 70 and further assume that the bank line is subjected to forces generated by pool fluctuations, boat waves, wind waves, surges, inflow of groundwater, and so forth. The action of these forces near the surface of the flow causes some erosion on the banks and may induce piping in lenses of non-cohesive material located in the upper part of the submerged bank. If these were the only forces to which the bank line was subjected, the bank would gradually adjust by developing a shelf or a platform area wide enough to dissipate the forces causing erosion, increasing upper bank stability as the adjustment occurred. The extent of this erosion landward would in most cases be limited to an average of 10 to 15 feet in a large river. Comparing Figure 70 with Figure 71, we see the ultimate type of erosion that would result on these upper banks when subjected to the forces that are most active in this upper zone. As indicated in Figure 71, after the bench or berm is formed growth of vegetation usually takes place, further increasing the stability and curtailing further significant upper bank erosion. One can conclude then that indeed bank erosion has occurred, but not the type of erosion that significantly shifts channel location within the floodplain. In general, the effect is limited to a relatively narrow zone as stated above, usually not extending more than 10 to 15 feet landward even in large systems.

The next phase of the erosion process to consider is the bank erosion caused by high velocity flows, or an exertion of a tractive shear stress on the banks by the flowing water. As pointed out earlier, this force acts with a maximum magnitude at a distance about two-thirds of the depth below the water surface. After considering Figures 70 and 71, now consider how the system will respond when subjected to a flood event. With the flood event, high velocity flows are produced which act on the bank as previously indicated. With this maximum force being submerged a considerable distance below the water surface, erosion of the total bank occurs and the major bank line moves landward. As the bank line moves landward, the berm formed by water surface fluctuations and related phenomena is overtaken, and in many instances the bank line may move so far landward that effects caused by past near-surface erosion phenomena are eroded. To illustrate, refer to Figure 72, which indicates the landward movement of the main bank line and furthermore indicates the total loss of bank material and the development of a new channel geometry. After the termination of the flood and with the new geometry indicated in Figure 72, the surface forces can go to work on the bank line again to form a new berm. After sufficient time, the bank line will take on an appearance as previously indicated in Figure 71. Consequently, one can

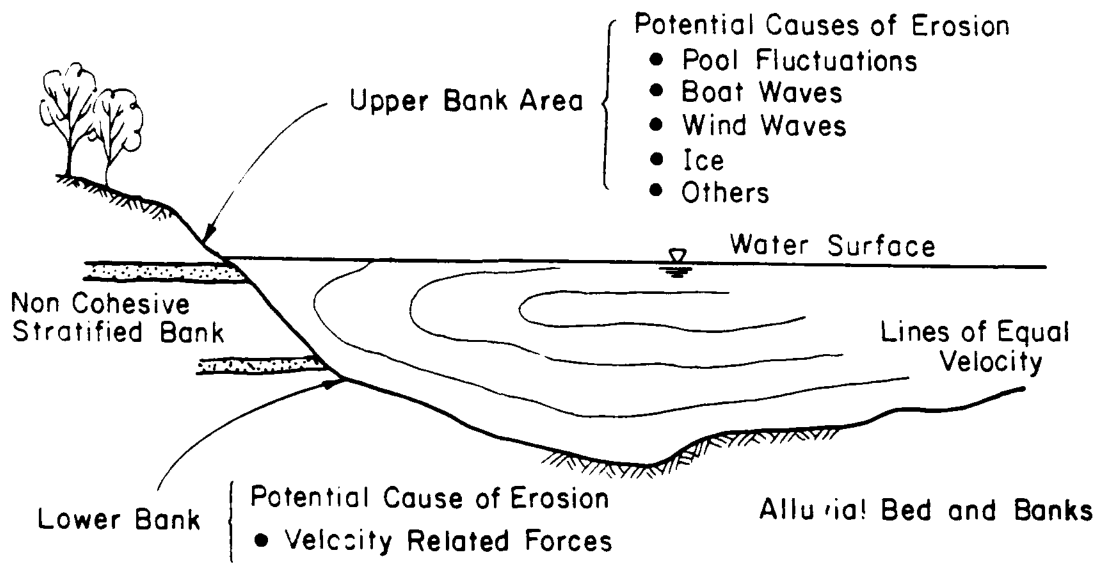


Figure 70. Assumed initial channel conditions.

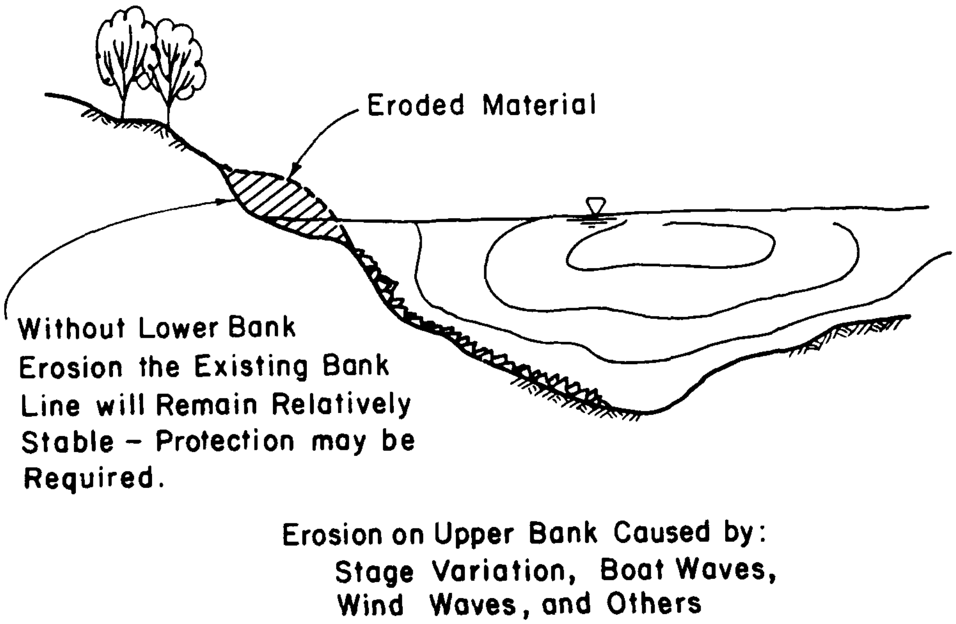


Figure 71. Potential bank line geometry generated by erosive force acting on the bank near the water surface.

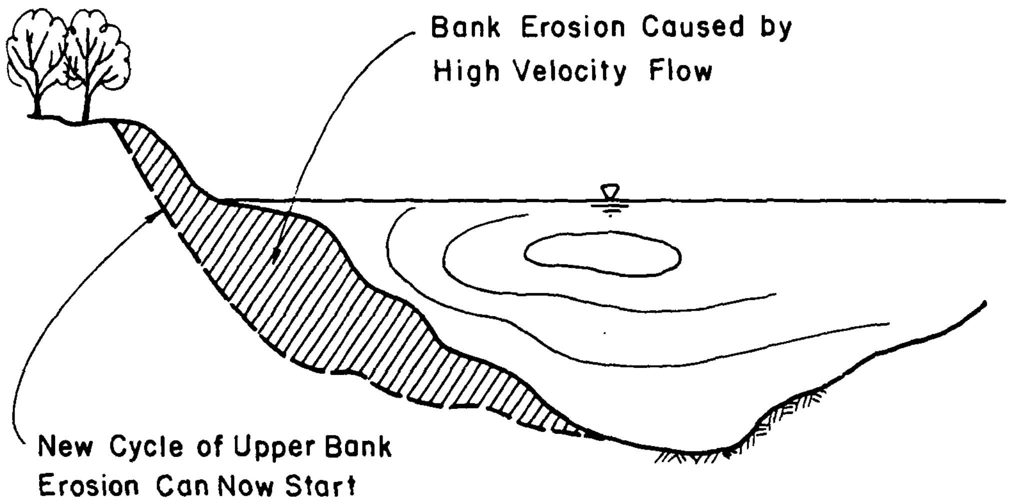


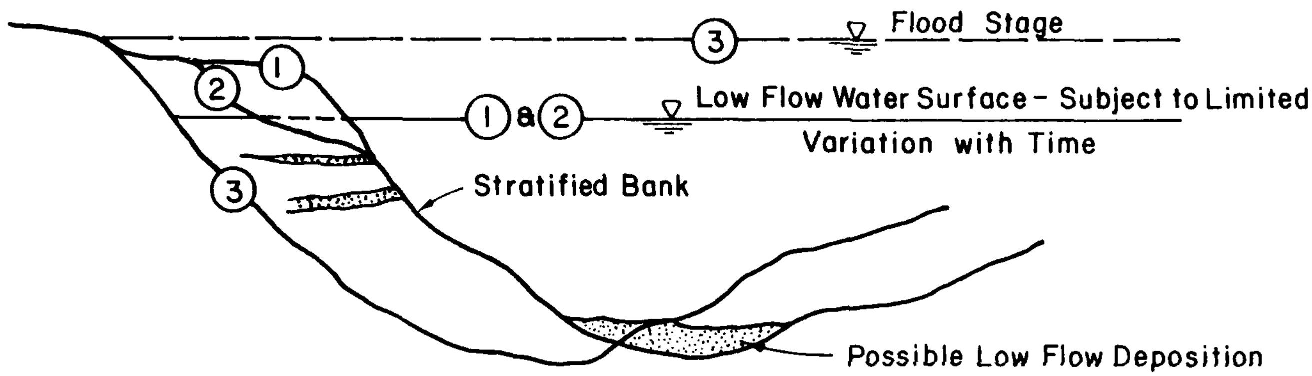
Figure 72. Bank erosion caused by flood stage high velocity flow.

conclude that upper bank protection provided to prevent erosional effects of pool fluctuations and wave-related erosion will not provide the protection required against lower bank erosion. It must be emphasized that upper bank protection will usually fail during major flood events. The bank beneath this upper bank protection is simply removed, causing the upper bank material to slough or slide into the main channel system. Realizing the way in which these two sets of forces act makes it obvious to the geologist, the engineer and the layman that total bank protection is necessary to provide total protection from the forces that act upon the bank lines if in fact the forces are sufficient to cause bank erosion. The limited exent of the development of beaches near the water surface is a further indication that this type of evidence of upper bank erosion is often erased by the erosion of the total bank by velocity related forces.

As an alternative to total bank protection, one could consider placing riprap on the lower bank as shown in Figure 71. In this instance, the riprap thus placed would prevent major lateral shifts caused by high-velocity flows. Without the shifting associated with high-velocity flows, a berm and setback vertical bank would form. After this phase of erosion is completed on the upper banks there would be little additional erosion to plague the riparian property owner.

In considering the progressive erosion of banks to ultimate stability, consider Figure 73. Line 1 illustrates the starting condition for the arguments presented. Line 2 shows the conditions of the channel bank after prolonged periods of relatively small flows during which very little erosion has occurred as a result of velocities, but these erosive forces acting near the water surface have attacked the upper bank, forming a berm. Finally, refer to Line 3, which illustrates the channel geometry that could develop while the system was subjected to a major flood. During periods of high flow and high velocity, bank erosion would occur and a new geometry would develop. After the flood the upper bank erosion cycle would start again. Referring to this figure and summarizing, it is important to stress then that in order to have bank stability, it is essential to protect the banks against erosion from the flowing water. If that type of bank erosion is not occurring, that is, if the banks are sufficiently stable to resist the erosive forces of the flowing water, then the upper bank will be the only part of the channel system subjected to erosive forces. The action of these forces near the air-water interface on the upper bank would cause a landward movement of the upper bank only, the forming of a berm, perhaps some settling down of blocks of banks and minimal mass wasting, but after the initial adjustment had occurred in the system, further erosion would be minor. Also as stated in the preceding paragraphs, it is necessary to emphasize that protection of upper bank areas by utilizing either non-structural or structural methods to prevent erosion caused by pool fluctuations, wind and boat waves, etc. will only be worth the investment when these are the only forces causing erosion in the system. Otherwise, if one protects the upper bank only, he or she stands an excellent chance of losing that protection during periods of major flow when the total bank line is subjected to the erosive forces of the flowing water causing the total vertical bank to move further landward depending upon conditions that exist at each particular site.

The impacts of hydropower development on bank stability in Turners Falls Pool have been and continue to be more severe than for the other pools. The increase in pool level, the larger pool fluctuations and flow reversals caused



1. Starting Conditions
2. After Prolonged Period of Low and Moderate Flows Local Erosion has Attack the Upper Bank Forming a Berm
3. Channel has Shifted Laterally Due to High Velocities Eroding the Total Bank, Cycle of Low Flow Upper Bank Erosion Can Repeat Again, etc.

Figure 73. Progressive bank erosion at erodible sites.

by the present hydropower operation all contribute to the documented bank instabilities in this part of the study reach.

In analyzing the causes of bank erosion in Turners Falls Pool it is suggested that the erosion analysis presented in Table 2 and subsequent tables should be utilized. From this analysis coupled with consideration of adverse hydraulic conditions related to power generation it is concluded that:

1. The maximum tractive forces that can be exerted on the banks of the river will occur during periods of moderate and major floods. Hence, power generation has not altered this condition.

2. The flow reversals, turbulence and changes in river stage caused by present power generation methods have increased the tractive force sufficiently to induce bank erosion in those locations where the bank alignment and bank material causes the rate to be vulnerable to these forces.

3. The increase in pool fluctuations on bank stability in Turners Falls Pool is a very significant factor. Pool fluctuations on the order of 5 feet are at least twice as destructive to banks or pool fluctuations of about 1-3 feet as experienced in the other hydropower pools.

To stabilize the eroding banks in Turners Falls Pool will require special attention. As verified by installation of rock riprap near the intake-outlet works, such measures can provide upper bank protection against erosive forces related to hydropower pool operation. However, such protection does not prevent lower bank erosion to susceptible banks and in these locations without accompanying lower bank protection the treatments implemented to prevent upper bank erosion can be undercut and fail. As another alternative, hydro-seeding has been tried as a non-structural treatment. However, analysis of results to date show that hydro-seeding as utilized to date has not prevented upper bank erosion. The major reasons for the limited effectiveness of hydro-seeding are:

1. The banks were not preshaped to a stable slope prior to hydro-seeding.

2. The large pool fluctuations significantly reduce effectiveness of hydro-seeding as a means of bank erosion control.

In summary, if upper bank erosion is to be controlled it will be necessary to implement some measure of upper bank protection capable of withstanding the forces to which it will be subjected; also the means to provide lower bank protection to prevent failure of upper bank protection must be considered, and the cost of such bank stabilization treatments is large. Conversely, if upper bank protection is not provided where such erosion is in progress, erosion will continue until a stable terrace or bench is formed. It is estimated that upper bank erosion will slow down and in many cases stabilize within a 5-10 year period unless conditions for further upper bank erosion are set up by lower bank erosion. Furthermore, in the Turners Falls Pool upper bank erosion may extend landward on the order of 20-25 feet at vulnerable sites before some semblance of upper bank stability is achieved.

SECTION 5

CHANNEL STABILIZATION

GENERAL

Channel stabilization has proved to be a reliable method for preventing bank erosion. On some streams, bank protection is placed to arrest the lateral movement of the stream when such movement threatens to destroy a man-made structure such as a levee, railroad, highway, or in some instances, a residence, town or city. The Connecticut River is not a navigable channel. Where navigation is not involved, isolated protective works can be constructed (usually in a bend and sometimes in a reach of the river) to furnish protection for the structures in that immediate area. In general, location, height, and strength of protection works vary. However, revetments can serve both purposes. Secondary objectives of channel stabilization may involve beautification, recreation, improved fish and wildlife habitat, water quality retention or improvement, and other beneficial effects important to the project. This section describes some of the stabilization measures that can be utilized in the Connecticut River.

TYPES OF STABILIZATION

Two general types of treatment are used for channel stabilization. These include continuous protection, such as revetments; and intermittent protection, such as dikes, and groins. The former is placed in direct contact with the bank where erosion has or is expected to occur. In some instances, intermittent structures are used for bank protection. In those cases, the bank is protected indirectly by deflecting the currents away from the critical area. Usually, this type of treatment is used to contract the channel to a specified width. Such treatment includes chute closures that confine the river to a single channel and train the stream to a desirable alignment. Potential types of bank stabilization for the Connecticut River follow.

ROCK RIPRAP

Bank protection by rock riprap is probably the most widely used method. Whenever available and economically justified, rock riprap protection is one of the surest methods. Advantages of using riprap are:

1. It is flexible and not impaired or weakened by slight movement of the embankment resulting from settlement or other minor adjustments,

2. Local damage or loss is easily repaired by the addition of rock where required,

3. Construction is not complicated and no special equipment or construction practices are necessary,

4. Appearance is more natural, hence acceptable in recreational areas,

5. If exposed to fresh water, vegetation will often grow through the rocks adding structural strength to the embankment material and restoring natural roughness,

6. Additional thickness can be provided at the toe to offset possible scour when it is not feasible to place it at adequate depth,

7. Wave runup is reduced (as much as 70 percent) as compared with smooth types, and

8. It is salvable, may be stockpiled, and reused if necessary.

However, if rock riprap is not placed properly, segregation of the particles reduces the interlocking effect between particles and reduces the stability of the riprap. In addition, riprap particles are susceptible to detachment by ice flows. Consideration must be given to this point when designing riprap in climates where rivers freeze. Also, when riprap is used in close proximity to residential areas, some rock particles may be removed and utilized for other purposes.

There are several factors that must be considered when designing riprap protection including: size, gradation, weight, shape, strength, thickness of the blanket, filter requirements, side slopes of the revetment, and method of placement.

Stevens, Simons, and Lewis (1976) and Simons and Sentürk (1977) present a review of the most widely used methods for determining the size of riprap rock. Stevens et al. (1976) proposed a method based on a safety factor defined as the ratio of the moments of forces resisting rotation of the rock particle out of the riprap blanket to the moments tending to dislodge the particle out of the riprap layer into the flow. The safety factor method permits use of four possible design options for a fixed set of flow conditions on a side slope.

1. For a given rock size and side slope, the safety factor can be computed and the design accepted or rejected on the basis of the value of the safety factor.

2. For a given rock size, the side slope can be chosen to provide a preselected safety factor.

3. For a given side slope, the rock size that gives a preselected safety factor can be computed.

4. For a given safety factor, the proper combinations of rock size and side slope can be computed.

All available design formulas for determining the size of rock riprap consider the shear stress or tractive force at the boundaries as known quantities. (Refer to EM 1110-2-1601, Design of Flood Control Channels.) However, experiments by Blinco and Simons (1974) showed that the shear stress or tractive forces vary randomly. Based upon this concept, Li, Simons, Blinco, and Samad (1976) developed a probabilistic model to predict the failure probabilities of riprap.

The size of riprap can be determined by estimating the velocity of flow along, across, and around the end of the structure. A recommended approach uses the method developed by Simons and Sentürk (1977). A summary of this work is presented.

Considering flow along an embankment (Figure 74), the forces on the rock particle are lift force F_ℓ, drag force F_d, and weight of the particle W_s. Rock particles on side slopes will tend to roll rather than slide, so it is appropriate to consider stability of rock particles in terms of moments at a contact point 0 about which rotation must occur. The components of forces relative to the plane of motion are shown in Figure 74.

At incipient motion there will be a balance of moments so that

$$S.F. = \frac{e_2 W_s \cos\theta}{e_1 W_s \sin\theta \cos\beta + e_3 F_d \cos\delta + e_4 F_\ell}$$

The factor of safety S.F. of particles against rotation is then determined by the ratio of the moments as

$$S.F. = \frac{e_2 W_s \cos\theta}{e_1 W_s \sin\theta \cos\beta + e_3 F_d \cos\delta + e_4 F_\ell} \tag{5-1}$$

or

$$S.F. = \frac{\cos\theta \tan\phi}{\eta' \tan\phi + \sin\theta \cos\beta} \tag{5-2}$$

where

$$\eta' = M + N \cos\delta$$

$$M = e_4 F_\ell / e_2 W_s$$

$$N = e_3 F_d / e_2 W_s$$

and η' is called a stability number and ϕ is the angle of repose of the material. If δ is equal to 0 (no angle between the resultant force and the drag vector), η can be defined as

$$\eta = M + N \tag{5-3}$$

where η is also a stability number that can also be written in terms of hydraulic variables as

$$\eta = \frac{21\,\tau_s}{(S_s - 1)\,\gamma\, D_{50}} \tag{5-4}$$

in which θ_s is the shear stress on the particles with size D_{50}. The unit weight of water is γ, S_s is the specific gravity of the riprap, and η' and η are related by

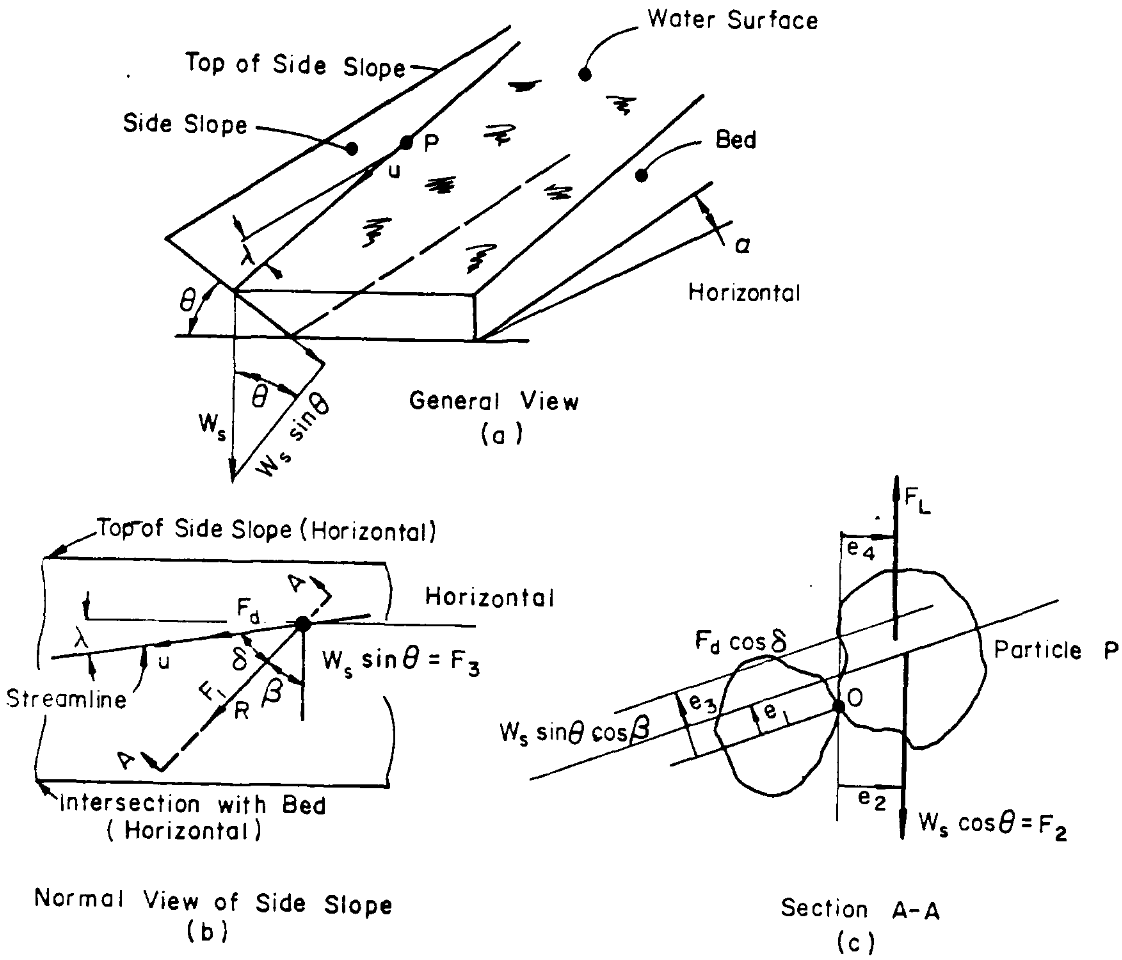


Figure 74. Diagrams for riprap stability analysis (after Campbell, 1966).

$$\frac{\eta'}{\eta} = \frac{\frac{M}{N} + \cos\delta}{\frac{M}{N} + 1} \tag{5-5}$$

It is reasonable to assume, in considering incipient motion of riprap particles, that

$$\frac{M}{N} = \frac{e_4 F_\ell}{e_3 F_d} \tilde{=} 1 \tag{5-6}$$

therefore,

$$\frac{\eta'}{\eta} = \frac{1 + \cos\delta}{2} = \frac{1 + \sin(\lambda + \beta)}{2} \tag{5-7}$$

It can be shown that

$$\tan\beta = \frac{\cos\lambda}{\frac{2\sin\theta}{\eta\tan\phi} + \sin\lambda} \tag{5-8}$$

where λ is the angle between the horizontal and the velocity vector in the plane of the side slope. By knowing or assuming values of θ, ϕ, λ, and S.F., Equations 5-1, 5-7, and 5-8 can be solved simultaneously for β, η, and η'. Once the value of η is known, Equation 5-4 can be solved for D_{50}. In many circumstances the flow angularity with the horizontal is small, (i.e. $\lambda \tilde{=} 0$), and Equation 5-8 reduces to

$$\tan\beta = \frac{\eta\tan\phi}{2\sin\theta} \tag{5-9}$$

and Equation 5-1 solved for η' becomes

$$\eta' = \left(\frac{S_m^2 - (S.F.)}{(S.F.)S_m^2}\right)\cos\theta \tag{5-10}$$

in which $S_m = \frac{\tan\phi}{\tan\theta}$ is the safety factor of rock particles from rolling down the slope with no flow. Equation 5-7 becomes

$$\frac{\eta'}{\eta} = \frac{1 + \sin\beta}{2} \tag{5-11}$$

Once the size of riprap is determined, which should be interpreted as the median size D_{50}, the next requirement that should be considered is its gradation. Simons and Sentürk (1977) suggest that riprap gradation should follow a smooth size distribution curve such as that shown in Figure 75. The ratio of maximum size to median size D_{50} should be about 2.0, and the ratio between median size and the 20 percent size should also be about 2.0. This means the largest stones would be about 6.5 times the weight of the median size and small sizes would range down to gravels.

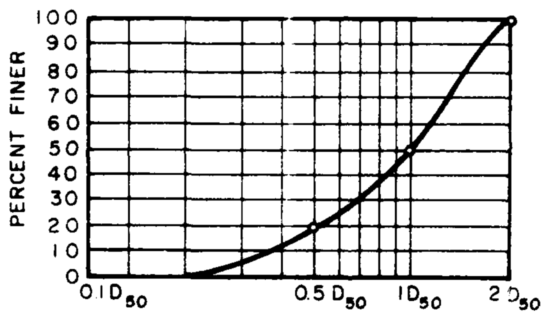


Figure 75. Suggested gradation for riprap (after Simons and Sentürk, 1977).

With a distributed size range, the interstices formed by the larger stones are filled with the smaller sizes in an interlocking fashion, preventing formation of open pockets. Riprap consisting of angular stones is more suitable than rounded stones. Control of the gradation of the riprap is almost always made by visual inspection.

Riprap should be hard, dense, and durable to withstand long exposure to weathering. Visual inspection is most often adequate to judge quality, but laboratory tests may be made to aid the judgment of the field inspector.

Riprap placement is usually executed by dumping directly from trucks during construction of the embankment. Rocks should never be placed by dropping them down the slope in a chute or pushing them downhill with a bulldozer. These methods result in segregation of sizes. With dumped riprap there is a minimum of expensive hand work. Poorly graded riprap with slablike rocks requires more work to form a compact protective blanket without large holes or pockets. Draglines with orange peel buckets, backhoes, and other power equipment can also be used advantageously to place the riprap.

On the Missouri River, the toe-trench revetment for riprap placement was found to be very successful. There it consisted of a stone fill of not less than 5 tons or more than 9 tons per linear foot placed to a depth of 7 feet below the normal low water plane in a trench excavated prior to erosion of the bank, or partly in a trench and partly in the area already eroded. Pavement for the upper bank is 15 inches thick at the point where it abuts the stone fill and 10 inches thick at the landward edge. Quarry run stone is also used in this work, and the limiting size of the stone is 250 pounds. Fines in the quarry run stone obviate the necessity of a gravel or crushed stone blanket underneath the pavement.

Rock riprap can also be windrowed along a desired alignment allowing it to fall into the river as the banks erode. This method was used to stabilize the Lower Colorado River and was described by McEwan (1961). It is recommended for rivers with banks composed of noncohesive alluvial materials, such as those of the Lower Colorado River.

Hand-placed rock riprap is another method of distribution. Stones are laid in more or less definite patterns, usually resulting in a relatively smooth top surface. This form of placement is used rarely in modern practice because it is more expensive than placement with power machinery.

The thickness of riprap should be sufficient to accomodate the largest stones in the riprap. For a well-graded riprap with no voids, this thickness would be adequate. If strong wave action is of concern, the thickness should be increased 50 percent.

Filters should be placed under the stone unless the material forming the core of the structure is coarse gravel or a mixture that forms a natural filter. Two types of filters are commonly used: gravel filters and plastic filter cloths.

With gravel filters, a layer or blanket of well-graded gravel should be placed over the embankment prior to riprap placement. Sizes of gravel in the filter blanket should be from 3/16 inch to an upper limit depending on the gradation of the riprap, with maximum sizes of about 3 to 3-1/2 inches.

Thickness of the filter may vary depending upon the riprap thickness, but should not be less than 6 to 9 inches. Filters that are one-half the thickness of the riprap are quite satisfactory. Suggested specifications for gradation are as follows:

$$\frac{D_{50}\ \text{(Filter)}}{D_{50}\ \text{(Base)}} < 40$$

$$5 < \frac{D_{15}\ \text{(Filter)}}{D_{15}\ \text{(Base)}} < 40$$

$$\frac{D_{15}\ \text{(Filter)}}{D_{85}\ \text{(Base)}} < 5$$

Plastic filter cloths are used with considerable success beneath riprap and other revetment materials such as articulated concrete blocks. The cloths are generally 100 feet long and 12 to 18 feet wide. The edges of the plastic sheets are hand sewn in the field with nylon twine. Overlap of 8 to 12 inches is provided with pins at 2 to 3 foot intervals along the seam to prevent separation in case of settlement of the base material. Some care must be exercised in placing riprap over the plastic cloth filters to prevent damage. Experiments and results with various cloth filters were reported by Calhoun, Compton, and Strohm (1971) in which specific manufacturers and brand names are listed. Stones weighing as much as 3,000 pounds have been placed on plastic filter cloths with no apparent damage. Filters can be placed subaqueously by using steel rods as weights fastened along the edges. Additional intermediate weights assist in sinking the cloth in place. Durability of filter cloths has not yet been established because they have only been used since about 1967. However, inspections at various installations seem to indicate little or no deterioration had occurred in the few years that have elapsed since the test installations were established.

TRANSVERSE DIKES

Transverse dikes are structures constructed transverse to the river flow and extend riverward from the bank at a certain angle. Along straight reaches dikes should be perpendicular to the bank. However, along sharp curves the dikes are slightly angled downstream in order to deflect the flow toward the center of the channel. Transverse dikes are an indirect method of bank protection and can be permeable or impermeable. Permeable dikes are those that permit the flow through but at reduced velocities. On the other hand, impermeable dikes allow either very little or no flow at all. Timber pile dikes are an example of the former type and stone dikes are an example of the latter type.

In planning dikes and dike fields, the parameters that must be evaluated and incorporated into the design are the shape of the dike, its root length, proper length, the angle it makes with the bank line, height, spacing between dikes, scour, and its protection in the vicinity of the dike and type of construction material.

Timber piles are the basic components of most permeable dikes. Single pile or clumps (three piles tied together at the top) constitute the basic unit in such types of dikes. Furthermore, one row or multiple rows of the basic unit, braced in two directions and used in a field, have been used. The rows are mostly straight; however, there have been situations on the Missouri River when L-shaped rows have been used to train the river into a certain alignment in order to protect the banks. The main effect of such dikes is to slow down the current velocity, allowing the suspended sediment to settle down and deposit and thereby protect the banks from further erosion. Increased lowering of the current velocity has been achieved by using screens fitted between the piles.

The arrangement of the basic components depends upon the velocity of flow, quantity of suspended sediment transport, and depth and width of the river. If the velocity of flow is large, timber pile dikes are not likely to be very effective. Stabilization of the bank by other methods should be considered. On the other hand, in moderate flow velocities with high concentrations of suspended sediments, these dikes can be quite effective. Deposition of suspended sediments in the pile dike field is a necessary consequence of reduced velocities. If there is not sufficient concentration of suspended sediment in the flow, or the velocities in the dike fields are too large for deposition, the permeable timber pile dikes will be only partially effective in protecting the banks.

Timber pile dikes are vulnerable to failure through scour. The piles can be driven to a large depth to achieve safety from scour or the base of piles can be protected from scour with dumped rock in sufficient amounts to form a combination of a permeable and impermeable dike. The various forms of timber pile dikes are illustrated in Figure 76.

The analysis performed by Water and Environment Consultants, Inc. (WEC) (1975) can be used as an estimate for the length L_s of dike requiring shank protection. The analysis is based on analytical considerations as well as field studies. They found that the upstream and downstream length ratios x:s and x':s', respectively, (Figure 77) can be qualitatively related to the Froude number of the main flow. Their results are given in Table 11.

Table 11. Relationship between Froude number and length ratio.

FROUDE NUMBER	x:s UPSTREAM LENGTH RATIO	DOWNSTREAM DEFLECTION ANGLE α	x':s' DOWNSTREAM LENGTH RATIO	DOWNSTREAM DEFLECTION ANGLE ψ
1.0	11:1	5.19°	1:1	45.0°
0.8	10:1	5.71°	2:1	63.43°
0.6	9:1	6.34°	3:1	71.56°
0.4	8:1	7.12°	4:1	75.96°
0.2	7:1	8.13°	5:1	78.69°

Note: x, s, α, x', s', and ψ are defined in Figure 77. The ratios of x:s and x':s' were estimated from limited flow separation data.

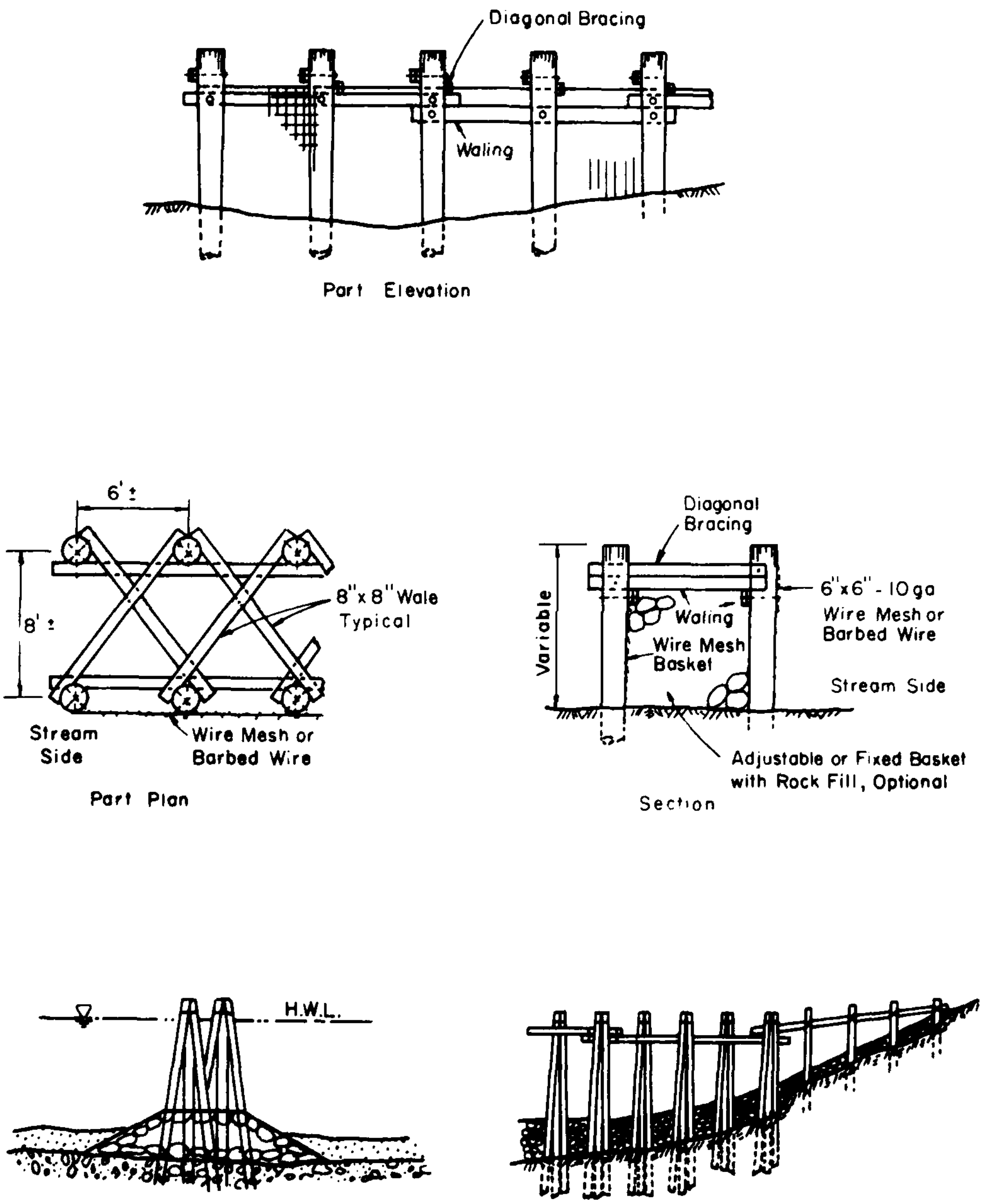


Figure 76. Timber pile dikes.

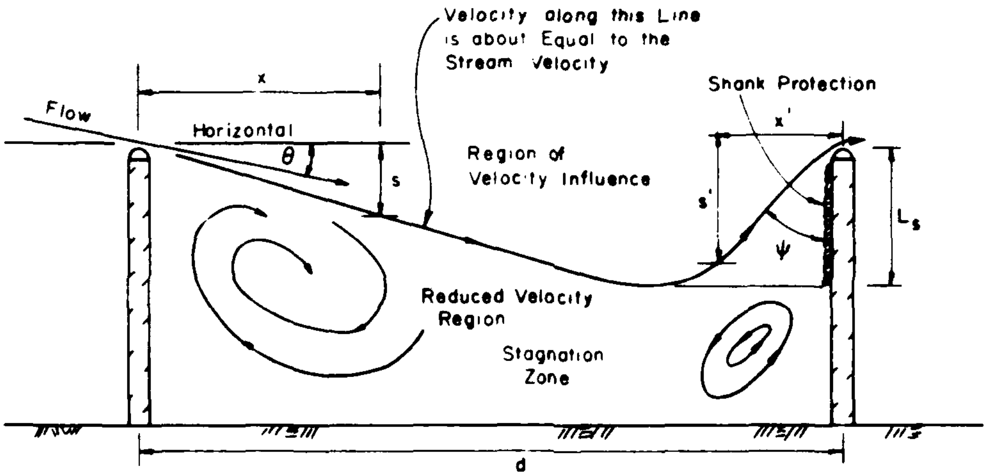


Figure 77. Sketch of flow between spur dike (after Water and Environmental Consultants, Inc., 1975).

By assuming equal dike length and form geometry, L_s can be determined from the following equation

$$L_s = \frac{L}{\cos(\alpha + \theta) + \tan \psi} \qquad (5\text{-}12)$$

For design purposes Water and Environment Consultants, Inc. (1975) suggested using a conservative estimate to approximate the velocity along the shank V_s as $V_s = V \cos \Psi$, where V is the magnitude of the velocity leaving the dike field at the angle Ψ. Using the velocity it is possible to determine whether or not protection is needed. Transverse stone dikes on the Middle Mississippi River (from the mouth of the Missouri River to the mouth of the Ohio River) have been used with excellent results. They have been constructed from quarry run stone with a limiting size of 5,000 pounds. The crest of the dike is usually placed at an elevation corresponding to 20 feet on the St. Louis Gage, which is approximately 24 feet above the Low Water Reference Plane. The dikes have a crown width of 5 feet and side slopes at the angle of repose of the stone that approximate 1 on 1-1/4. The dikes are connected to the bank with a stone root keyed approximately 15 feet into the bank. The bank for 100 feet below the structure azimuth line is graded to a 1 on 3 slope and paved with a 12-inch thickness of stone paving. Quarry run stone is also used for paving.

FENCES

Fences are constructed of two components, vertical posts driven past the anticipated scour depth and fencing material made from wood, steel wires, or steel wire-mesh that ties these posts horizontally. Additional supports constructed from steel pipes, angles, rails, beams, or wood or concrete are sometimes used to strengthen the posts. Fences are placed either parallel or transverse to the streambank and are built in single, double or multiple rows. They can also be made permeable, partly permeable, or impermeable to water and sediment movements. When constructed in a single line they are normally of the permeable type and are usually placed parallel to the bank (Figures 78 and 79).

Double line fences filled either partly or completely with brush, rock, gravel, or broken concrete are more stable than single line fences, and therefore can be subjected to more severe flow conditions. Figures 80 and 81 are examples of this type. O'Brien (1951) reported the successful use of this type along the concave banks of bends. When multiple lines of fences are used, they are placed transverse to the channel bank and spaced 3 to 10 feet apart (Figure 82).

If extra resistance to flow and added protection against scour are required, rock, hay, or brush is placed between the fences. Because construction works are relatively simple and require no special technology and because of the wide range of possible construction material, fencing can be an effective bank protection method under favorable flow conditions.

The basic principle in this bank protection method for the case of permeable and partly permeable fences is to slow the current below the eroding velocity, encourage sediment deposition, and allow sufficient time for the

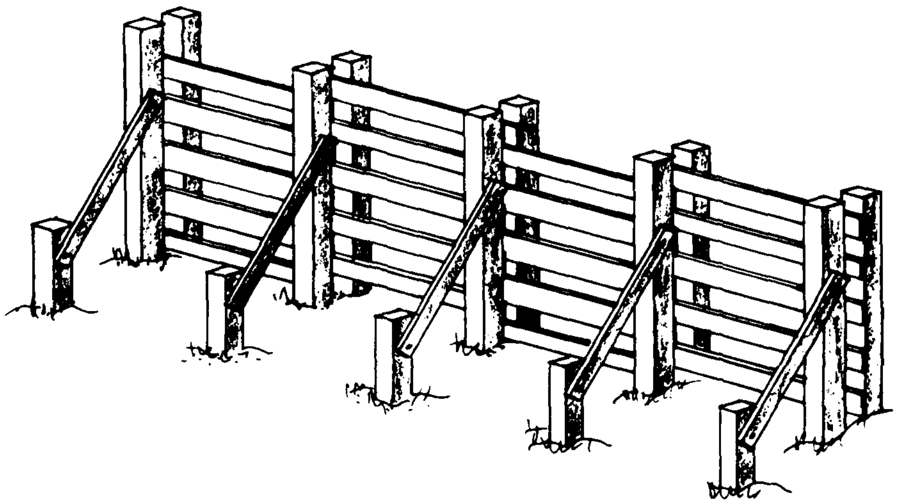


Figure 78. Wooden fence constructed parallel to bank.

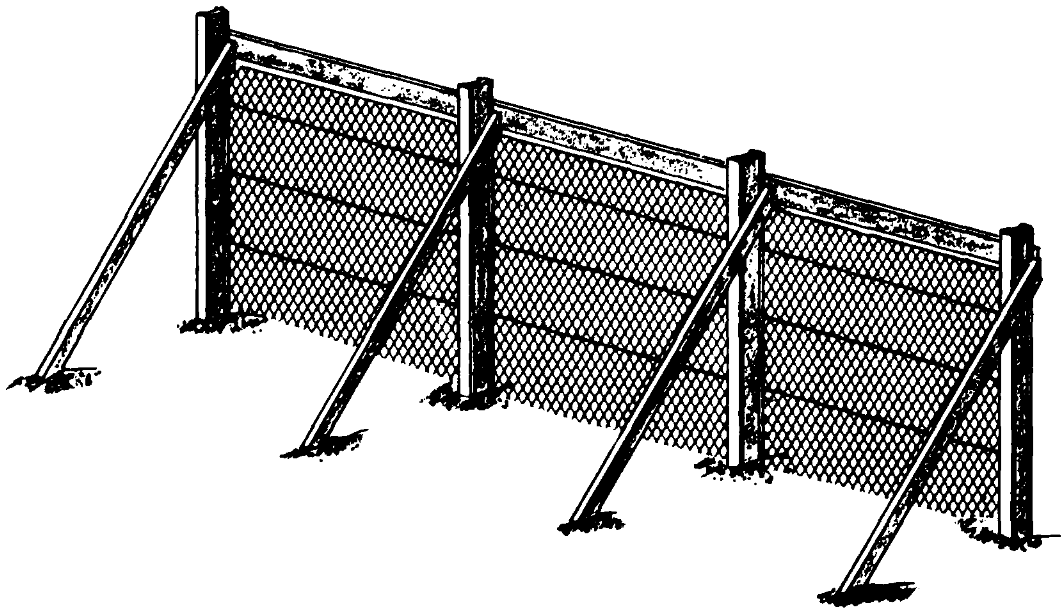


Figure 79. Single line of steel rail,
wire-meshed fence as a retard.

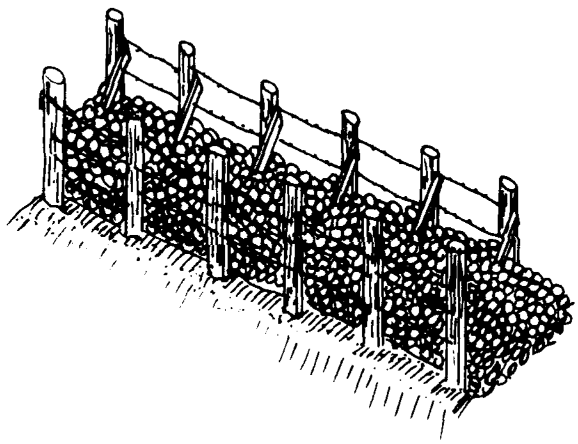


Figure 80. Double row fence of timber posts and barbed wire with rock fill.

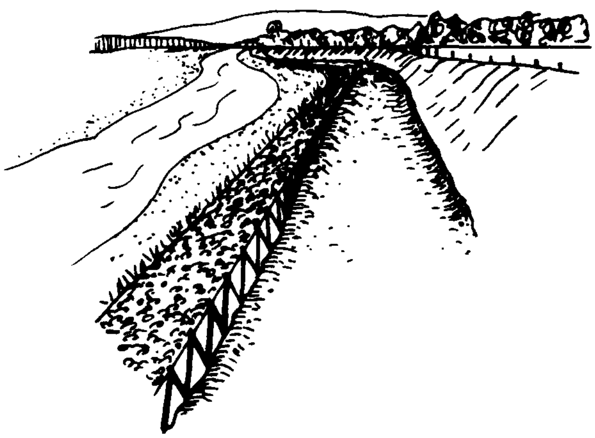


Figure 81. Double line of steel pipe fence filled with broken concrete.

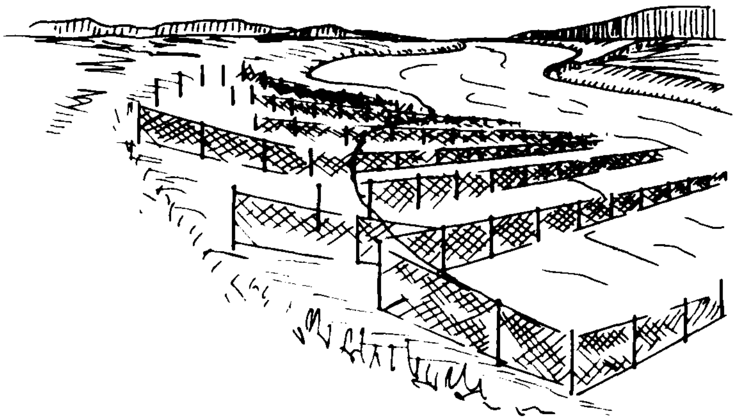


Figure 82. Wire fences constructed transverse to stream flow.

establishment of vegetation. However, impermeable fences provide protection by acting as a buffer zone between the bank and the erosive water currents.

One major problem with permeable fences in Alkali Creek, Wyoming was random gravel bars formed during high flows caused the diversion of intermediate and low flows to attack the bank. Fences are recommended for low gradient streams having discharges less than 500,000 cfs (Keown, Oswalt, Perry, and Bardeau, 1977).

VEGETATION

Vegetation of the banks has been observed to be an effective method of protecting the topsoil and reducing erosion. This is achieved in at least three ways. First, the roots of the vegetation provide structural reinforcement and stability to the soil, thereby increasing its resistance to erosion. When exposed, the roots can, in many instances, stop further erosion. Second and most important, the leaves, branches, and stems of the vegetation cause the reduction of the current velocity below the eroding values, and consequently, deposition of sediment may follow. Keown et al. (1977) reported that a well-established stand of selected grass can reduce the stream velocity as much as 90 percent at the boundary layer between the water and the soil. Third, the vegetation can act as a buffer zone between the stream bank soil and any floating material such as ice, logs, and debris that can cause soil failure by impact.

In general, vegetation used for stream bank protection can be classified into two broad categories: grasses and woody plants. Woody plants are slower than grasses in developing protective covers. However, they have a longer protection time period and can resist higher flow velocities. Furthermore, a combination of woody plants and grasses can be used. Woody plants are used for the protection of the toe of the stream bank slope and the grasses for the upper bank protection.

The effectiveness of vegetation as a stream bank protection method is directly related to the length and width of the stems, density of the leaves, areal density, depth and spread of the roots, stage of growth, healing ability and recovery of growth after floods.

Parsons (1963) showed that stage of growth is a very significant factor in the effectiveness of grasses. They are most effective when green, then become less effective as they approach the dormant period.

In selecting vegetation for a particular area, consideration should be given to the climatological conditions, quantity, intensity and distribution of precipitation, soil and vegetation characteristics, and periods of submergence.

Besides using vegetation as a direct means of bank stabilization, it can also be used as a supplementary method. Miller and Borland (1963) reported the planting of willows and Russian olive seedlings in the jetty fields to protect the fills after deterioration of the structures.

Before planting the grasses, the topsoil should be removed in order to prevent the growth of weeds that have a tendency to choke the vegetation. Planting can be achieved by sodding, springing, and mechanical broadcasting of mulches consisting of seed, fertilizer, and other organic mixtures (Figure 83).

The use of vegetation as stream bank protection is probably the cheapest method. Furthermore, it can blend nicely in the river environment and has a pleasing look. It is recommended for use on all small gradient streams and also as a supplementary method to be used in conjunction with permeable type protection methods. More discussion on the use of vegetation is given in Section 6.

GABIONS

Gabion cages or mesh cases are boxlike rectangular baskets made of galvanized wire-mesh material. They are either prefabricated or shaped in the field. To protect them against bulging, steel wires tying the opposite inside walls and/or diaphragms are used. The cages are normally filled with coarse gravel or high-density weather-resistant rocks. However, prior to filling, they should be placed in a supporting apron made of some material that must extend at least 6.56 feet beyond the toe of the gabion edge. The suggested minimum height of the apron is 1.5-2 times the depth of the probable scour at the toe of the bank. According to Mamak (1964), the width of the cages can range from 6.56 to 13.12 feet, the height up to 4.92 feet, and the length not more than 19.68 feet. After preparation of the apron, the cages are set one beside the other, securely wired to the apron wire-mesh and to each other and then filled.

Figure 84 illustrates suggested arrangements and dimensions of gabions used for the stabilization of the banks of shallow and deep rivers as well as for inclined and vertical banks. When back seepage and loss of fine material are of concern, graded filters or filter cloths are used behind the gabions. Otherwise, easy drainage through the gabions prevents the build-up of excessive hydrostatic pressure. Stabilization of river banks by gabions offers an effective, flexible structure that blends nicely with the river environment (Figure 85). It is recommended for use on rivers having small sediment loads since large sediment loads can cause the abrasion of the galvanized wire-mesh and its subsequent corrosion and failure.

GROUTED ROCK

When rocks of sufficient size are not available or when it is desired to reduce the quantity of rock used for bank protection, it is helpful to grout the voids with portland cement concrete. The grout is normally composed of good strength portland cement concrete and aggregates having a maximum size of 3/4 inch. To obtain a rough textured grout, sand mixes are added. When this method is used for bank protection, filter material may not be needed. However, special care should be exercised in preventing underscour and scour due to overtopping. Because this method is a rigid bank protection method, underscour and back pressure can lead to its failure.

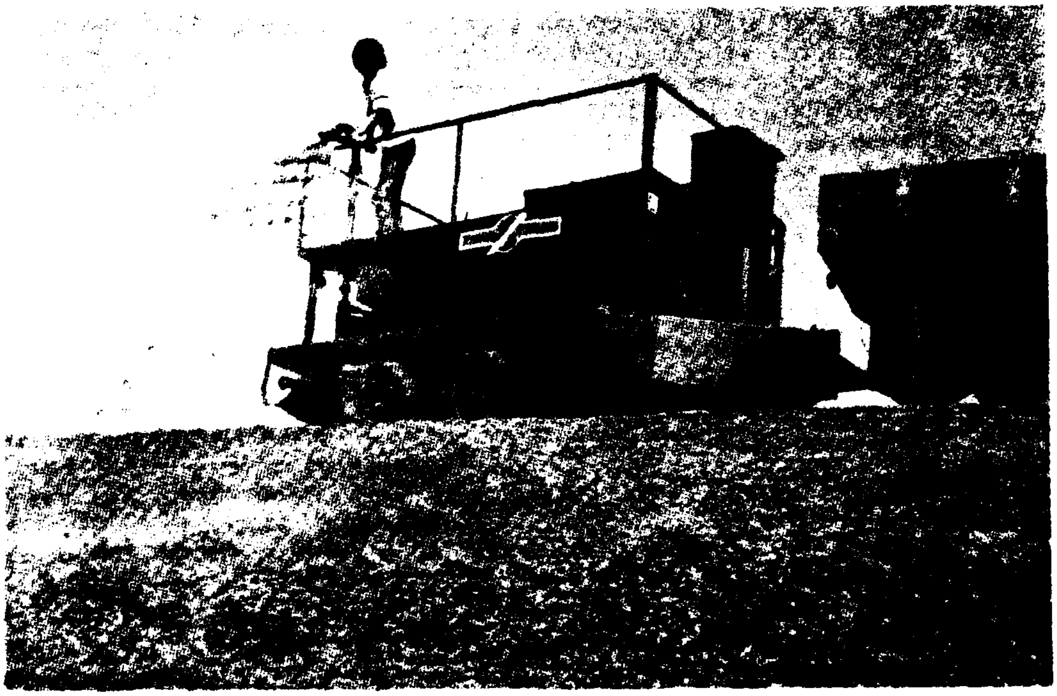


Figure 83. Mechanical broadcaster spreading mulch
on top of stream bank.

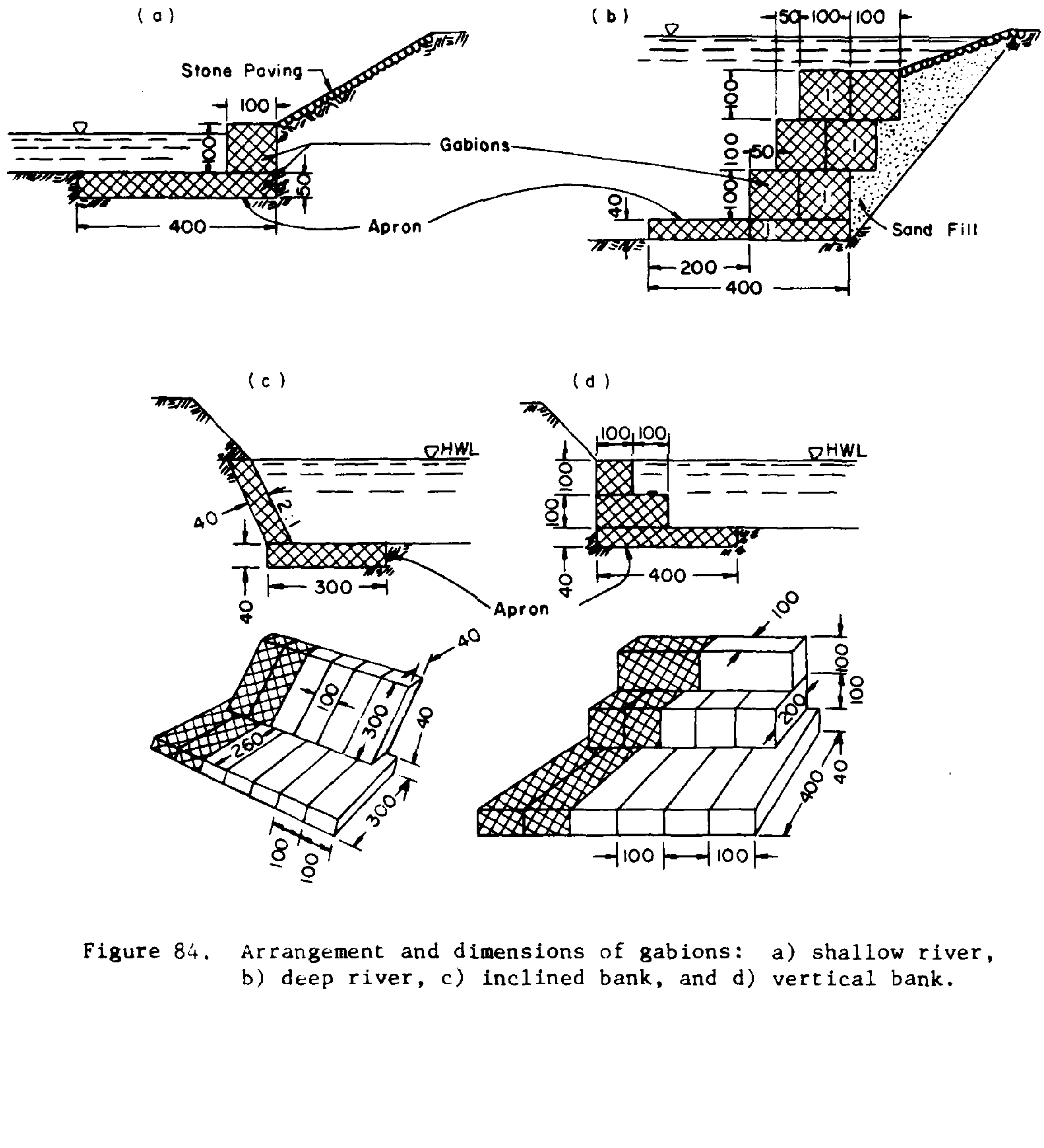


Figure 84. Arrangement and dimensions of gabions: a) shallow river, b) deep river, c) inclined bank, and d) vertical bank.

Figure 85. A gabion placed for work (after Keown et al., 1977).

Grouting can also be applied when the available rock is of inferior quality. In such a case, the rocks and the voids between them should be well covered by 3 to 4 inches of grout. Bank slopes should not be steeper than the angle of repose of the embankment. A slope 1-1/2 H : 1 V is suitable. However, flatter slopes are neither economical nor necessary. Figures 86 and 87 show examples of grouted rock protection.

SACK REVETMENT

Burlap grain sacks accomodating one cubic foot of filling material have been widely used as a quick construction method for emergency work in streambank protection. Soil and concrete are the most common types of filling materials. When sack revetment is intended for long-term protection, concrete or soil cement is used. In this case, a four-sack mix with enough water to produce a slump of 3 to 5 inches is specified. After filling, the sacks are placed by hand on a sloping embankment (slopes can range from 1:1 to 2:1; a 1.5:1 slope is most common) either as headers or stretchers. Common causes of failure of this type of bank protection method are either underscour or end-scour and, therefore, special care should be exercised to prevent these forms of scour.

DESIGN OF CONTROL MEASURES

As shown in Table 8, the predominant type of bank erosion is sloughing. This erosion type can be controlled utilizing nonstructural measures such as flattening slopes, and/or establishing vegetation, etc. If structures are used, the most common methods include riprap and transverse dikes.

Methods for designing riprap and for determining total shear stress were given earlier. An example of design follows.

Assuming that the shear stress on the side is 3.59 lb/ft^2 and the side slope is 2.5 to 1, the safety factor of riprap with a median size of 2.0 ft is determined.

Fo mediar size of 2.0 ft, the parameter η is (from Equation 5-4)

$$\eta = \frac{21\tau_s}{(\gamma_s - \gamma)D_{50}}$$

$$= \frac{21 \times 3.59}{102.96 \times 2.0}$$

$$= 0.37$$

The size of 2.0 ft has an angle of repose ϕ of 41 degrees for dumped riprap (very angular) (see Figure 57). The 2.5 to 1 slope has a side slope of 21.[illegible] degrees. Assuming the horizontal flow on a sideslope, (from Equation [illegible]9)

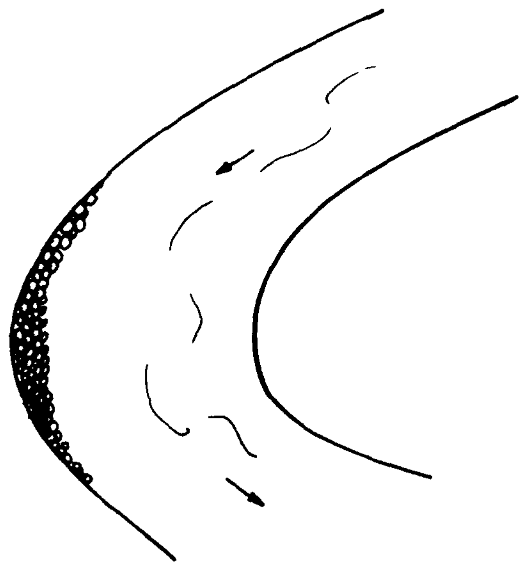


Figure 86. Grouted-rock protection at river bend.

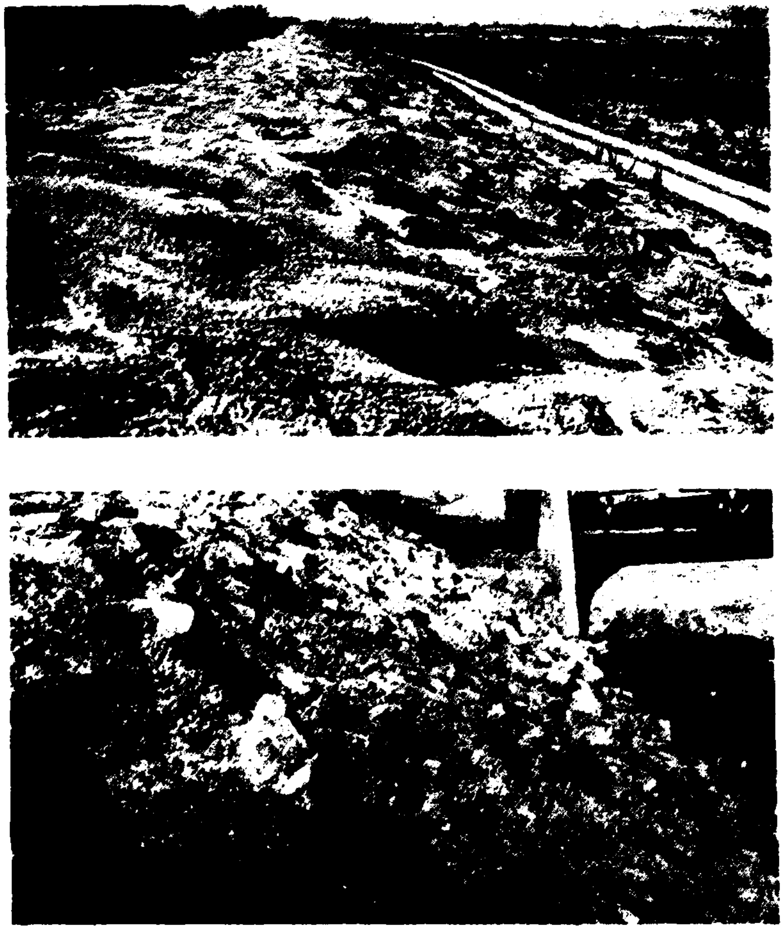


Figure 87. Examples of rough textured grouted-rock protection (California Division of Highways, 1960).

$$\beta = \tan^{-1} \frac{\eta \tan \phi}{2 \sin \phi}$$

$$= \tan^{-1} \frac{0.37 \tan 41°}{2 \sin 21.8°}$$

$$= \tan^{-1} \frac{0.37 \times 0.87}{2 \times 0.37}$$

$$= 23.51°$$

Following Equation 5-11

$$\eta' = \eta(\frac{1 + \sin \beta}{2})$$

$$= 0.37(\frac{1 + \sin 23.51}{2})$$

$$= 0.26 \quad .$$

The safety factor is (from Equation 5-2)

$$S.F. = \frac{\cos\theta\tan\phi}{\eta'\tan\phi + \sin\theta\cos\beta}$$

$$= \frac{\cos 21.8° \tan 41°}{0.26 \times \tan 41° + \sin 21.8° \times \cos 23.51°}$$

$$= \frac{0.92 \times 0.87}{0.26 \times 0.87 + 0.37 \times 0.92}$$

$$= 1.43 \quad .$$

To function properly, transverse impermeable dikes should not depend upon the deposition of sediment between them. Rather, their principal function is to deflect the flow away from the banks. To accomplish this, dikes should extend into the stream past the point where the eroding velocities occur in order to change the position of these velocities along an eroding bank to a new alignment controlled by the location of the dikes.

This type of dike has been known by different names such as groins, spurs, and spur dikes, and can be a variety of shapes: straight, T-head, L-head, hockey and inverted hockey. The different shapes are illustrated in Figure 88. Although stone filled dikes are the most popular impermeable transverse dikes, a wide range of different construction materials such as soil, gravel, sandbags, brush, trees, and gabions have also been used.

Dikes may be angled downstream, upstream, or constructed normal to the bank. Variations such as sloping-crest dikes with decreasing riverward top elevation (systems having either the stepped-upstream or stepped-downstream effect) have been used. For bank protection purposes, the normal-to-the-bank, stepped-downstream system, and sloping crest dikes are recommended. Figure 89 shows an illustration of a typical dike of the stone fill type. Transverse stone dikes are becoming the most widely used impermeable dikes.

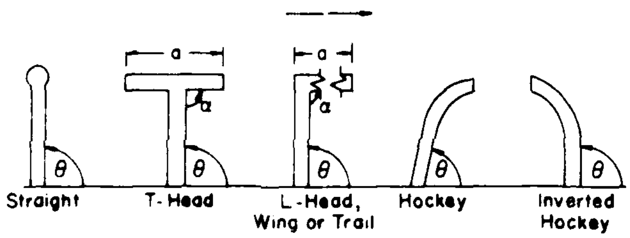


Figure 88. Shapes of transverse impermeable dikes.

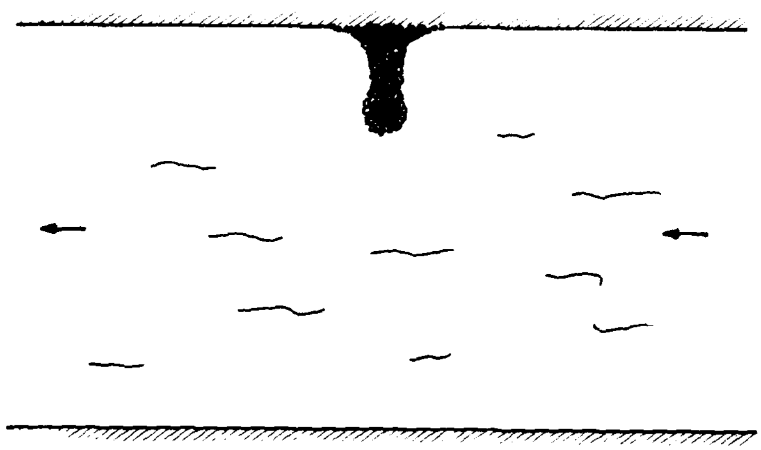


Figure 89. Transverse stone dike.

They are usually constructed from quarry-run stone with specified limitations on the maximum size of stone and amount of fines. These dikes are built with crowns of various widths up to 10 feet or more depending on the severity of the expected attack, the method of construction, and the requirement for maintenance.

The spacing between dikes in a dike field is usually expressed as a function of the length of the dikes, and the relationships that have evolved through the years, based on model studies and some field investigations of existing dike fields. When dikes are placed along the concave bank of the channel for stabilization purposes, it appears that a dike spacing of 4 to 6 times the length of the upstream dike is sufficient for deflecting the attack of the main current away from the bank, thereby protecting the shoreline between the dikes. Dikes used for bank protection along braided or straight reaches, and in mild bendways, can be spaced further apart due to the nature of the flow and the angle of attack of the main current. If dikes are used to contract a section of the channel to develop a well-defined throughway for navigation, it is recommended they be spaced very close together, about 1.5-2 times the length of the upstream dike. In any instance, the dikes should not be spaced farther apart than 1/2 the meander length of the thalweg, otherwise the effectiveness of the dike in controlling the flow may be lost because the stream may impinge upon the bank upstream of the dike.

These recommendations are only guidelines since consideration must be given to the size of the separation zone created behind the dike and the angle of attack of the flow entering the dike field. The extent of the separation zone behind a dike is important in the design of proper spacing between dikes. Unfortunately, information pertaining to this phenomenon of flow around dikes is lacking, and further studies in this area are required before an analysis of dike design can be performed satisfactorily. When designing the layout of a dike field, it must be remembered that the angle of attack of the flow can change with different discharges, especially during flood stages. The angle of the flow relative to the dike position will affect the extent of the area protected from the attack of the flow. When scour is a problem at the riverward end of a dike, riprap is recommended.

It is generally thought that riprap protection is only required around the nose of the spur where velocities are high. However, during high flows the influence of the main stream current may be felt further back into the dike field than previously anticipated if the approach of the flow is changed so that it enters the dike field at an angle θ (Figure 77).

SECTION 6

EFFECT OF VEGETATION ON RIVER BANK STABILITY

IMPORTANCE OF EXISTING VEGETATION ON BANK STABILITY

In evaluating streambank vegetation and relating it to the quality and stability of the river environment, several important factors must be identified. In general, researchers have concluded that bank vegetation adds to the bank stability of rivers of small and intermediate size. In larger rivers experiences have shown that tree roots do not extend to sufficient depths to provide even limited bank protection and lower banks are subjected to erosion by the flowing water that causes bank caving. With the nitching of the lower banks by the flow, it has also been noted that the weight of the trees contributes to bank caving and mass wasting.

Considering specific and detailed effects of vegetation on bank erosion and bank line stability, several important concepts have been verified.

1. Vegetation type is important when evaluating the degree of bank line protection. Small plants, grasses, and shrubs help prevent surface erosion by slowing the velocity of the flow, dampening surface wave activity, and enforcing the soil system to the depth of the root system. Also, roots extending into the flow below the flow line retard velocity, and under some conditions encourage deposition of fine sediments that strengthen the bank and reduce seepage losses.

2. Larger vegetation such as large shrubs and trees may have extensive root systems that extend through a greater mass of the bank providing some reinforcement to the banks, thereby increasing the resistance of the bank line to erosion by the flowing water.

3. With large rivers, the banks are usually of such height that tree roots do not protect the lower bank areas. Hence, the flowing water can erode the underlying layers of bank material making the upper bank unstable, leading to failure by slumping and sliding. In this case, the trees may actually decrease bank stability because of the weight they add to the upper bank, thus contributing to gravitional forces acting on the upper banks.

4. In all cases, all types of bank vegetation help protect the bank line at the surface of the flow from erosion due to wave action, surface velocities, and minor variations in stage.

5. With a few exceptions bank vegetation helps protect the bank from erosion and piping caused by subsurface flows. In a river system like the Connecticut River, frequent changes in river stage may accelerate piping. When piping progresses without trees the overlying block of bank may become unstable and settle down, blocking the passages caused by piping and, consequently, healing itself. With trees the bank is reinforced and does not settle as readily as piping progresses. Thus, piping can extend farther into the bank line before settlement and/or bank sliding occurs. When motion of

the unstable bank material begins, the extra mass of soil plus the added weight of trees growing in the unstable mass may contribute to a greater degree of failure.

6. Wind acting on the trees can transmit forces to the bank material that reduce bank stability. Generally, analyses of the effects of vegetation on bank line stability are favorable for the Connecticut River. The removal of trees, unless replaced with a more effective form of bank protection such as rock rip rap, reduces bank stability. The degree to which bank erosion is retarded by vegetation has been estimated and was shown in Table 2 (page 81). Bank line erosion may increase slightly or very significantly after removal of bank line vegetation depending on river alignment, magnitude of flow, duration of flow, type of bank material, and wave action of seepage forces. It is perhaps unfortunate that processes of bank erosion are so complex. This makes it virtually impossible to quantify the effects unless a particular site is subjected to a detailed analysis.

EVALUATION OF VEGETATION TO STABILIZE RIVER BANKS

The importance of existing bank line vegetation on river bank stability was described in the previous section. It is important to consider the possible benefits of utilizing vegetation to help stabilize eroding banklines. One of the major problems is controlling bank erosion until new vegetation has developed sufficient maturity that the surface of the bank is protected by the vegetation and the bank material is reinforced by the root system. Even then only limited bank protection can be achieved utilizing vegetation in most cases.

There are major considerations that must be observed if new vegetation is to be grown to protect river banks from erosion.

1. The site must be prepared by grading the bank to an acceptable slope of about two horizontal to one vertical depending on the properties of the bank material.

2. Once the bank is graded, some form of temporary bank protection may be needed to limit bank erosion while the vegetation grows to an effective size. This protection might be provided by willow mattresses, combinations of filter cloth and wire mesh, undersized rip rap, or other use of acceptable materials.

3. Types of vegetation utilized depend on the climate, growing season, hydraulic conditions in the river, type of bank material, location of bank to be vegetated, i.e., on a bend or a straight reach. Various types of willows are widely used and often willow logs are buried and anchored to the bank line.

Natural vegetation provides varying degrees of bank protection. However, when attempts have been made to stabilize unprotected banks by planting new vegetation, positive results have been very marginal. Current bank stabilization studies conducted by the U.S. Army Corps of Engineers may provide additional data necessary to help identify the effectiveness of vegetation as well as many other methods of bank stabilization.

Vegetation plays various roles in promoting and maintaining soil stability, acting as a soil binder, promoting infiltration, dissipating rainfall energy, increasing soil porosity, and enriching the surface soil. Maintaining an adequate plant cover then becomes the key to soil stability. The presence or absence of vegetation may determine whether the soil stability balance is maintained or whether soil loss occurs.

There are several general points that need to be made when using vegetation for streambank protection: (1) On those streambanks where erosion is a problem, there is probably little if any vegetation left on the banks. If revegetation is required, good conditions for growth must be established. Fertilization, mulching, and watering are often needed. Site preparation procedures such as sloping or scarification may also be required. It may also be necessary to incorporate organic matter into the subsoil material. (2) Revegetation techniques alone probably will not be sufficient to stabilize the streambanks. Structural techniques, such as riprapping, should also be used on those portions of an actively eroding bank which are placed under prolonged submergence. (3) Timing is highly important. For vegetation to provide useful protection, it must be developed on the bank face between floods. Because it also takes time for vegetation to establish itself as a good protective measure, auxiliary devices should be considered. By knowing the amount of time required by various kinds of vegetation to become established, an effective program can be designed that incorporates both structural and non-structural measures. For example, woody species take longer to mature than do grasses, but they afford long-term protection. Quick-growing grasses should be seeded first, along with shrubs and small trees. (4) Grasses can provide a very effective stream channel lining. It keeps the fast-moving water and transported coarse materials away from the bank's soil surface. Something should be known, then, about species length, number per unit area, and the physical qualities of the tops. The roots are also highly important. Good sod grasses, such as Bermuda grass, have an extensive root system that resists the pull of the tops that results from the drag of the flow. The ability of a lining to protect the soil surface depends in part on the toughness and resilience of its species component. To remove the trees and introduce hydro-seeding without proper studies of site preparation, species selection and maintenance has proven to be ineffective along the Connecticut River.

The decision as to which species should be used is contingent on site conditions. At the outset, information should be gathered on the soils, vegetation, and topography. For example, soil sampling will help determine the fertilizer and soil amendments that might be necessary to establish a good vegetative cover. Soil properties important to revegetation include:

1. Electrical conductivity (EC)

2. Percent base saturation

3. Exchangeable sodium percentage (ESP)

4. K-factor

5. pH

6. Soil texture

The natural vegetation serves as an index for determining revegetation potential. Distribution of natural species is a response to factors such as slope, aspect, soil, elevation, and moisture availability. Knowing types, numbers of plant species, and growth habits indicates the type of plants that would establish well and aids in designing a species mixture that leads to a return to natural vegetation. Many riparian or bottomland species grow well in an environment that could be considered "disturbed" (i.e., by periodic flooding). Therefore, it is important to consider the willows, maples, grasses, and sedges in the species mixture.

Site Preparation

Selecting the right species mixture is not sufficient to guarantee plant establishment on the site. No doubt very poor growing conditions exist; the topsoil has been destroyed and the site is in a highly unstable condition. Site preparation procedures should be followed. These procedures, which include the addition of topsoil, fertilizer, mulches, and soil binders, constitute the majority of the program in both materials and cost. The exact program followed will be based on the results of the initial soil testing and vegetation survey. It may also be necessary to modify the site by sloping and soil ripping. Depending on the properties of the bank material, the bank should be graded to an acceptable slope of about two horizontal to one vertical.

Mulches are extensively used as a temporary erosion control measure. They also improve the soil environment by augmenting germination and seedling growth. Determination of the best mulch treatment is based upon the predicted effectiveness and site characteristics. An effective mulch will certainly shorten the time required to revegetate a site. Organic mulches, straw, hay and wood products are the most commonly used. They effectively moderate soil temperature and improve moisture relationships. Determining the most effective mulch for any one site may require that the applications of different mulch treatments be tested. On areas that are highly unstable, mulches of crushed stone or gravel (1 inch deep) have been found to provide more effective erosion control than 4,000 lb/ac of straw. A ground cover of gravel stabilizes and protects a site against wind and water erosion, permitting invasion by indigenous species. An eroding riverbank is obviously not the most favorable place to establish vegetative cover. Use of rock mulches should be considered, then, for those sections of the streambank that are especially unstable, where regular revegetation techniques would fail.

Species Selection

The species selected for utilization will depend on the site conditions, hydraulic conditions in the river, the bank material, and the location of bank to be stabilized, i.e., on a bend or a straight reach. To reiterate, the goal of revegetation is to ensure long-term stabilization, eventually returning the site to its natural vegetative state. Plants that naturally encroach on disturbed sites in the area can be used as indicators for selecting plants that adapt to adverse conditions. In selecting species for use along the Connecticut River, the following characteristics should be considered:

1. Flood tolerance

2. Life cycle

3. Vigor of seedling habits

4. Physical characteristics:

   - stem length
   - number/unit area
   - root density (soil-binding value)
   - stiffness

5. Ability to spread naturally and readily by seed or vegetative means

6. Ease of establishment

7. Availability--the choice of species and variety is limited most often to what is commercially available and to species that can be established on disturbed soil. Transplanting species already growing in the area should also be considered.

Again, a survey of site conditions greatly enhances species selection, especially in directing the revegetation toward natural vegetative cover.

Because the initial goal is to stabilize the soils, a seed mixture must be selected that will establish quickly. For this reason, a grass mixture is usually seeded in the initial stages of rehabilitation. Table 12 lists grass species that have been used in eastern states. Often, soils located in sites that have been seriously eroded are deficient in nitrogen and phosphorus. For this reason, legumes are often added to the species mixture. However, Parsons (1963) stressed that legumes are not as effective for streambank stabilization because they are generally weak in retarding flow. Use of a good nitrogen fertilizer would be preferable.

An effort should be made to include woody species in the revegetation program to promote long-term site stability. Table 13 lists trees and shrubs that have been used in revegetation in Massachusetts and which could have some potential for use in Connecticut (EPA, 1975).

Willows have been used in a variety of ways. Willow logs are often buried and anchored to the bank line. A growth of willows springs from the logs, while the logs help provide bank protection until a good growth develops. They can also be planted. When such procedures are used, the banks need to be seeded with a herbaceous ground cover just prior to or immediately after planting. The best time to launch such a program is after the high spring flows. This allows maximum opportunity for the vegetation to grow during a period of minimum flows.

The advantage of using large shrubs and trees is their extensive root systems which extend through a greater mass of the streambank, thereby providing reinforcement to the bank. This increases the resistance of the bank line to erosion by flowing water. The result is long-term stabilization.

Table 12. Grass species used for revegetation and erosion control.

Ryegrass (perennial)
Creeping Red Fescuegrass
Cereal Rye
Fall Fescuegrass
Kentucky Bluegrass
Smooth Brome
Reed Canarygrass
Crownvetch

Table 13. Woody plants for erosion control.

Autumn Olive
Bayberry
Bearberry
Oriental Bittersweet
Red Cedar
Indigo Busy
Inkberry
Japanese Larch
Bristly Locust
Plum
Willows
Ash
Alder
Dogwood

SECTION 7

EVALUATION OF ECONOMIC AND ENVIRONMENTAL IMPACTS CAUSED BY CORRECTIVE MEASURES

IMPACT OF EROSION

Rivers are continually subjected to erosion. The rates and magnitudes of erosion are dependent on the magnitude and duration of the forces causing erosion. Man's activities on the watersheds and in the channel system can significantly alter channel bank erosion. Because of concern for these systems, it is essential to identify and evaluate both natural and man-related causes of erosion. Where it can be proven that man's activities have accelerated the erosional processes, it may be necessary to take remedial action to avoid deterioration of channels by sediment deposition, loss of valuable adjacent land and property, loss of reservoir storage, and adverse impacts on the biomass of the system. However, it should be recognized that channel stabilization can result in possible adverse impacts on the natural environment. For example:

1. Bank erosion control measures impede natural erosion.

2. Bank stabilization encourages development of the riparian land and floodplain.

3. Maintenance of channel stabilization works will require periodic access to the bank lines.

4. Bank stabilization affects natural vegetative succession and floodplain and aquatic habitat.

5. Bank stabilization will reduce aesthetic value of the area for recreational and riparian land owner uses.

With adequate knowledge of the natural dynamics of rivers supplemented with knowledge of the response of watersheds, most serious adverse impacts that may occur as a consequence of natural phenomena or man's activities can be identified and, in many instances, avoided.

There are two methods of predicting system response. These methods utilize physical and mathematical models. The physical model is severely limited in its application because it is difficult to consider a component of a watershed, river channel, or shoreline of appreciable size. The mathematical model is not limited in this sense; however, the size and detail of the model may be limited by computer facilities. To study a transient phenomenon in the watershed and/or river environment, the equations of motion and the continuity equations for water and sediment are valid. However, because of the complexity of the equations, many solutions can only be obtained by numerical analysis using iteration procedures and digital computers. Only recently has the potential of mathematical models for analysis of flood and sediment routing, bank erosion, aggradation and degradation, and long-term system response studies been recognized.

When considering river problems it is apparent that both natural and man-related factors have a significant effect on the geomorphology and hydraulics of systems. In addition, there may be an immediate response as well as a long-term delayed response of the system to the conditions to which it is subjected as described below.

1. Constrictions due to encroachments generally cause local scour of the bed and banks of rivers. The sediments derived from this source are often deposited in the immediate wider reach downstream, thus affecting the hydraulics and biomass of the system.

2. Construction sites are often the source of appreciable sediments since these areas are usually highly susceptible to erosion. Also, the erosion usually increases the suspended fine sediments in adjacent drainages. These suspended fine sediments can have very significant effects on the biomass of the system.

3. Development of the bank line areas has the potential of adversely affecting water supply.

4. Shortening the bends by implementing a cutoff will generally cause local scour at the cutoff, create oxbow lakes and deposition in the old bends, and aggradation in the downstream reach.

5. The operation of the hydro-pools increases bank erosion in the pools and to a limited extent downstream of the pools.

6. The use of recreational power boats and others will generate waves that increase bank erosion.

All of man's river activities can have significant impacts on other components of the system. These components include man's affect on channel stability, biological, and economic concerns.

MAN'S EFFECTS ON CHANNEL STABILITY

All rivers exhibit some degree of bank erosion. Only rock controlled and concrete lined channels are fixed and even these can be altered during periods of extreme flood, earthquake, and other severe conditions. Studies of the Missouri River document major lateral shifting with time. One farmer documented the total loss of a 360 acre farm to bank erosion in one growing season. This was not an uncommon occurrence prior to recent channel stabilization and flood control projects. A study of Mississippi flood records documents that the river has shifted its position in a random manner from one valley wall to the other; a distance of many miles.

It is accepted by geologists, engineers, and geomorphologists that bank erosion is a natural phenomena common to all alluvial rivers. However, the rates of erosion may vary significantly depending on discharge, slope, channel slope, bed and bank material, freezing and thawing, pool fluctuation, and wave action. As explained earlier, erosion is most always observed on the outside of river bends and in straight reaches opposite alternate bars form if the bank consists of erodible materials. Figure 46 illustrates the typical phenomena of lateral movement of a meandering river. Lateral movement increases the natural erosion in alluvial channels. In addition, man-induced changes can increase or decrease bank erosion rates depending on how the channel and flow conditions are modified. The Connecticut River is an excellent example.

One of the principle factors that reduce bank erosion is the operation of both large and small storage reservoirs constructed for flood control, hydro power development, recreation, etc. Large reservoirs can significantly reduce the peak flows as is true on the Connecticut River (see Figures 23 and 24, pages 33 and 34). Also, hydro power pool fluctuation can significantly increase bank erosion depending on reservoir operation, type of bank material, vegetation, etc. (Table 7). As emphasized, the major portion of bank erosion that involves significant shifts in bankline position is associated with major floods. On one hand major reservoirs reduce flood peaks and therefore increase channel bank stability but conversely small reservoirs that experience frequent and significant pool fluctuations give rise to bank erosion of a different type that slowly erodes unstable banks causing local problems for those that occupy the river bank environment.

The utilization of reservoirs with a high trapping efficiency also affects the sediment load of the system. The storage of sediment in the reservoirs reduces the sediment discharge downstream. This reduction in sediment load can result in a series of responses in the system--either favorable or adverse. With reduced sediment load and reduced peak flows, the system transports less sediment. The reduced transport of sediment may create a more stable system. There are exceptions, however. If the river bed downstream of a dam is erodible, the channel may degrade because of the release of clearer water causing a deeper channel with less stable banks. With channel degradation the gradients of tributaries are steepened causing increased velocities and increased sediment loads that may adversely affect the main stem. These adverse effects are negligable on the Connecticut River because of rock controls, dams, and coarse material in the bed of the tributaries and the river that prevent significant channel degradation.

Another type of dam is the low head hydropower dams on the main stem. These dams trap some of the sediment moving during periods of low flow; some of which is flushed out of the pools during periods of high flow. A major effect of the low head pools on river regime is a higher than normal stage during periods of low flow. Also, periodic variations in pool stage caused by sequential drawdown during the week days with some recovery at night leads to a relatively low pool level on the last work day of the week. Refilling of the pool over the weekend depends on river discharge and power demand. The fluctuation of pool level with time causes variations in stage and cyclical flow reversals in the banks that result in seepage forces and piping that reduce bank stability and cause bank erosion to a limited degree (Table 7). As a result of higher stage in pools: wave action impinges on the banks at an elevation higher than normal (This may be advantageous since the waves could be more detrimental to bank stability if they dissipate their energy at a lower level on the bank.), and seepage forces, piping, and mass wasting that would otherwise occur under normal river operation may be reduced. These advantages do not necessarily outweigh the adverse impact of pool fluctuations caused by pool operation, and the increased river stage which allows the water and associated forces out on more erodible upper bank and terrace material, but do tend to offset this effect to a limited degree. Kurz (1979) reported that the dam is the direct cause of the bank erosion in the Connecticut River.

IMPACT OF BOAT-GENERATED WAVES ON BANK STABILITY

Referring to Table 2, the various causes of bank erosion are identified and quantified in comparison with bank erosion caused by the shear stress exerted on the channel banks by flowing water. Based on Table 2, the relative magnitude of bank erosion for different factors is summarized in Table 7. This table documents that average boat waves generate erosive forces on the banks with a magnitude on the order of 9-12 percent of the shear stresses caused by the flowing water in an unrestricted channel system.

To reduce the impact of boat-generated waves on bank stability, the following actions could be implemented singly or in unison:

a) Restrict power boating to a maximum speed of approximately 5 mph.

b) Limit recreational boating to a zone in the central part of the river keeping a buffer "no traffic zone" within 25-50 feet of the bank.

As stated above, such actions could reduce bank erosion on the order of 5 percent.

IMPACT OF HYDRO-POOL OPERATION ON BANK STABILITY

The operation of the hydro-pools increases bank erosion in the pools and to a limited extent downstream of the pools. Referring to Table 7, shows that erosional forces acting on the banks due to pool fluctuation are on the order of 15-18 percent of the shear stresses caused by the flowing water in the unrestricted reaches of the river.

In general:

a) Complete elimination of hydro-pool fluctuations would increase bank stability in the pools on the order of 15-18 percent.

b) Reduction of bank erosion as related to pool fluctuations is assumed to be linear. Hence, reducing pool fluctuations by 50 percent would reduce bank erosion on the order of 7-9 percent.

As one considers the adverse impacts of hydro-pool fluctuations on bank erosion, it is essential to simultaneously consider the favorable impacts of pools on bank stability. Referring to Table 2, it may be noted that within the pools, velocities and shear stresses are reduced. Figure 53 demonstrates that on the average the computed velocity in the pools is 20 percent smaller than in the natural river. This results in a reduction of shear stress on the order of 40 percent. These reductions may increase the stability of the system on the order of 20-50 percent depending on bank height, type of bank material, location in the pools, etc. Based on Table 2, the relative magnitude of bank erosion for different conditions (natural river, pools, high banks, low banks, etc.) is summarized in Table 8. This table shows that factors causing bank erosion in the pools are on the order of 5-41 percent less than for the natural river. Hence, the benefits outweigh the adverse aspects. Also, upstream storage provides an effective means of reducing peak flows during periods of flooding, which further reduces bank erosion in the study reach.

An analysis of the data at the test sites established by the Corps of Engineers verifies that bank erosion is at least as severe in the non-pool reaches as within the limits of the pools. In fact, the measured data indicates that the natural river is 1.30 times more susceptible to bank erosion than are the pools (Table 9). This is very close to the theoretical evaluation, which yielded a value of 1.34. In other words, the presence of pools reduces bank erosion on the order of 34 percent compared to the natural river.

By altering the operation of the hydro-pool in order to maintain selected pool levels for extended periods of time (for example 30 days plus), the pool fluctuation at most will be reduced about 50 percent. This will reduce the bank erosion on the order of 7-9 percent as mentioned earlier. This may represent an insignificant gain in erosion control compared to the loss of power generation. A similar conclusion applies to a complete elimination of hydro-pool fluctuations. It should be stressed here that the pool fluctuations at most contribute approximately 18 percent of the bank erosional forces. This quantity is much smaller than the determined 34-percent increase in bank stability due to reduction of shear stress in the pools as compared to the natural river. Hence, a total elimination of hydro-pool fluctuations will not eliminate bank erosion in any river system.

ADVERSE IMPACTS THAT WOULD RESULT FROM LIMITING HYDRO-POOL FLUCTUATIONS

The four hydro-power plants generate a small but very important part of the energy needs. The power demands of the region are a function of time of day, season, characteristics of the power network, etc. To meet the increasing needs for energy, particularly during peak demand periods each day, it has been necessary to run larger discharges for short periods of time, which has increased pool fluctuations. If pool fluctuations were limited, it would impose larger power requirements on other sources of energy (fossil fuel and nuclear) to offset reduced peak power production. A meaningful cost analysis of the impacts of curtailed hydro-pool operation is a complex task that requires an analysis of all power sources and methods of pooling and utilizing them. Such a detailed cost analysis was not possible with the limited funds, time and data available. Discussions with the power companies during the field data collection period and subsequent analysis revealed that operation of the pools at various levels would require an extensive study to evaluate the economic impact to the power network within the area. However, basic operational design underlying the low-head hydro-turbine operation indicates that an extensive reduction in power output will result from small decreases in the design head. Therefore, any reduction to the hydro-pool fluctuation that provides a detectable increase in the bank stability will result in a significant reduction in power generation. This is a significant loss in benefits considering that such plants provide:

a) a clean method of producing energy, and

b) an excellent means of helping to meet peak power demands.

Certainly restrictions on hydro-pool fluctuations would adversely effect ability to meet current and future energy requirements.

BIOLOGICAL IMPACT

The primary impact of streambank erosion is on the riparian land and the aquatic system. The sediment affects the water quality that in turn impacts the structure and composition of plant and animal communities. Thus, any form of bank protection assures higher levels of water quality by controlling sediment.

Sediment affects the physio-chemical character of the aquatic system, and consequently influences the aquatic biota. Sedimentation affects such parameters as temperature, light, nutrient load, dissolved solids, and BOD--all of which determine the makeup of the plant and animal communities dependent on the river.

Sediment added to aquatic systems influences the systems' living organisms through contact in the aqueous medium due to the physical and chemical changes that silt introduces in the waters. Alterations in bottom conditions resulting from subsequent settling of all or part of the silt load affects the living organisms as well. Within the water itself, sediment alters aquatic environments by limiting the penetration of visible light and by altering water temperature. The reduction in total light will alter the composition of communities of submerged aquatic macrophytes with photosynthetic organs submerged below the water surface. A relatively small increase in turbidity in an already marginal or near-marginal aquatic light climate as a result of naturally high turbidity will be of immense significance to plants growing there.

Limiting the floral composition will naturally limit vertebrate and invertebrate groups both in number, species, and composition. Specifically, sediment will have direct and indirect effects on numbers of plankton, benthic organisms, and fish species. Negative impacts are due not only to the absence of macrophytic vegetation, but also possible adverse effects of the sediment on water quality. In these ways, sediment may alter quality of the aquatic habitat.

ECONOMIC IMPACT

If the water quality of the river is affected, the composition of the fish populations is also affected. This could potentially affect not only recreational fishing but commercial fishing as well. Therefore, effective bank protection would help improve the present quality of the river for recreational and commerical fishing. Also, selective bank protection is necessary if one is to protect the riparian land owners regardless of cause of bank erosion. However, the feasibility of utilizing structural bank protection works is questionable and each unstable site must be evaluated on its own merits.

SUMMARY

The preservation of environmental quality is a matter of national interest and priority. Hence, all activities that may impact the natural channel environment must be carefully evaluated prior to implementation in order to verify that adverse impacts have been minimized considering the effects on natural, recreational, biological, and economic values along with the resources of the area.

In conclusion, the river banks are relatively stable. All rivers experience some degree of bank erosion. The river valleys were in fact formed by the erosional and depositional characteristics of the river system. To totally eliminate bank erosion is impossible. The outside of a bend usually experiences erosion unless reveted. Major floods will always have the capability to make significant changes in bank lines regardless of steps taken short of total regulation and installation of extensive costly bank protection works. In the case of the Connecticut River, natural forces causing erosion have been reduced by smaller velocities, reduced boundary shear, and reduction of flood peaks due to presence of upstream reservoirs and the hydro-pools associated with the low dams. However, the increase in river stage due to impoundment allowed the water and associated forces to act at more erodible upper bank and terrace material. Thus, the bank erosion due to the presence of an impoundment may actually increase due to a much larger availability of sediment.

SECTION 8

RECOMMENDATION OF BANK STABILIZATION MEASURES

GENERAL

The magnitude of the forces causing bank erosion was discussed in Section 4. These forces represent the capacity of flowing water to erode the bank. The erodibility of bank material is also an important factor in controlling bank erosion. As described earlier, the bank erosion problem is very complex. Both the magnitude of forces and the erodibility of soil play an important role in channel bank erosion. In Section 5, types of channel stabilization that are potentially applicable to the Connecticut River were described. Section 6 discussed in further detail the use of vegetation and Section 7 evaluated the potential economic and environmental impacts caused by corrective measures. Based on the analyses made in the previous sections, general recommendations of bank stabilization measures for the Connecticut River are made in this section.

The concepts, principles, and methodologies presented in the previous section are directly applicable in designing an erosion control measure for each study site. This section only provides a general rule for bank stabilization, no site by site recommendations are made. Only detailed analysis of each site will yield a more precise and practical design. Approximately a two to four man-month effort will be required to design a stabilization work for a study site.

The construction and operation of low head hydro power dams has significantly changed the erosion patterns along the banks of the rivers. In the natural river depth of flow was dictated by the resistance to flow and geology of the channel and discharge. Over recent geologic time a fairly stable channel system evolved, i.e., only 15-20% of the bank was estimated to be subjected to varying degrees of erosion. The major forces causing bank erosion in the natural system are shear stress or velocity associated with the flowing water and seepage forces associated with changes in river stage. During periods of low to moderate flow bank erosion was minimal, occurring mostly on the outside of river bends and opposite major bar formations. The channel was relatively stable because the river channel bed adjusted to the average flow regime. Most erodible materials in the natural channel bed had been removed during recent geologic times.

After construction of the low head dams, water levels were permanently increased to levels that experienced flooding only during unusual flow events. This means that at the new average water levels, land surfaces behind the dams were now at levels more susceptible to erosion. In fact in many locations surface materials are at the point of incipient motion such that any small force would be sufficient to initiate bank erosion. These conditions are illustrated in Figure 90.

Dams deepened the water and slowed velocities such that bank erosion due to the flowing water was reduced. Even though the shear stress on the banks is reduced except during periods of flooding, shear forces combined with seepage and gravity forces induced by power-pool operation and wave action induced by wind and boat waves were and are adequate to cause erosion at the

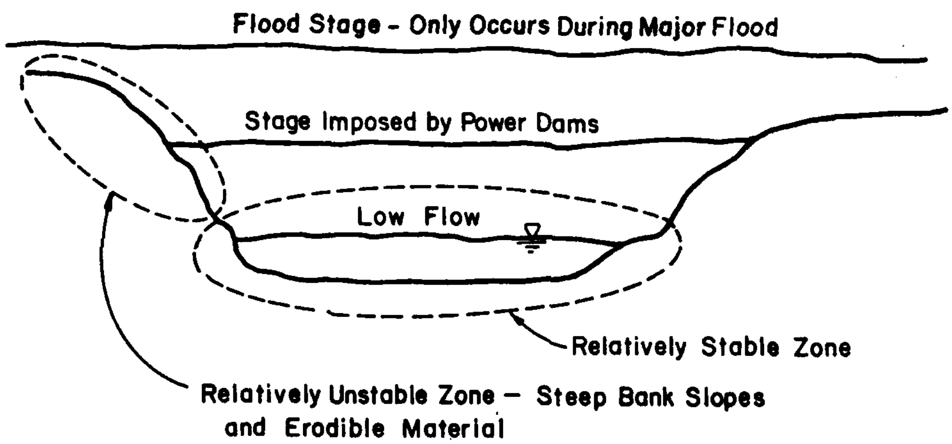


Figure 90. Bank stability at levels of low flow, hydropools and flood stage.

new water level on the upper bank. Although these forces are not large in magnitude compared to forces acting on the banks at flood stage, the erosion caused by this combination of factors is significant because the forces have acted continuously and are confined within a fixed zone imposed by the dams and adopted operational techniques. The erosive action imposed on the high banks has resulted in the development of terraces or berms as indicated by Figure 91.

Having recognized that the current pool level and methods of pool operation induce bank erosion, several questions arise that must be answered:

1. If remedial actions are not taken how much bank erosion can be anticipated?

2. What remedial actions are feasible for limiting bank erosion?

In response to these questions it is estimated that as erosion occurs its progress will slow and in time, and with some additional loss of bank line, the system will stabilize. As much as 10-15 feet of additional bank erosion can occur, the amount being dependent on height of banks, types of bank material, magnitude and frequency of pool fluctuations, wave action, and other forces acting on the bank. Only detailed analysis of each site will yield a more precise estimate. Table 2 provides a means of estimating the magnitude of forces acting on the banks within the study reach. As discussed in Section 4 different forces act on and are responsible for upper and lower bank erosion. If the control is only effective to limit upper bank erosion, the protection will fail when the lower bank erodes due to shear stress exerted during major floods. Then the additional bank erosion will occur.

With regard to question (2) various structural and non-structural methods could be utilized to stabilize the banks as identified in Sections 5 and 6. A summary of both non-structural and structural stabilization measures is given in Table 14. However, few of the structural measures are economically feasible for riparian owners at this time. Hence nonstructural approaches provide the most viable method of limiting erosion at present erosion sites. A general recommendation for each erosion type as defined in Figures 59-63 is given in Table 15. From the table it is thus possible to make broad recommendations for each particular erosion site.

The bank erosion problem has been analyzed and a general summary, identifying the total magnitude of the erosion forces, was presented in Table 3 for the six general conditions that occur along the study reach. Also, similar data are given in Table 6 for the six index sites.

In selecting remedial measures to control or prevent bank erosion, it is necessary to consider the forces causing erosion, the geometry of the site including bank height and channel curvature, the lack or presence of natural controls such as rock outcroppings, the type of bank material, the presence or absence of vegetation, and the location of the specific site with respect to dams and pools.

Table 14 summarizes the most generally applicable non-structural and structural methods of controlling and/or preventing bank erosion. These methods are consistent with the recommendations made in the section dealing with channel stabilization.

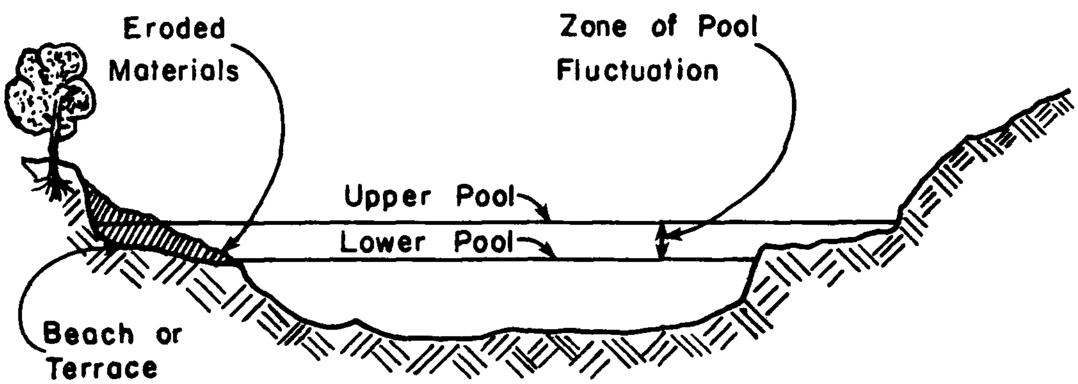


Figure 91. Beach formation due to bank erosion caused by pool fluctuations.

Table 14. Summary of stabilization measures.

Non-Structural	Structural
1. Flattening bank slopes	1. Rock riprap
2. Control pool fluctuation	2. Transverse dikes
3. Vegetation	3. Fences
4. Limit recreational boating	4. Gabions
5. Do nothing (natural stabilization)	5. Grouted rock
	6. Sack revetment

Table 15. General recommendation of bank stabilization measures.**

Erosion type	Recommended Corrective Measure
Mass wasting on a vegetated or barren bank	No efficient remedy except flattening slopes plus other measures such as vegetation
Material slides down bank as individual particle	Flattening; vegetation; rock riprap; tire revetment
Head cutting on a vegetated or barren bank	Rock riprap; drop structure (gabion)
Shallow washing on a barren bank	Vegetation (hydro seeding), netting, jutte mattress, sack revetment,* transverse dike
Undercutting on a vegetated bank	Fences, transverse dikes, gabions, flattening and revegetation

*Temporary measure.
**See Appendix C for other stabilization options.

Extending the concepts of stabilization further, bear in mind the subdivision of forces causing erosion into those which attack the upper bank mainly forces resulting from pool fluctuations, wave action, ice, etc.) and forces attacking the total bank (namely the tractive force exerted on the banks by the flowing water). As discussed in Section 4, when evaluating the forces causing bank erosion it is essential to consider both categories of forces when both sites out on the channel banks in question.

To summarize, the most important concepts applicable to the control and/or prevention of bank erosion along the Connecticut River it should be noted that:

1. For the reaches of each channel above the headwaters of the pools, channel stability could be achieved by stabilizing the lower two-thirds of the active bank. This would prevent further lateral channel erosion during periods of high flow. Simultaneously the upper banks where vulnerable would erode but only sufficiently to form a narrow berm, ultimately becomes stable and in time develop a reinforcing vegetative protection of willows, shrubs, grasses, etc.

2. If it is desirable to accelerate development of upper bank stability in the study reach, two techniques could be considered.

   a. The upper bank could be flattened to an angle 5-10 degrees flatter than the angle of repose of the natural material and it could be protected with an acceptable structural treatment; for example, a reduced size of riprap placed on a suitable filter.

   b. After the berm had developed as a result of upper bank erosion, the raw unstable bank line at the limit of the berm could be shaped to an angle 5-10 degrees flatter than the angle of repose of the natural material. Then the upper bank could be revegetated. Of course, it would be very beneficial to the development of upper bank vegetation if pool fluctuation could be greatly reduced for a period of 2 to 3 years while the new vegetation developed some maturity. Thereafter, pool fluctuations on the order of 2-3 feet should not be significantly detrimental to upper bank erosion.

3. In the pools depth has been increased by pondage and velocities are reduced except during the periods of floods. The lateral shifting of the main channel where active erosion is occurring could be controlled as in (1). However, the placement of lower bank protection is more difficult because of greater depth of water. The cost of placement of lower bank protection in this environment would be larger than for the non-pool reaches of river for a variety of reasons, including difficulty of shaping the submerged lower bank, placement of filter material and finally placement of protective materials capable of withstanding the forces exerted on them during periods of high flow.

4. Also, selection or placement of protective treatment on the upper banks in the pool areas is more difficult and more costly. However, treatment as described in (2) is possible. The major difficulties pertaining to upper bank protection are a general change in the species of vegetation that will grow along the waterline. With the increase in stage caused by the dams, many of the natural species may not survive the wetter environment.

SPECIFIC RECOMMENDATIONS

A combination of seepage forces, increased gravitational forces, and wave action tends to develop a typical beach in the upper bank as indicated in Figure 92. As the beach widens the erosion rate of the vertical bank is reduced. Following the suggestion made earlier, a specific non-structural measure to control bank erosion for this example case is recommended as follows:

(1) Operate pool at reduced level - maximum at the top of the beach, minimum will depend upon energy requirements.

(2) Plant willows on beach.

(3) Allow time for willows to start roots and grow 1-3 feet in height.

(4) Willows will stabilize beach area.

(5) With adequate willow growth go back to original pool operation.

(6) Willows will dissipate the energy of waves.

(7) Seepage forces will continue to slowly fail vertical embankments as shown at location A in Figure 93.

(8) Much of the materials from the bank will deposit in the willow zone. This zone will continue to act as a buffer and will be stabilized by willows.

(9) Ultimate bank line should take the form indicated in Figure 94.

(10) The bank will be stable except for periods during major floods. Remedial work may be required after all significant floods at locations only partially stabilized and where banks may be subjected to ice action.

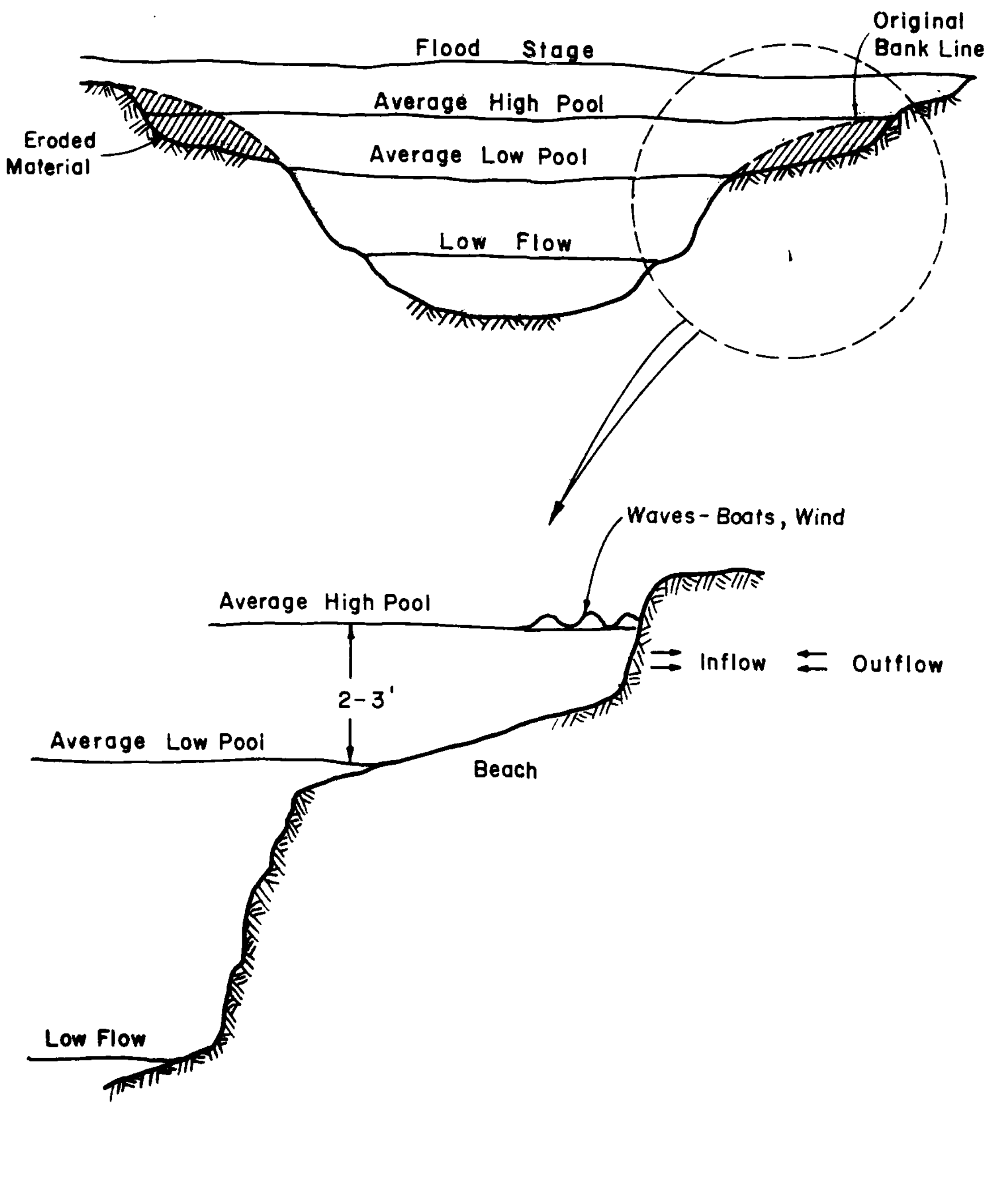


Figure 92. Bank erosion due to seepage forces, increased gravitational forces and wave action.

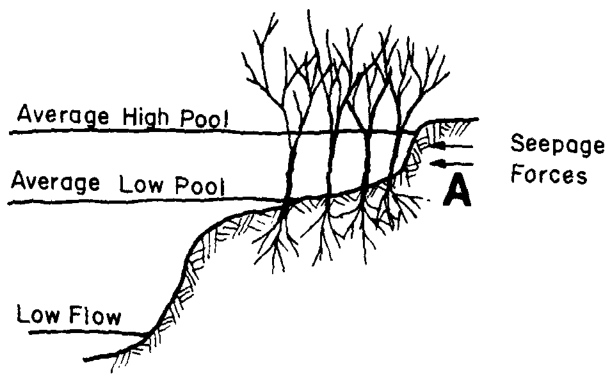


Figure 93. Willow stabilization of beaches at reduced pool levels.

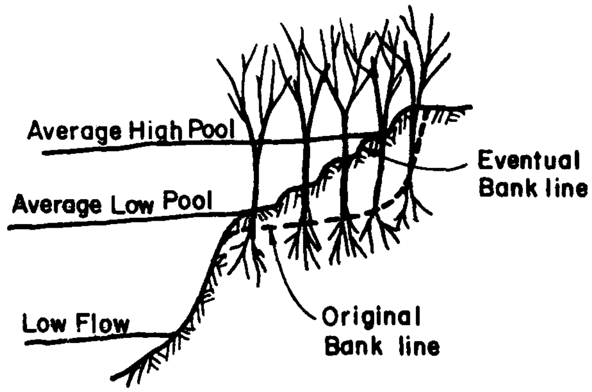


Figure 94. Willow stabilization of beaches at higher pool levels after additional bank failure.

SECTION 9

IDENTIFICATION OF INFORMATION GAPS

GENERAL

Due to the complex nature of erosional processes, and interaction of the variables and forces that cause erosion, the mechanics of erosional patterns of channels and banks are inadequately understood at the present time. A better understanding of these erosional processes can only be made through a detailed evaluation of adequate data. If such data are not available, data collection programs on either a short- or long-term basis should be initiated. Such programs will assist in delineating problems and provide insight into remedial measures that may increase the stability of the river system. However, data should be collected on a predetermined, systematic basis, and from critical locations in such a way that the data will have extensive application to analyses of the dynamics of the riverine environment.

As discussed in Section 2, data are available for the study reach of the Connecticut River that can be used to identify the most important hydraulic and hydrologic parameters within the system. However, geometric and sediment data are inadequate for a detailed analysis of the stability of the study reach. It is strongly recommended that to aid future studies, a data collection system should be designed and implemented to correct these deficiencies.

Within any river system, it is essential to predetermine the spatial and temporal resolution required in order to fulfill specific short- and long-term data requirements and related needs. With possible increased future development and utilization of the Connecticut River, data collection becomes especially important in order to predetermine potential changes in the physical attributes and the environmental quality of the present and future system.

The most significant data gaps on the Connecticut River are the lack of an adequate description of velocity distributions as a function of river geometry and stage, channel geometry and basic delineation of characteristics of the river system in quantitative terms. Analyses indicate that inadequate information exists regarding the relations connecting hydraulic parameters, such as depth, width, cross-sectional area, and bed slope, with discharge at sites throughout the study reach. Additionally, specific studies are suggested to evaluate the effects of channel geometry, sediment transport, resistance to flow, ice development and break-up, wave action, seepage forces and removal of bank vegetation on channel stability.

It is recommended that a basic monitoring program be established at several locations on the river, and that these sections be referenced to permanent mean sea level benchmarks to allow comparative evaluation of cross-sectional shape changes over long- and short-time periods. These data are essential to quantifying the volumes of material eroded from the banks, to better define potential erosion sites, and to better determine degradation and aggradation problems in the study reach.

The number of new channel cross section measurement stations should be determined based upon this stability analysis of the river, accessibility, cost, and qualified personnel available to collect the data. Important sites are above and below confluences of major tributaries, reaches of observed instability, on bends, and above and below the hydropower dams.

In conjunction with channel changes, it is strongly recommended that a sediment data collection program be implemented to assist in further delineation of water quality and sediment transport throughout the study reach. Sediment load is becoming increasingly important as indigenous and new fish species are placed in the system. Water quality monitoring with respect to sediment will assist in delineating possible effects that sediment and pollutants that travel sorbed on the sediment may have on the biomass of the system. Knowledge of sediment discharge and its physical characteristics is essential in the analysis of breeding and rearing areas for all aquatic species in the system. Sediment and velocity data collection stations may be established at a few locations where channel geometry is documented as a function of discharge and time. Discharge measurements, sediment data, and cross-sectional data may be collected simultaneously. It is not recommended the stations constantly record but it is suggested that sediment, discharge, and cross-sectional surveys be collected during the annual spring runoff, and at 3-month intervals during the remainder of the year.

Sampling of sediment should include both wash and suspended bed sediment. The bed load can be determined indirectly. Such data are required to provide a better understanding of the dynamics of the river. In addition, samples of bed and bank materials should be collected at each site for comparison with sediment samples collected at the site and with materials obtained at other sites. Also, sediment data provide important information on erosive and transporting forces in the river.

Sediment data can be used to develop water discharge-sediment transport relationships at the sampling sites. These relationships can then be used to help calibrate mathematical models that describe the river system. Such models can be used to analyze all unsteady flow conditions including those associated with the failure of ice jams. The resulting relationships will provide a basis for evaluating anticipated developments along the Connecticut River. In addition, the relationships will better define current conditions. Although not presently considered a very important problem, additional data will assist the power companies in determining the quantity and type of sediment trapped in their reservoirs and help determine the useful life of generating plants and storage facilities. It should be noted that within Wilder Pool, extensive aggradation may be occurring at certain locations during the recession part of the spring runoff hydrograph. This potential alteration of channel geometry within Wilder Pool should be evaluated.

Sediment wash load has a significant effect on the transport of coarser sediments on general channel shape and on water quality. Removal of the wash load by construction of dams or by changing land-use practices adjacent to the river may affect the environment within and along the river. Removal of the wash load may cause an increase in seepage with a reduction in channel bank stability and increased growth of aquatic plants, therefore reducing transport of the bed material. Potential impacts must be understood and utilized in the development of any river that carries a significant wash load. Also, wash load may affect aquatic life, especially fish, within the river system. Before adverse or enhancing impacts can be assessed, the sediment, channel geometry, and discharge data must be known and analyzed at various locations within the river system.

Evaluation of available data indicates the need to initiate a program for periodically obtaining aerial photographs of the upper Connecticut River Valley. Black and white and color infrared photography should be procurred on a three- to five-year time basis. Coverage should include the entire river valley and significant tributaries. Aerial photography provides a means of economically evaluating changes in geometry channel, changes in land-use, and commercial and industrial development that may adversely affect the river environment. In addition, aerial photography can be used in determining potential problem areas where the river appears to be cutting or incising new channels in the alluvial floodplain.

More comprehensive studies should be undertaken to evaluate the effects of boat- and wind-generated water waves on bank stability. The magnitudes and frequencies of these waves do affect river bank stability. Sites where boat waves or wind waves are common should be studied in greater detail in order to better evaluate and cope with these erosion-producing forces.

Other forces acting on the stream banks relate to both natural and man-induced activities including ice movement, pool fluctuations, seepage forces, vegetation, drainage of the land surface, and land-use changes. A better quantitative understanding of the relative magnitude of each of these forces is needed to fully appreciate its contribution to bank erosion. The U.S. Army Corps of Engineers has been conducting a major national-wide effort on stream bank erosion (Corps of Engineers, 1978). Much more information will be available for quantitative evaluation of causes of bank erosion.

In summary, it is not recommended that extensive constant recording, automatic collection,and monitoring equipment be established in the field at the present time. It is recommended that a basic data collection system be designed to further enhance information about river stability. This is particularly important in relation to the hydraulic, geometric, sediment, and aerial photographic data for the system. However, such a network would require significant economic expenditures. To reduce costs only critical data relevant to channel stability should be collected but the program should be designed to be expanded to include systematic collection of additional data that may be required such as water quality parameters as the need develops.

The Connecticut River is an extremely valuable water resource to residents within the tri-state area and it can be anticipated that with increased future development and utilization its importance will increase. To ensure that the river remains in its comparatively stable condition and future development does not endanger this condition, physical bench mark data should be collected to further quantify the existing river conditions.

Within the 141-mile study reach it is recommended that approximately 30 permanent cross-sectional ranges be established to collect the following additional data.

1. Cross-sectional parameters (reduced to elevations above M.S.L.)
2. Velocity and depth measurements
3. Bed and bank material samples
4. Suspended sediment samples
5. Water temperature

The location of the cross sections should be permanently staked on both sides of the river with steel or concrete pins and elevations surveyed in from the closest bench marks. Cross sections may be surveyed from these pins using conventional surveying techniques and sonic equipment.

Following data collection, it is recommended that the following analysis be conducted.

1. Full cross sections should be plotted on a master sheet.
2. Compute the discharge resistance to flow based on velocity and depth measurements.
3. Analyze the bed and bank material samples for size distribution.
4. Compute the amount of total suspended solids and compute the wash load at the section.
5. Compute the total sediment load at the time of sampling using an appropriate total load computational procedure.

Personnel and time estimates to establish and monitor the data collection network are estimated to be approximately one to one and one-half man years of professional service.

LIST OF SYMBOLS

Symbol	
A	cross-sectional area of a particle
C_S	concentration of suspended sediment by weight as a percentage
C_D	coefficient of drag
C_L	coefficient of lift
$C/\sqrt{g}$	dimensionless Chezy resistance to flow coefficient
D	representative fall diameter of the bed material
D_S	particle size
D_v	change in velocity
d	average depth
d	depth of flow at location where shear stress is to be estimated
d_{max}	maximum depth of flow in the cross section
D_{35}, D_{65}	sizes at which sediment in sample is 35 and 65 percent finer
e_1	distance defined in Figure 74 (p. 125)
e_2	distance defined in Figure 74 (p. 125)
e_3	distance defined in Figure 74 (p. 125)
e_4	distance defined in Figure 74 (p. 125)
F	fetch length
F_D	fluid lift--drag force
f_s	seepage forces in the bed of the stream
f	parameter
F_ℓ	fluid lift--lift force
g	gravitational acceleration
H	height of wave from crest to trough
L	length of the dike
L_S	length of dike requiring shank protection

Symbol

M	moment ratio in riprap stability analysis
N	moment ratio in riprap stability analysis
n	resistance to flow
O	a contact point where rotation must occur
R	hydraulic radius
Re	particle Reynolds number
r_c	radius of curvature to the centerline of the bend
S	slope of energy gradient
S	shear stress
S_E	slope of the energy grade line
S_p	shape factors of the particles
S_r	shape factor for the reach of the stream
S_c	shape factor for the cross section of the stream
S_s	specific gravity of the riprap
S_m	$\frac{\tan\phi}{\tan\theta}$
U	wind speed
u_1,u_2	point velocities measured at distances y_1 and y_2 from the boundary of the channel
U_*	shear velocity
U	average velocity of flow
V	magnitude of the velocity leaving the dike field at the angle ψ
V_S	velocity at the shank
W_s	weight of the particle
W	weight
W	channel width
w	variable

Symbol

X	horizontal distance from the centerline of the channel to the point of maximum velocity
α	the angle that the stream bed makes with a horizontal line
β	angle between the downslope force and the resultant force on a particle on the sideslope of a channel
γ	specific weight of the water
γ_s	specific weight of sediment
Δ	angle of bend
Δ_γ	difference of specific weight of sediment and water
Δv	change in velocity
η'	stability number
θ	angle of side slope of the bank
θ_s	shear stress on particles with size D_{50}
ρ	density of water
ρ_s	density of sediment-water mixture
λ	angle between the horizontal and the velocity vector in the plane of the side slope
τ	stress/tractive force
τ_c	critical shear stress
τ_s	shear stress acting on the bank of a channel
τ_b	shear stress acting on the bed of a channel
σ	measure of the size distribution of the bed material
ϕ	angle of repose of bank material
μ	dynamic viscosity of the water
ν	kinematic viscosity

GLOSSARY

BED LOAD - sediment that moves by saltation (jumping), rolling or sliding in the bed layer of a stream.

CRITICAL SHEAR STRESS - the minimum amount of shear stress exerted on a sediment particle that causes the particle to begin to move.

ENERGY SLOPE - the slope of a line representing the total band or energy possessed by a river. For open channel flow the energy slope is located a distance of $U^2/2g$ above the water surface (U = velocity).

FALL DIAMETER - is the diameter of a sphere that has a specific gravity of 2.65 and the same terminal uniform silting velocity as a sediment particle when each is allowed to settle alone in quiescent, distilled water of infinite extent at a temperature of 24°C.

FLOOD FREQUENCY - the relationship between flood magnitude and the period of time expected before a given flood magnitude may occur again.

FLOW REGIMES - the state of flow and bedform at which a stream is flowing, e.g., ripples, dunes, plane bed, standing waves, antidunes, chutes and pools.

HEADCUT - upslope progression of a channel caused by flowing water and mass wasting of channel banks. Common where large changes in base level have created rapid downcutting.

HYDROGRAPH - a graph of the discharge or depth of water flowing by a particular point versus time.

MASS WASTING - downslope movement of earth and vegetative materials under the force of gravity. There are many types of mass wasting including landslides, sloughing and mudflows.

NATURAL RIVER - a river that is usually unaffected by backwater curves caused by dams and other hydraulic structures.

SHEAR STRESS - the force exerted by flowing water on the bed of a river.

SEIVE ANALYSIS - a method of determining the size distribution of a sediment sample.

SHAPE FACTOR -

Sediment Particles - a ratio of lengths of a sediment particle along three axes. $S = c/\sqrt{ab}$ where a, b and c are the 3 lengths, c being the shortest.

Stream Reach - a factor affecting energy losses through a reach.

Cross-section - a factor affecting variation in velocity, width, depth and boundary shear stress.

SLIDE - movement involving failure along discrete planes of weakness such as a rotational slump.

SLOUGHING - mass wasting involving small amounts of material moving as clumps on individual particles.

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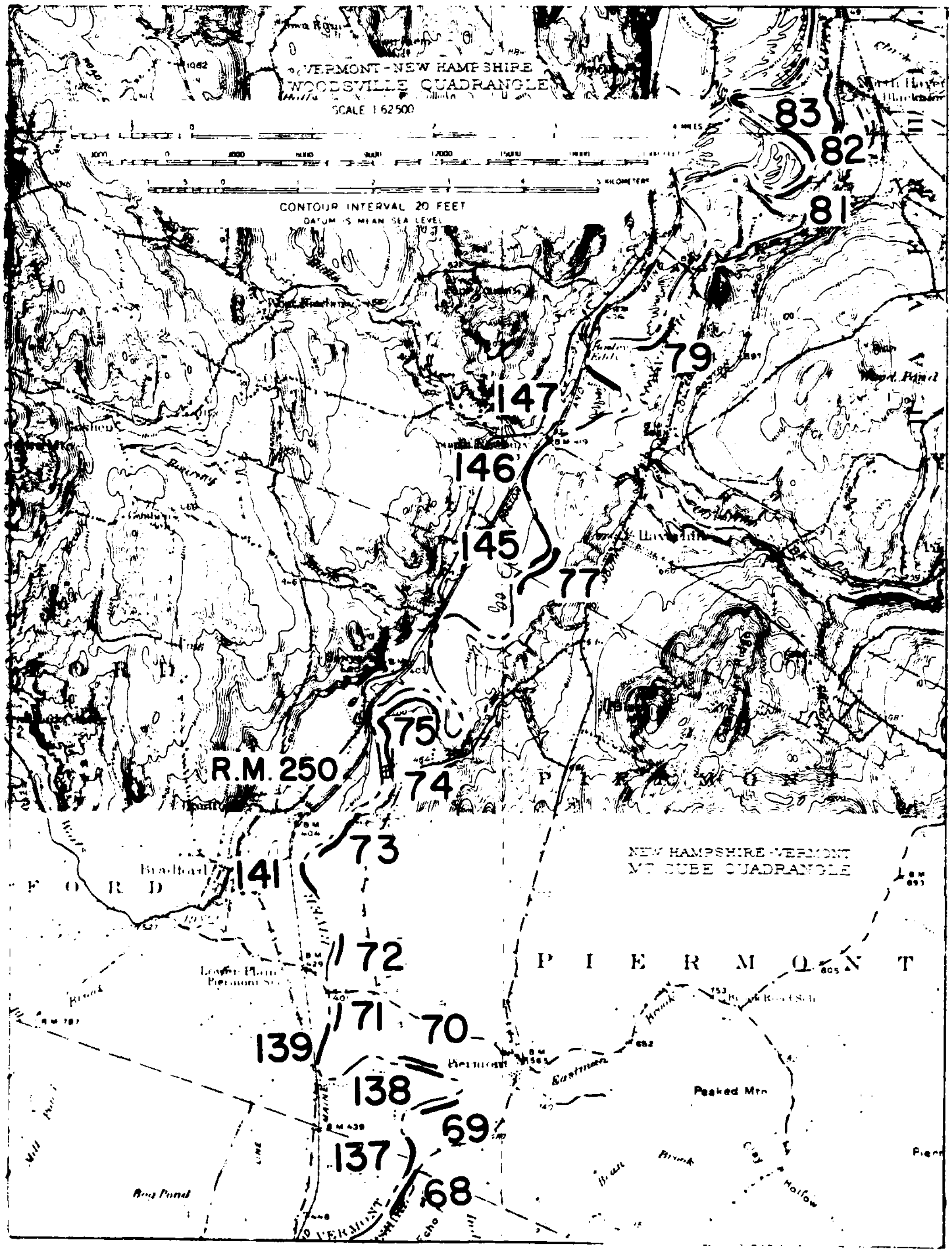
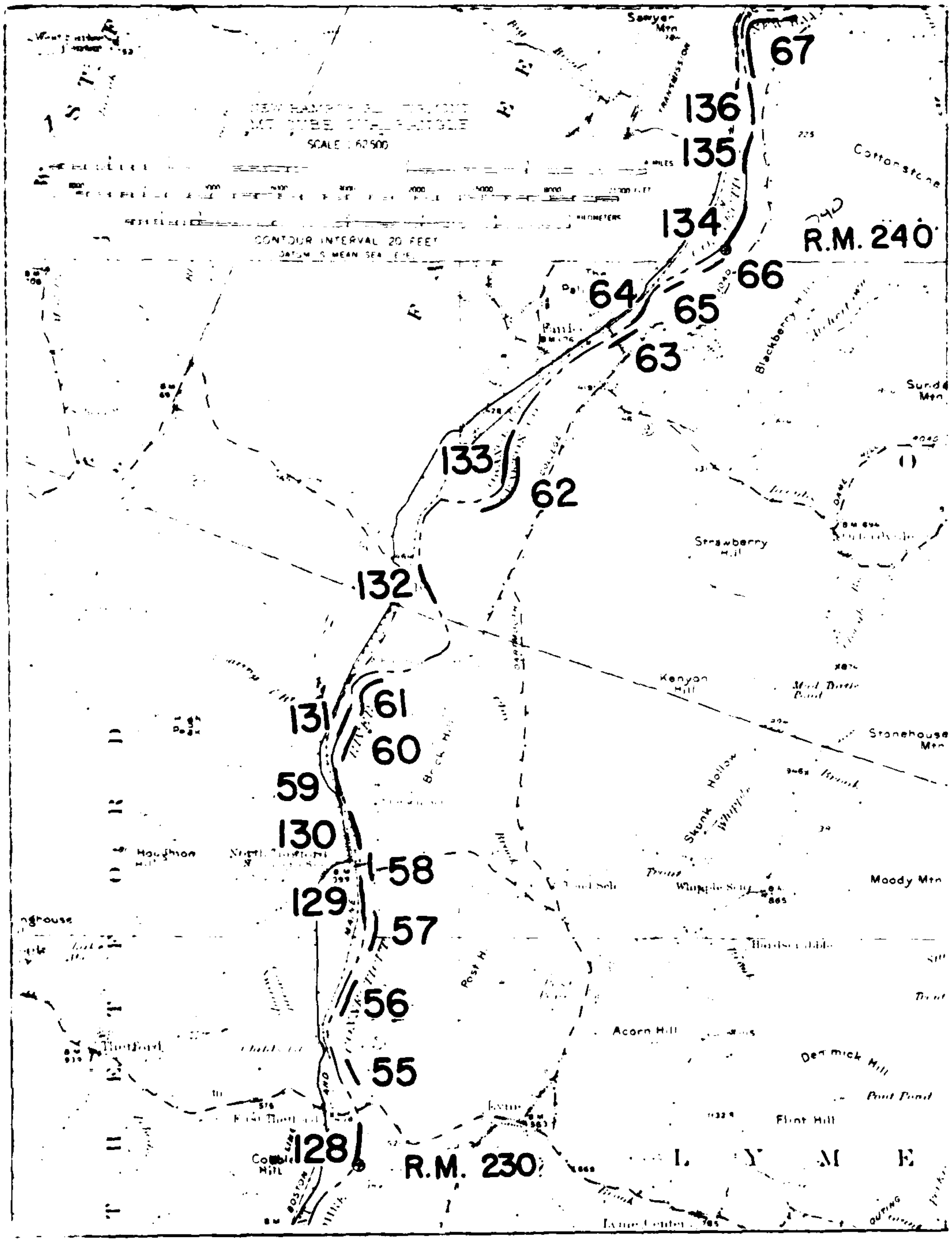
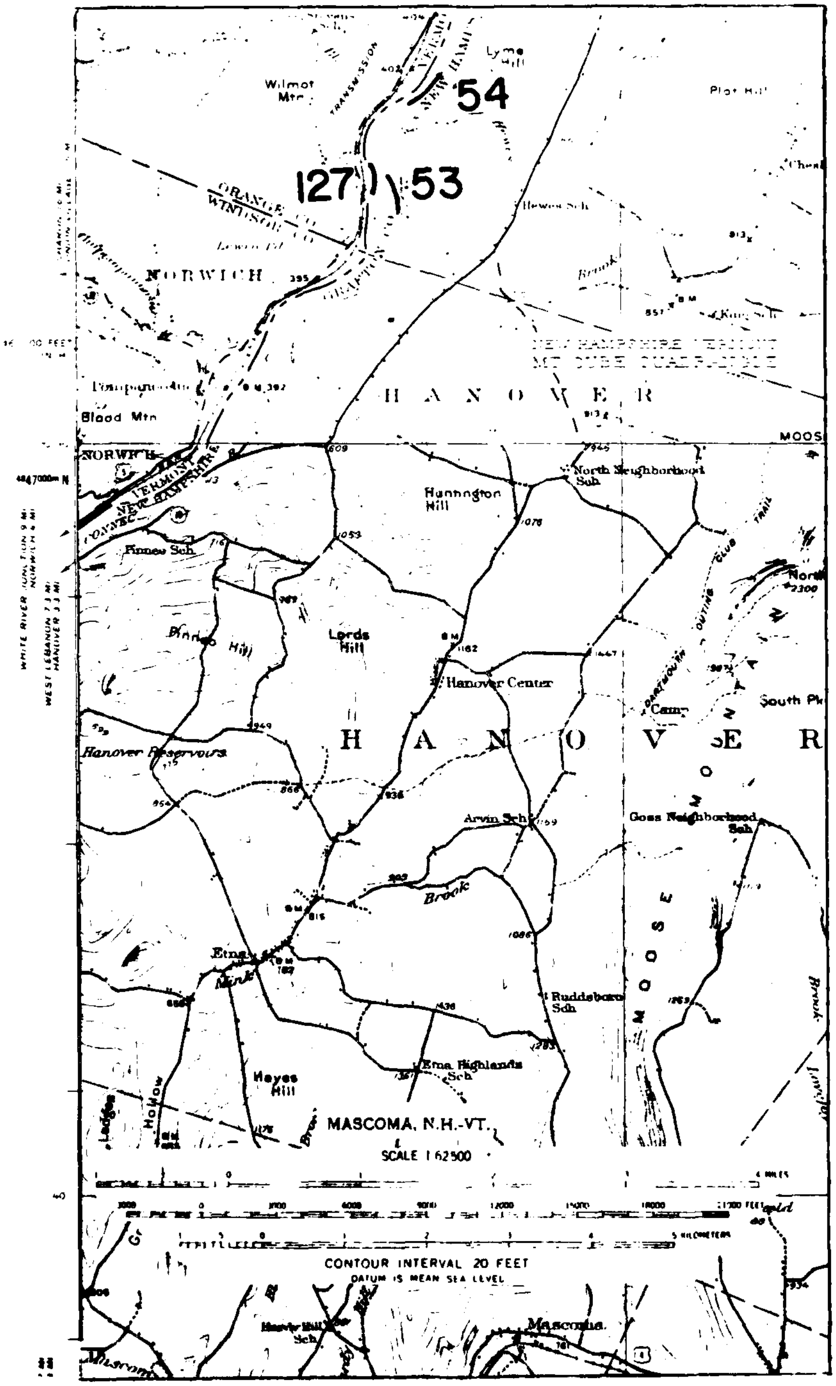
APPENDIX A

LOCATIONS OF EROSION SITES IN THE STUDY REACH

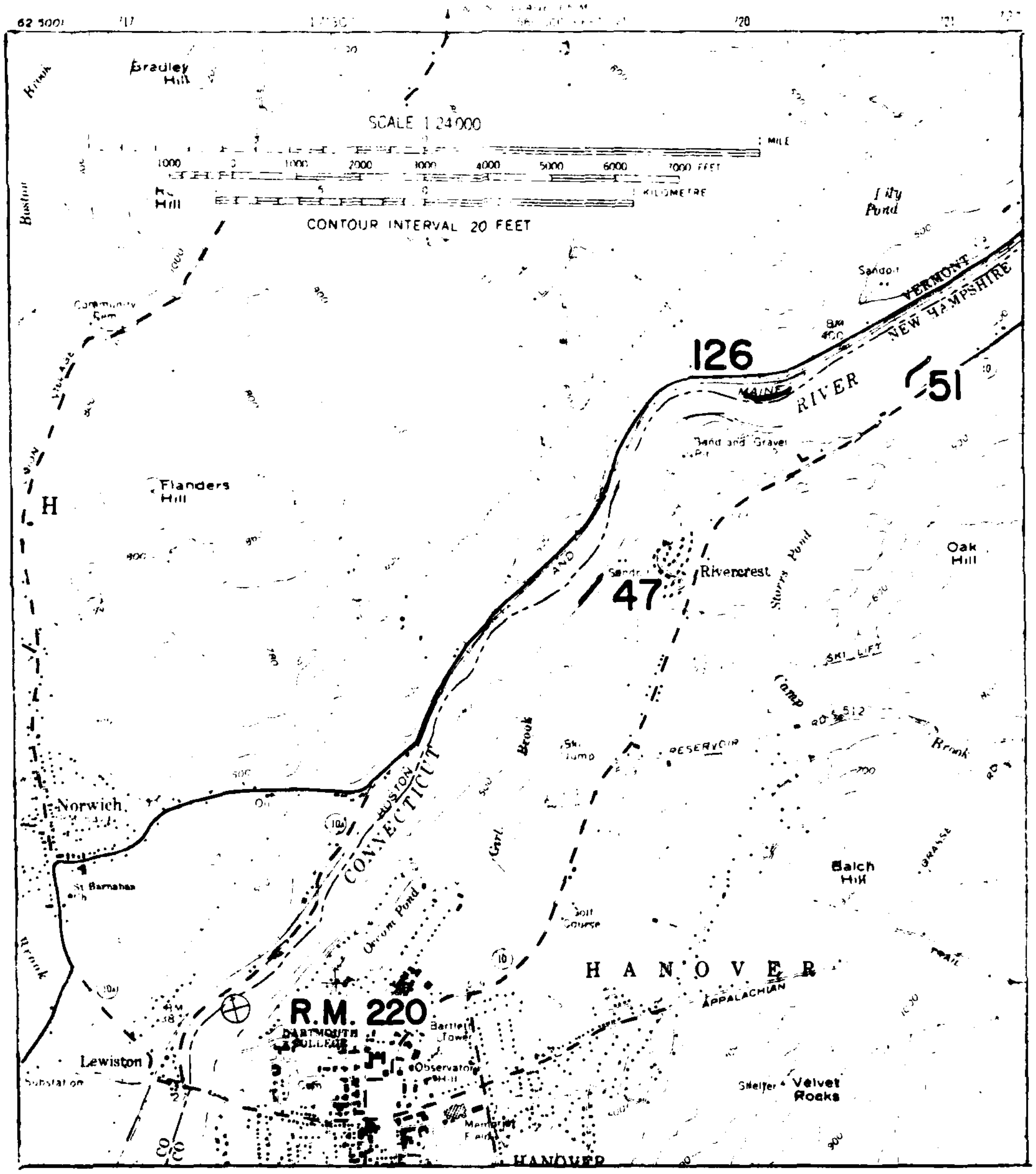

VERMONT-NEW HAMPSHIRE
WOODSVILLE QUADRANGLE
SCALE 1:62500
CONTOUR INTERVAL 20 FEET
DATUM IS MEAN SEA LEVEL
R.M. 260

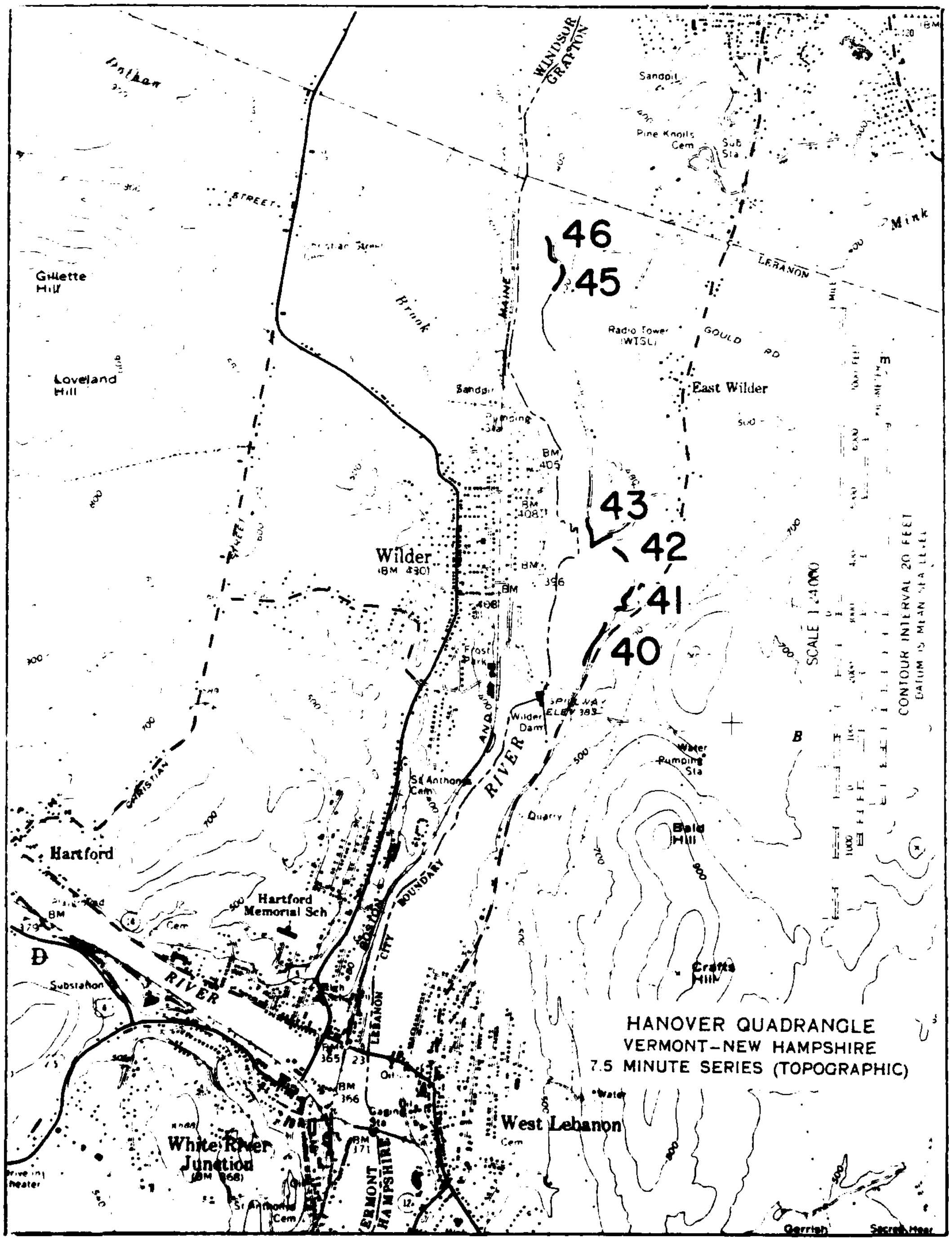

VERMONT - NEW HAMPSHIRE
WOODSVILLE QUADRANGLE
SCALE 1:62500
CONTOUR INTERVAL 20 FEET
DATUM IS MEAN SEA LEVEL
83
82
81
79
147
146
145
77
75
R.M. 250
74
73
141
72
71
70
139
138
69
137
68
NEW HAMPSHIRE - VERMONT
MT CUBE QUADRANGLE
P I E R M O N T
Peaked Mtn
Bog Pond

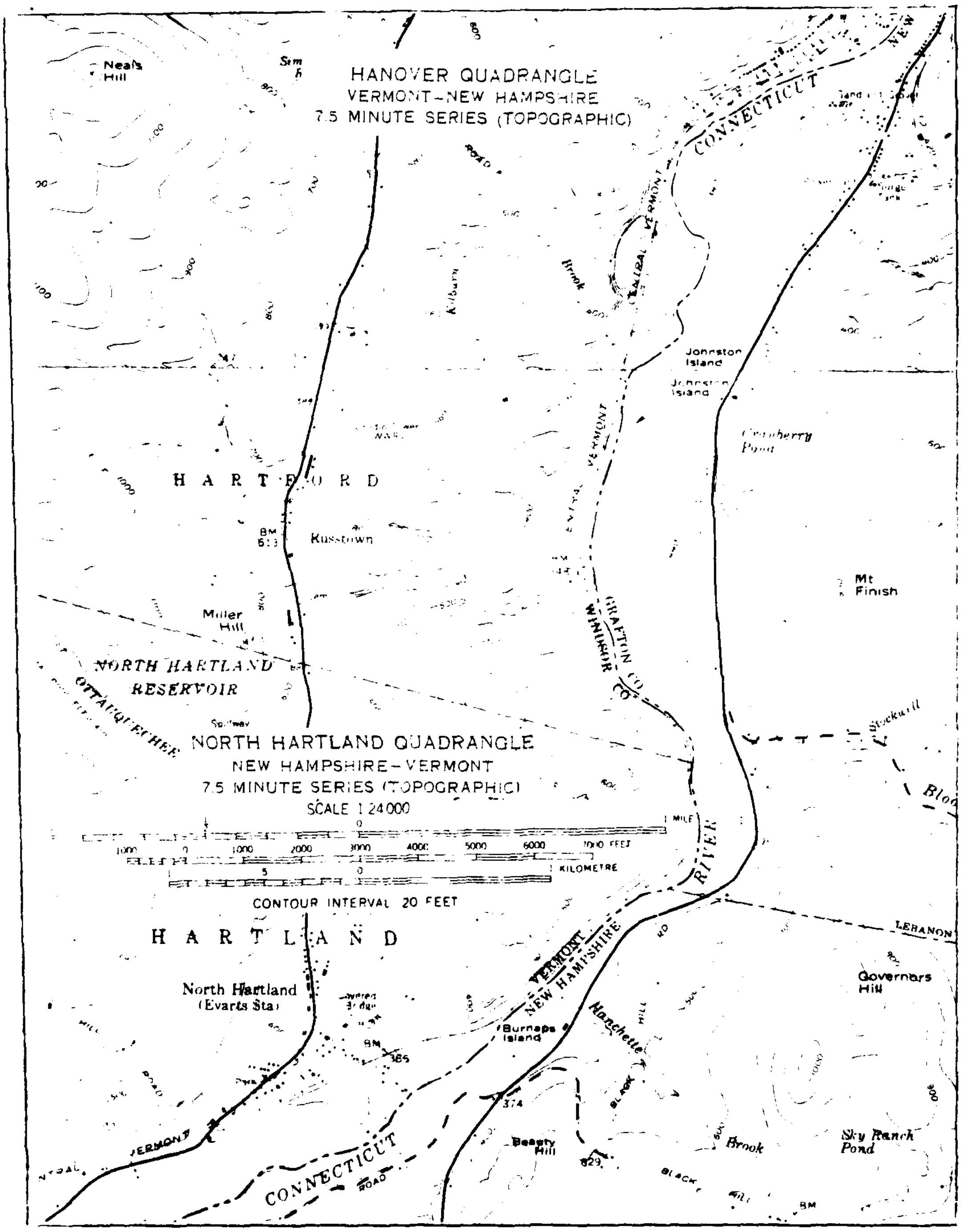

67
136
135
134
R.M. 240
66
65
64
63
133
62
132
131
61
60
59
130
58
129
57
56
55
128
R.M. 230
SCALE 1:62500
CONTOUR INTERVAL 20 FEET
Sawyer Mtn
Cottonstone
Blackberry Hill
Strawberry Hill
Kenyon Hill
Stonehouse Mtn
Skunk Hollow
Moody Mtn
Houghton Hill
Post H
Acorn Hill
Flint Hill
Post Pond
Cobble Hill
L Y M E


Wilmot Mtn
Lyme Hill
54
127
53
TRANSMISSION
ORANGE CO
WINDSOR CO
NORWICH
Hewes Sch
Blood Mtn
HANOVER
NEW HAMPSHIRE VERMONT
MT CUBE QUADRANGLE
Huntington Hill
North Neighborhood Sch
Lords Hill
Hanover Center
Hanover Reservoirs
HANOVER
Arvin Sch
Goss Neighborhood Sch
MOOSE MOUNTAIN
DARTMOUTH OUTING CLUB TRAIL
South Peak
Camp
Brook
Etna
Mink
Ruddsboro Sch
Etna Highlands Sch
Hayes Hill
Hollow
MASCOMA, N.H.-VT.
SCALE 1:62500
CONTOUR INTERVAL 20 FEET
DATUM IS MEAN SEA LEVEL
Mascoma
WHITE RIVER JUNCTION 9 MI
NORWICH 4 MI
WEST LEBANON 7.3 MI
HANOVER 3.3 MI

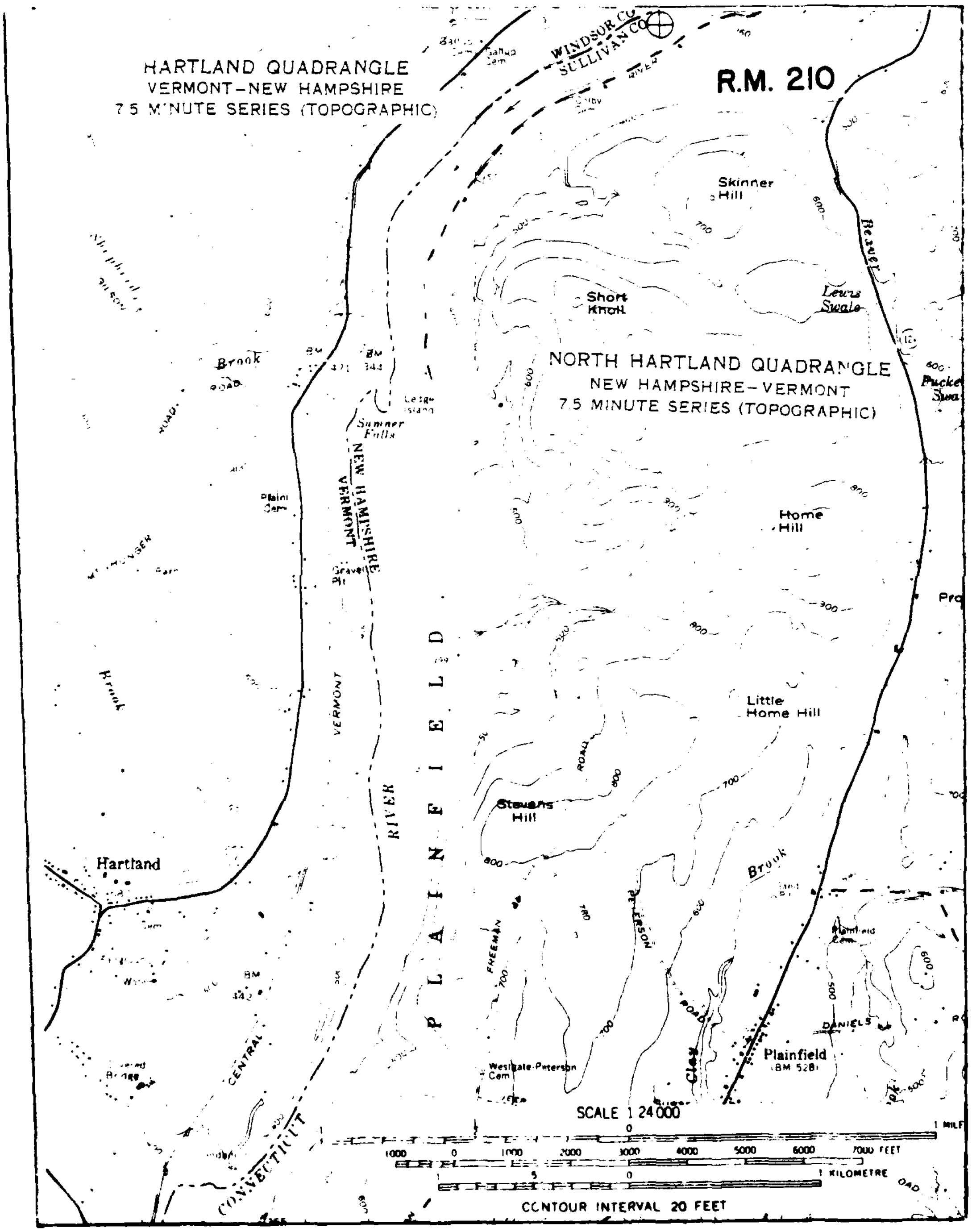
HANOVER QUADRANGLE
VERMONT-NEW HAMPSHIRE
7.5 MINUTE SERIES (TOPOGRAPHIC)

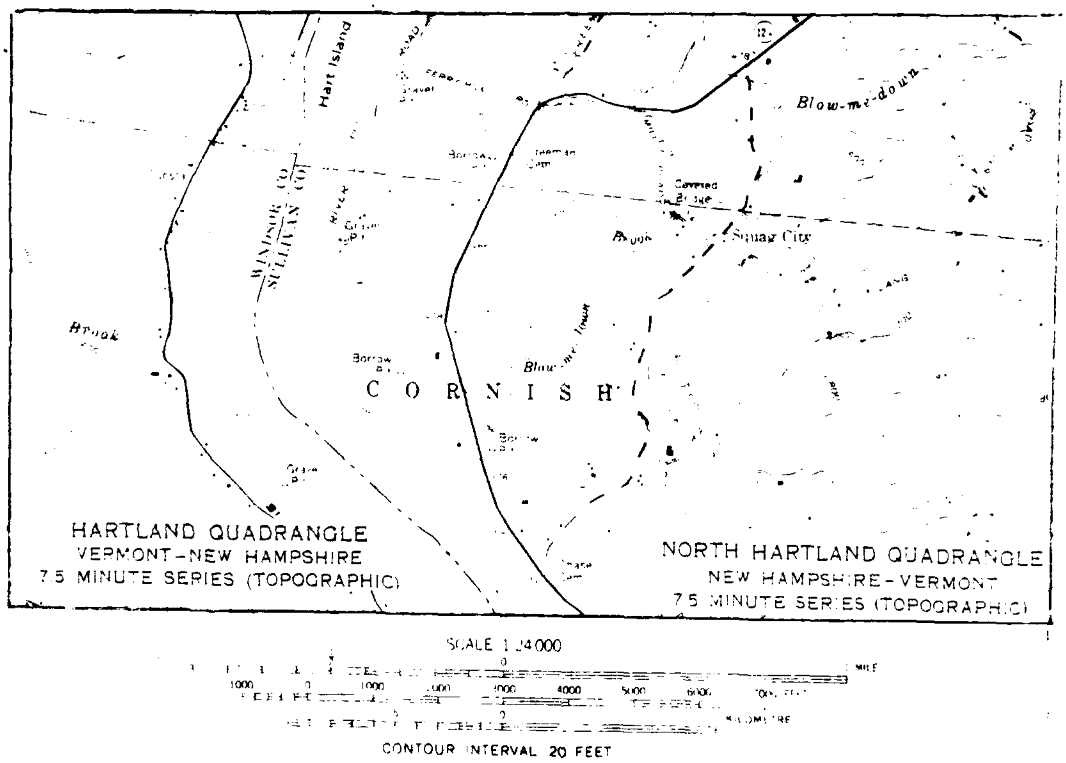


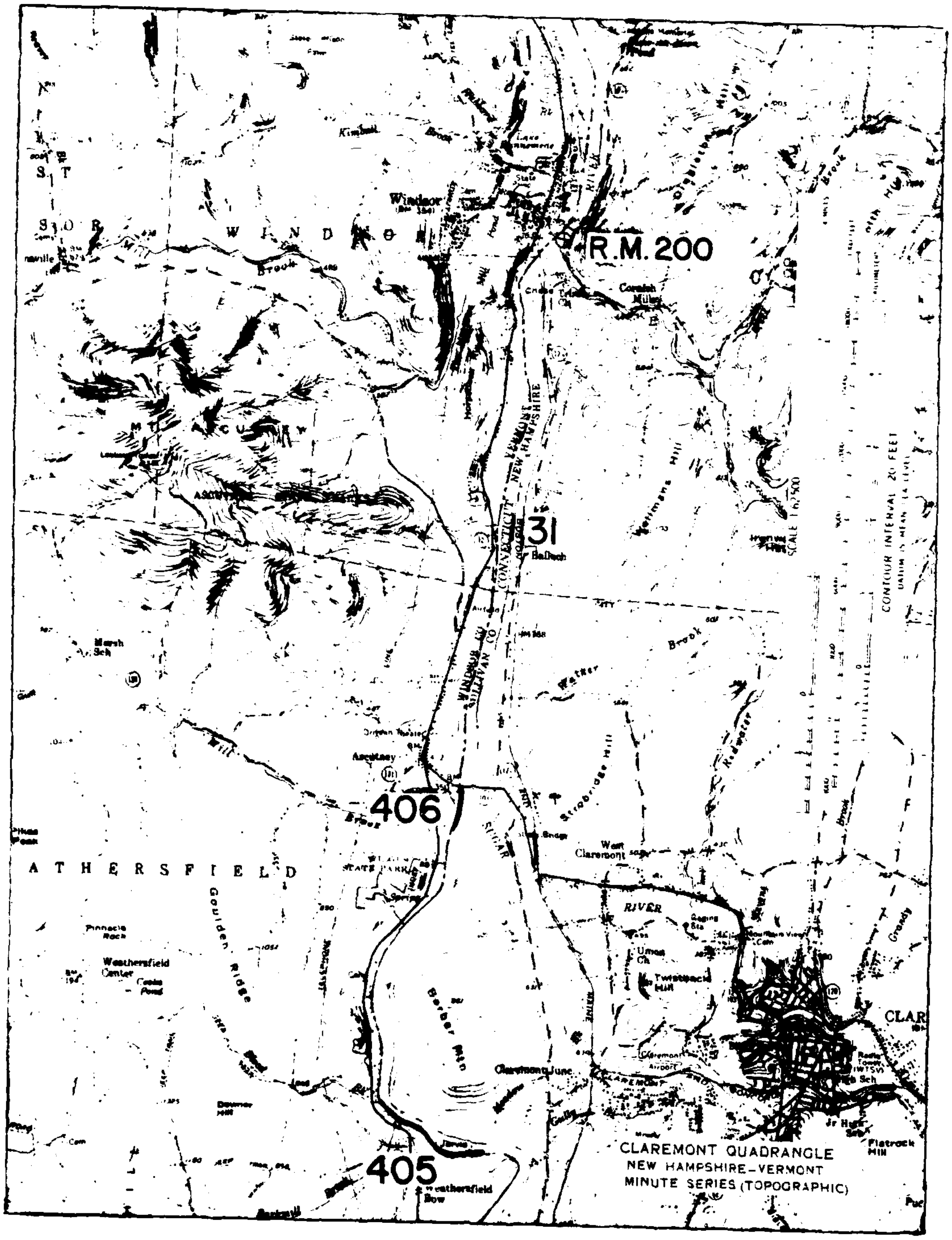

46
45
43
42
41
40
Gillette Hill
Loveland Hill
Wilder
East Wilder
Radio Tower (WTSL)
Pine Knolls Cem
Hartford
Hartford Memorial Sch
White River Junction
West Lebanon
Bald Hill
Crafts Hill
Wilder Dam
Water Pumping Sta
RIVER
VERMONT
NEW HAMPSHIRE
HANOVER QUADRANGLE
VERMONT–NEW HAMPSHIRE
7.5 MINUTE SERIES (TOPOGRAPHIC)
CONTOUR INTERVAL 20 FEET

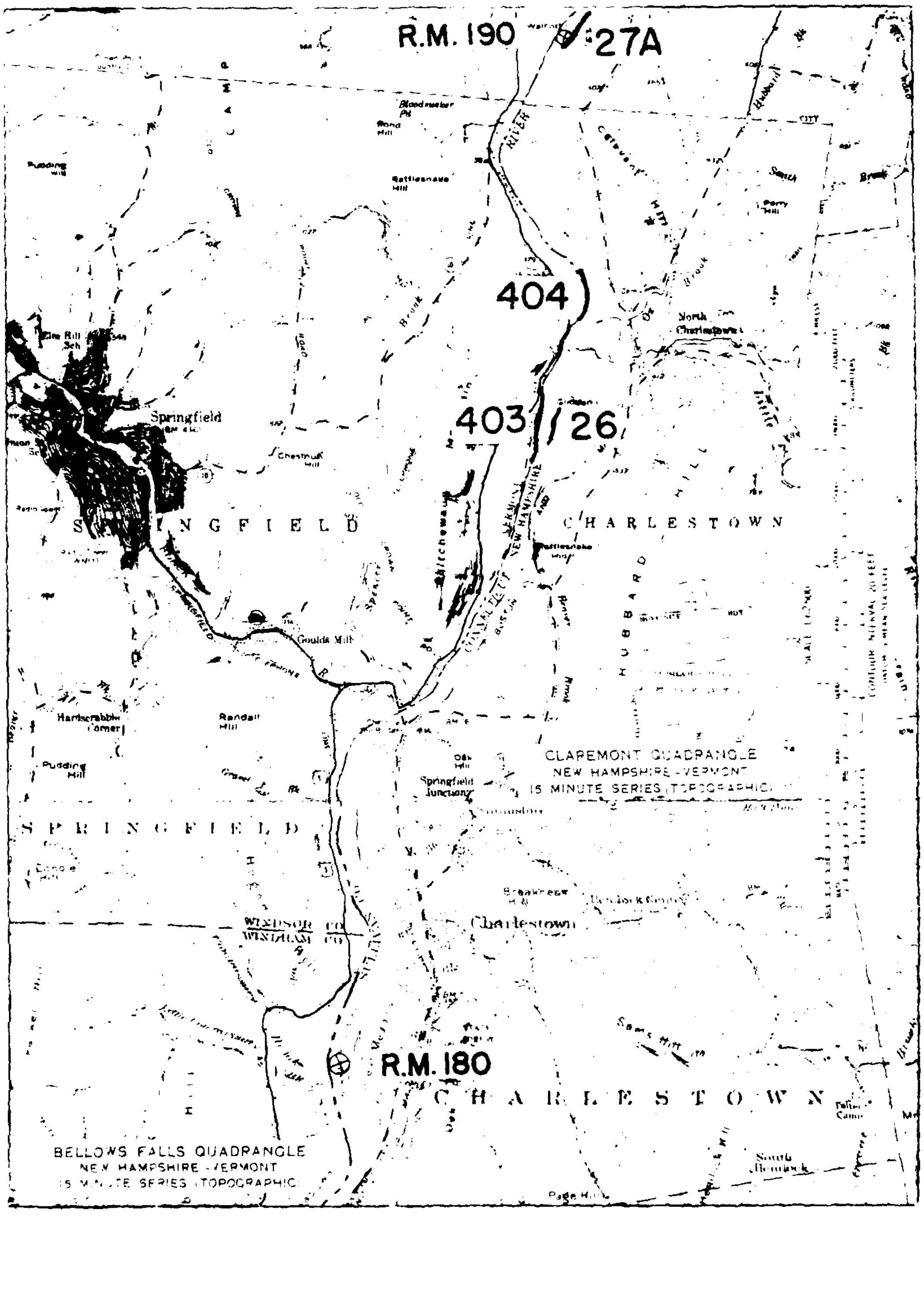


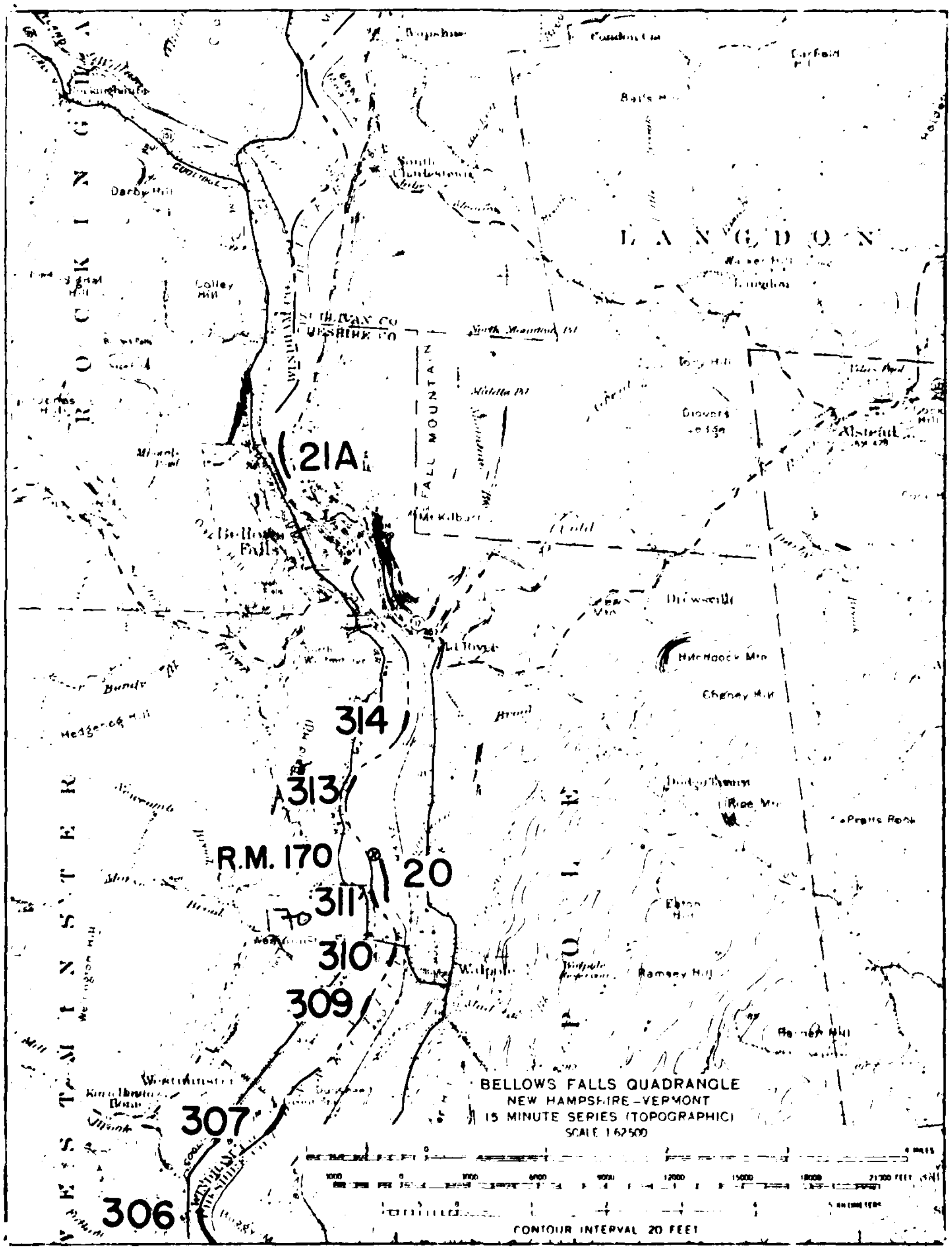
HANOVER QUADRANGLE
VERMONT–NEW HAMPSHIRE
7.5 MINUTE SERIES (TOPOGRAPHIC)
Neals Hill
CONNECTICUT
Johnston Island
Cranberry Pond
HARTFORD
Russtown
Mt Finish
Miller Hill
NORTH HARTLAND RESERVOIR
OTTAUQUECHEE
Spillway
NORTH HARTLAND QUADRANGLE
NEW HAMPSHIRE–VERMONT
7.5 MINUTE SERIES (TOPOGRAPHIC)
SCALE 1:24000
1 MILE
7000 FEET
1 KILOMETRE
CONTOUR INTERVAL 20 FEET
HARTLAND
North Hartland (Evarts Sta)
RIVER
LEBANON
Governors Hill
Burnaps Island
Hanchette
Beasty Hill
Brook
Sky Ranch Pond
VERMONT
NEW HAMPSHIRE
CONNECTICUT

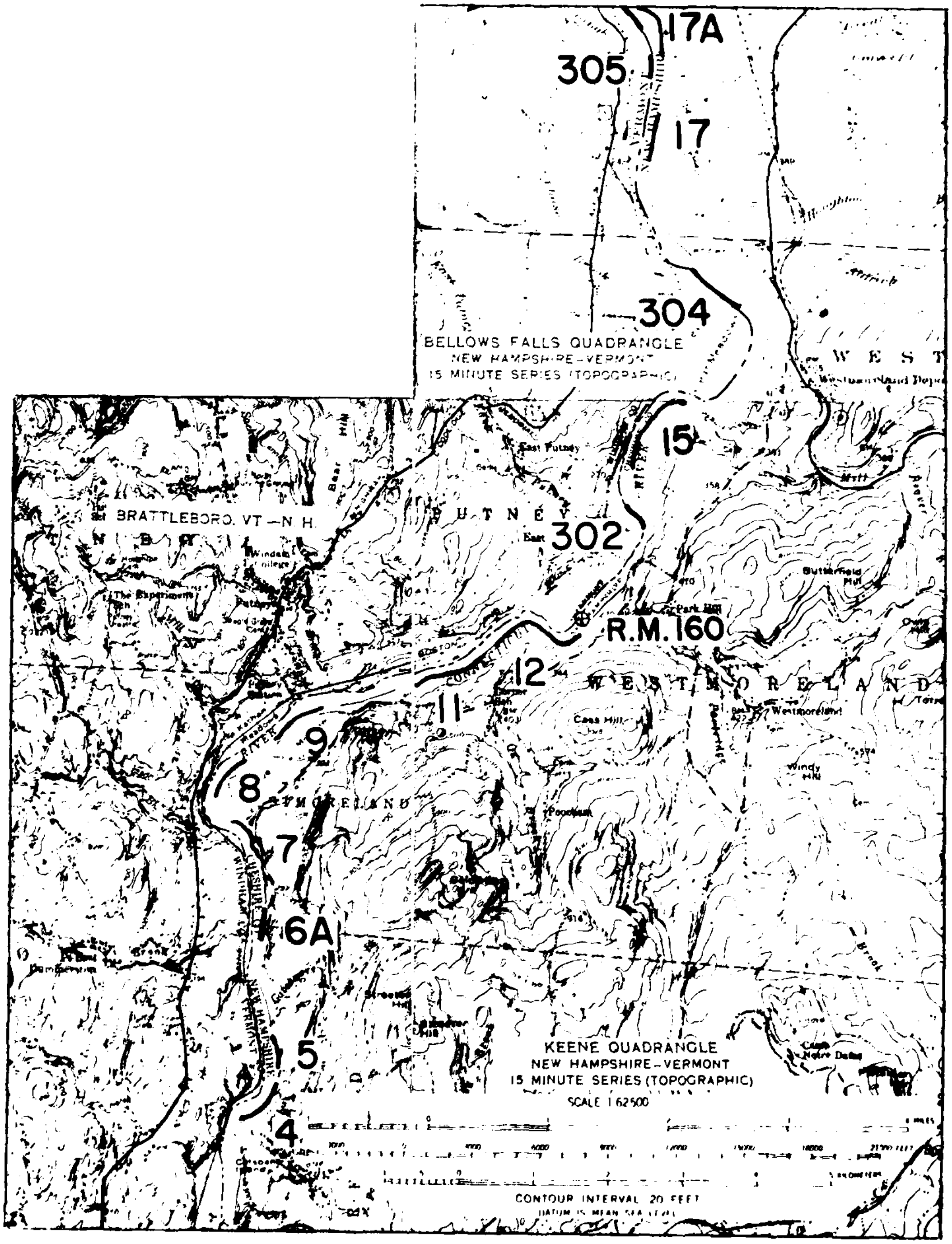

HARTLAND QUADRANGLE
VERMONT-NEW HAMPSHIRE
7.5 MINUTE SERIES (TOPOGRAPHIC)
R.M. 210
WINDSOR CO
SULLIVAN CO
NORTH HARTLAND QUADRANGLE
NEW HAMPSHIRE-VERMONT
7.5 MINUTE SERIES (TOPOGRAPHIC)
Skinner Hill
Short Knoll
Lewis Swale
Home Hill
Little Home Hill
Stevens Hill
Sumner Falls
NEW HAMPSHIRE
VERMONT
RIVER
PLAINFIELD
Hartland
Brook
Plainfield
(BM 528)
Westgate-Peterson Cem
CONNECTICUT
SCALE 1:24 000
1 MILE
7000 FEET
1 KILOMETRE
CONTOUR INTERVAL 20 FEET


Hart Island
WINDSOR CO
SULLIVAN CO
Blow-me-down
Covered Bridge
Squag City
Brook
C O R N I S H
HARTLAND QUADRANGLE
VERMONT-NEW HAMPSHIRE
7.5 MINUTE SERIES (TOPOGRAPHIC)
NORTH HARTLAND QUADRANGLE
NEW HAMPSHIRE-VERMONT
7.5 MINUTE SERIES (TOPOGRAPHIC)
SCALE 1:24 000
CONTOUR INTERVAL 20 FEET

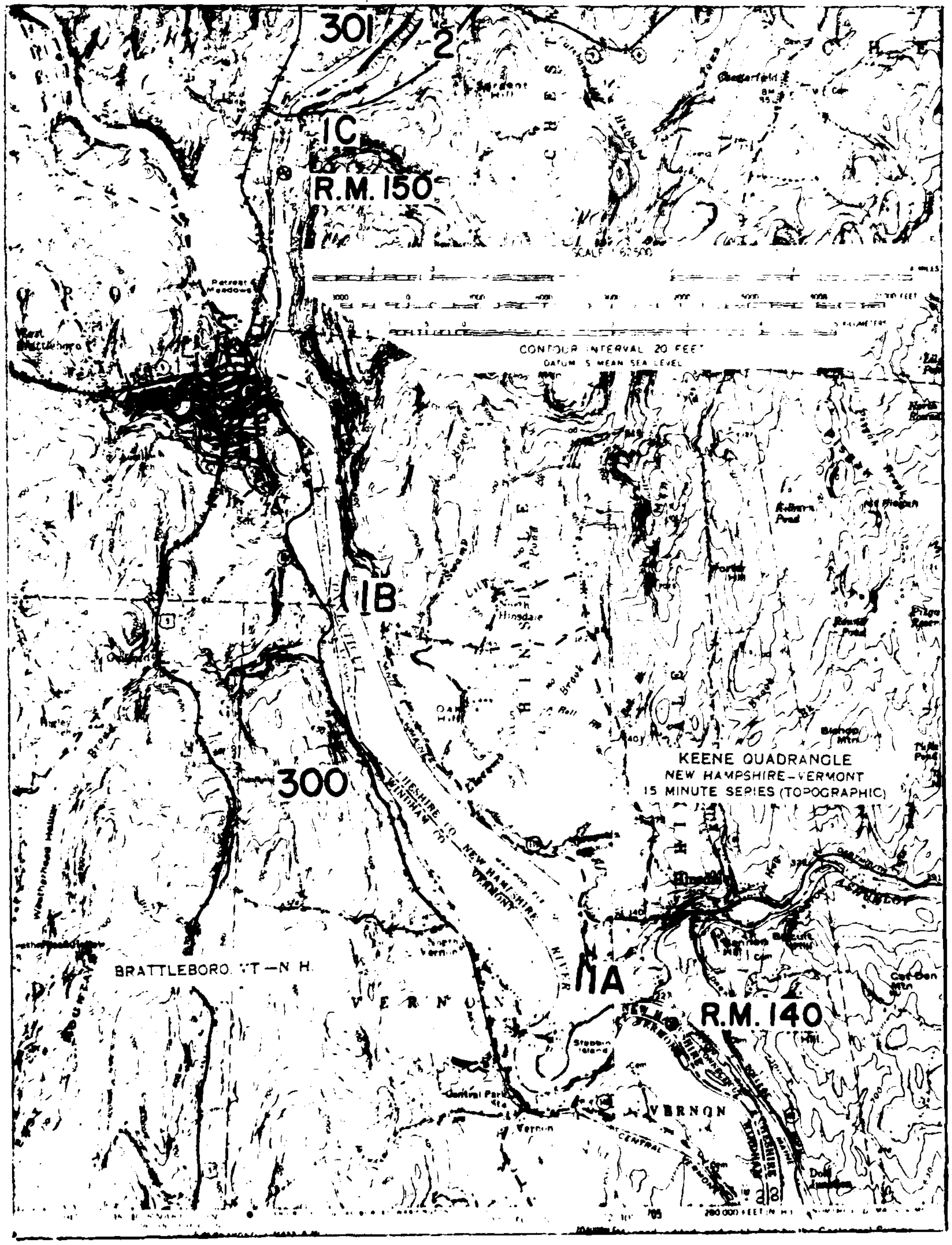

R.M. 200
131
406
405
WINDSOR
WEATHERSFIELD
Windsor
Cornish Mills
Ascutney
Marsh Sch
Weathersfield Center
Pinnacle Rock
Goulden Ridge
Barber Mtn
West Claremont
Claremont Junc
Twistback Hill
Weathersfield Bow
Flatrock Hill
CONNECTICUT
VERMONT
NEW HAMPSHIRE
WINDSOR CO
SULLIVAN CO
SUGAR
RIVER
CONTOUR INTERVAL 20 FEET
CLAREMONT QUADRANGLE
NEW HAMPSHIRE-VERMONT
MINUTE SERIES (TOPOGRAPHIC)

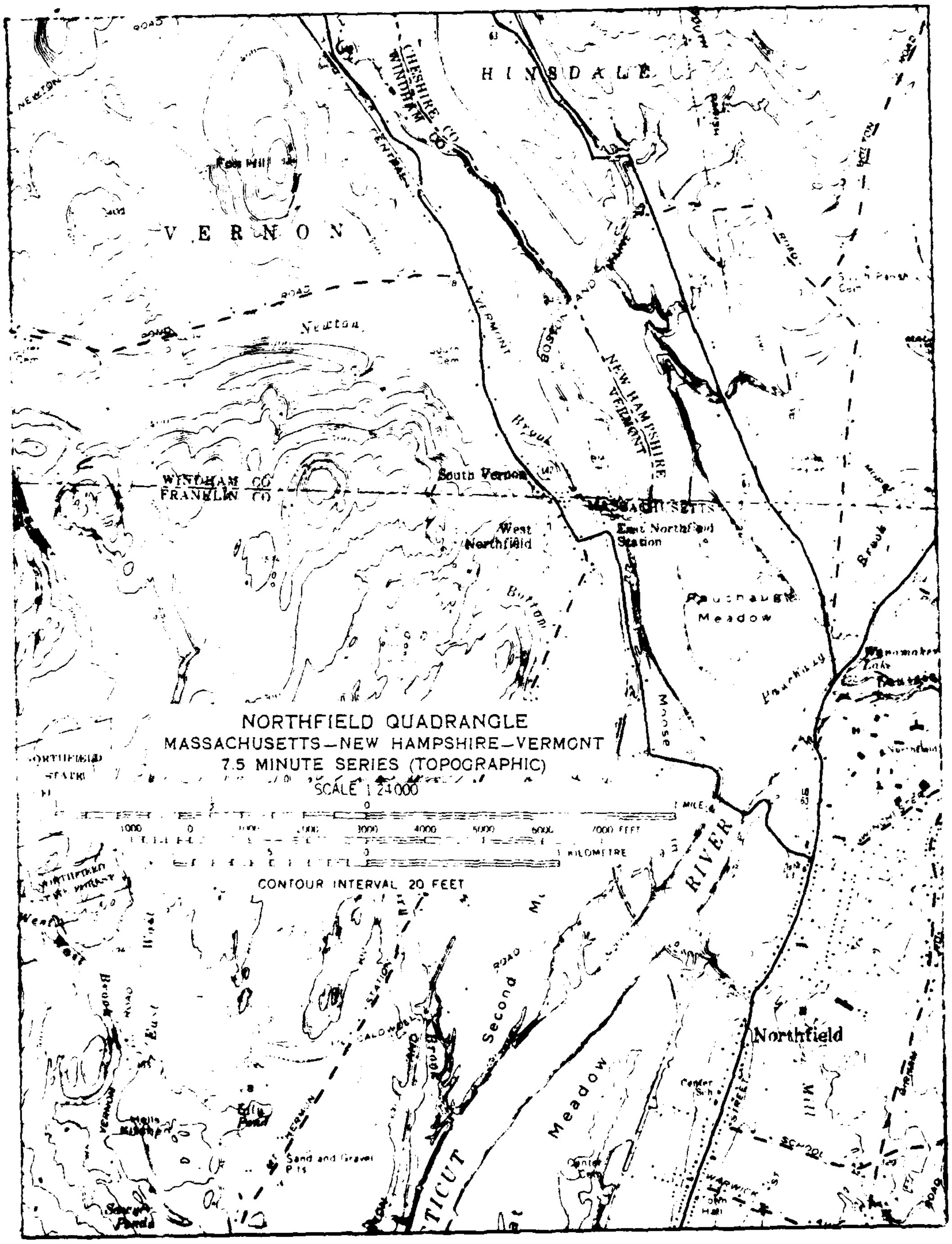

R.M. 190
27A
404
403
26
Springfield
SPRINGFIELD
CHARLESTOWN
CLAREMONT QUADRANGLE
NEW HAMPSHIRE - VERMONT
15 MINUTE SERIES (TOPOGRAPHIC)
SPRINGFIELD
Charlestown
R.M. 180
CHARLESTOWN
BELLOWS FALLS QUADRANGLE
NEW HAMPSHIRE - VERMONT
15 MINUTE SERIES (TOPOGRAPHIC)

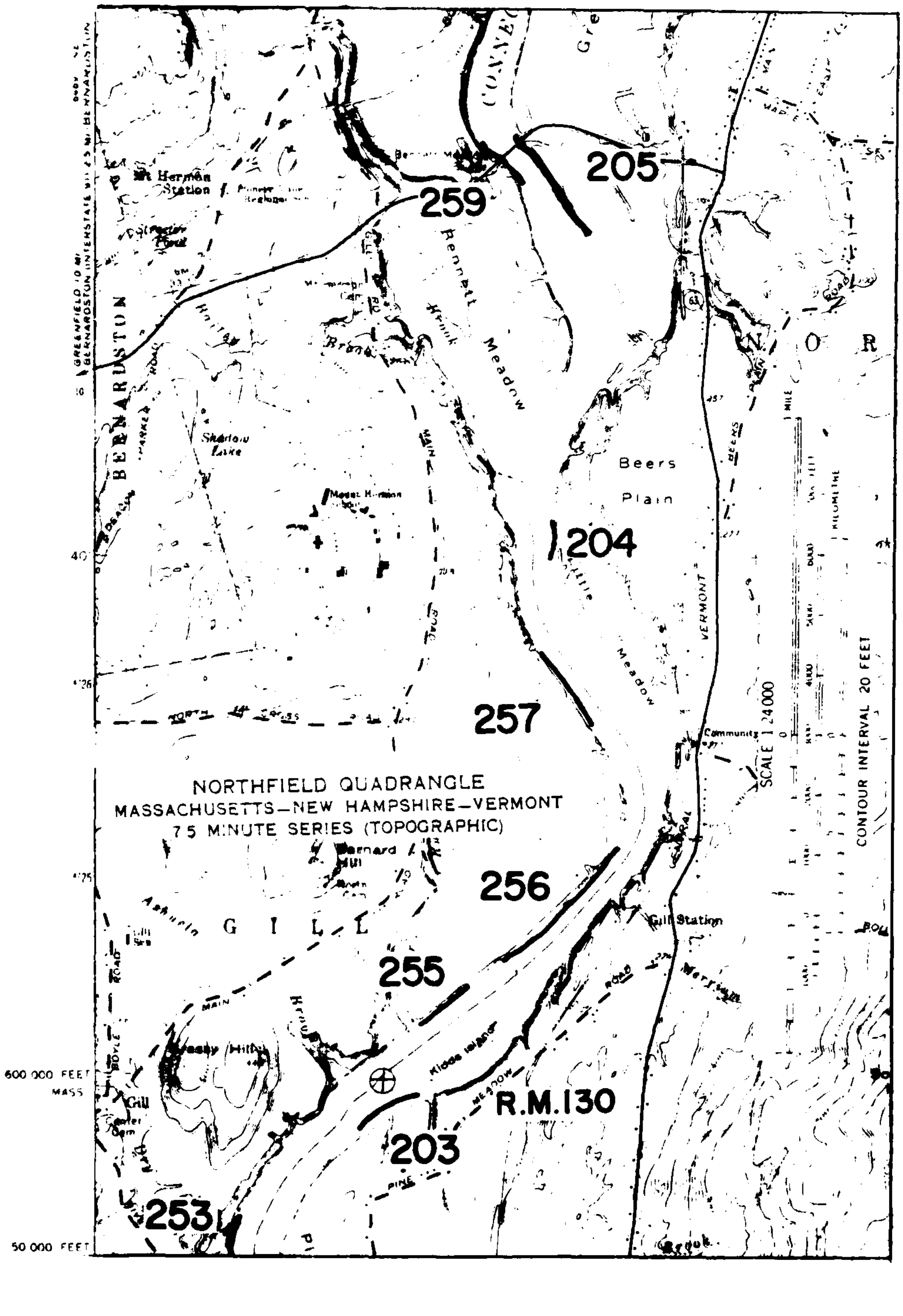

21A
314
313
R.M. 170
20
311
310
309
307
306
BELLOWS FALLS QUADRANGLE
NEW HAMPSHIRE-VERMONT
15 MINUTE SERIES (TOPOGRAPHIC)
SCALE 1:62500
CONTOUR INTERVAL 20 FEET

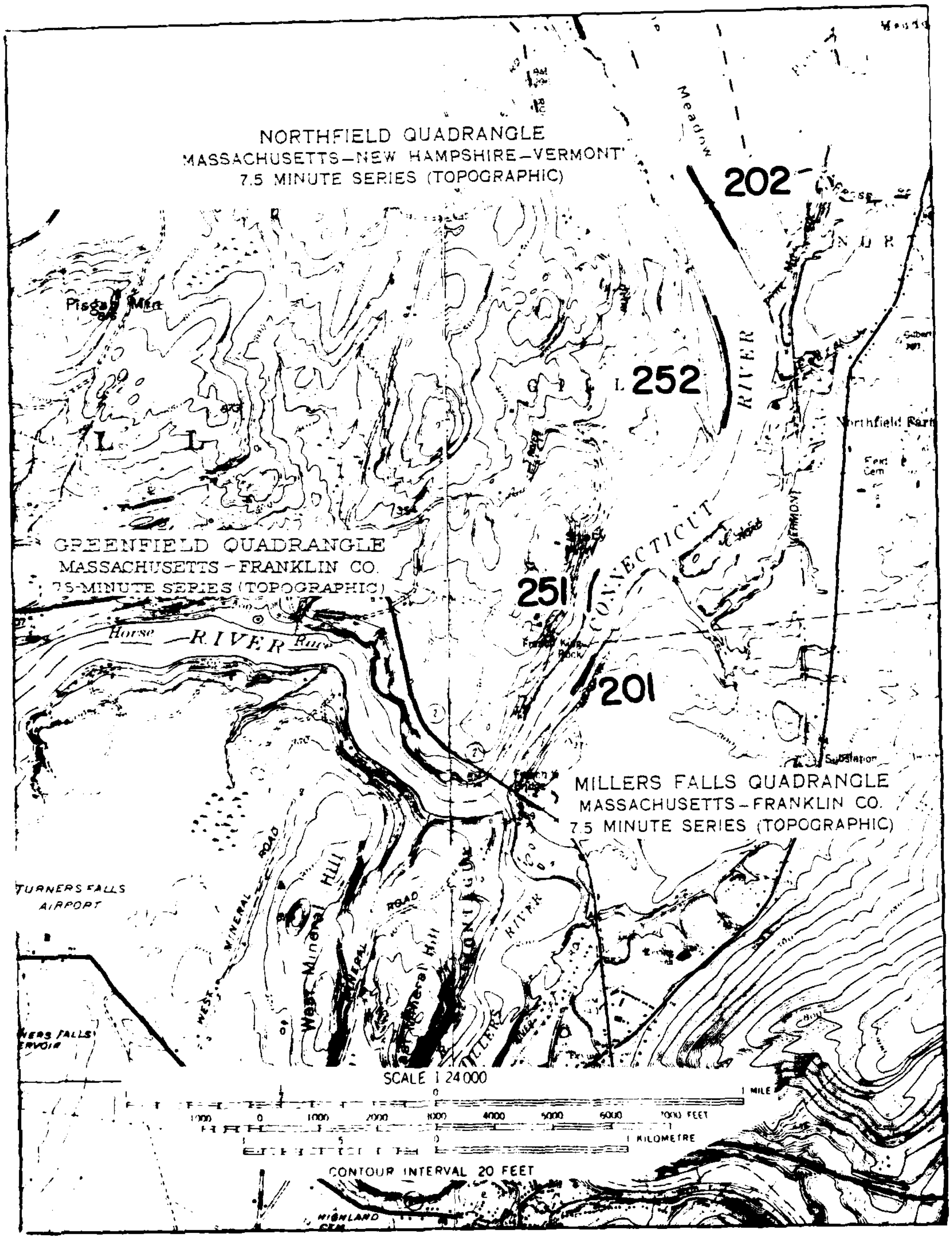


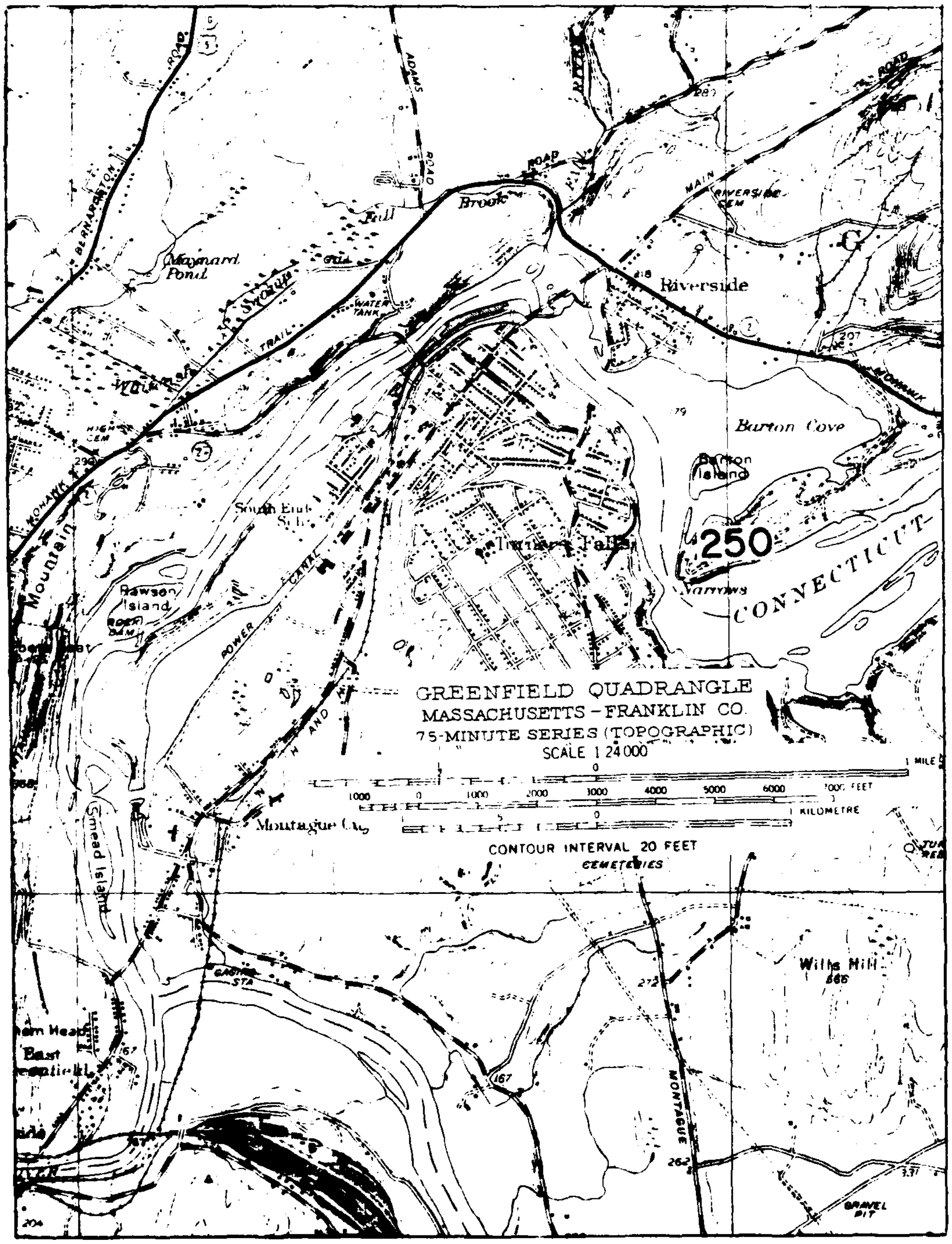
17A
305
17
304
BELLOWS FALLS QUADRANGLE
NEW HAMPSHIRE-VERMONT
15 MINUTE SERIES (TOPOGRAPHIC)
15
BRATTLEBORO, VT.—N.H.
302
R.M.160
12
11
9
8
7
6A
5
4
KEENE QUADRANGLE
NEW HAMPSHIRE-VERMONT
15 MINUTE SERIES (TOPOGRAPHIC)
SCALE 1:62500
CONTOUR INTERVAL 20 FEET


301
2
1C
R.M. 150
CONTOUR INTERVAL 20 FEET
1B
300
KEENE QUADRANGLE
NEW HAMPSHIRE-VERMONT
15 MINUTE SERIES (TOPOGRAPHIC)
BRATTLEBORO, VT—N.H.
1A
R.M. 140
VERNON


HINSDALE
VERNON
CHESHIRE CO
WINDHAM CO
NEW HAMPSHIRE
VERMONT
WINDHAM CO
FRANKLIN CO
South Vernon
MASSACHUSETTS
West Northfield
East Northfield Station
Pauchaug Meadow
Northfield
RIVER
Second
Meadow
Sand and Gravel Pits
NORTHFIELD QUADRANGLE
MASSACHUSETTS—NEW HAMPSHIRE—VERMONT
7.5 MINUTE SERIES (TOPOGRAPHIC)
SCALE 1:24000
CONTOUR INTERVAL 20 FEET


NORTHFIELD QUADRANGLE
MASSACHUSETTS–NEW HAMPSHIRE–VERMONT
7.5 MINUTE SERIES (TOPOGRAPHIC)
259
205
204
257
256
255
203
253
R.M.130
Beers
Plain
Bennett
Meadow
Gill Station
BERNARDSTON
GILL
SCALE 1:24 000
CONTOUR INTERVAL 20 FEET
VERMONT
600 000 FEET
MASS
50 000 FEET


NORTHFIELD QUADRANGLE
MASSACHUSETTS–NEW HAMPSHIRE–VERMONT
7.5 MINUTE SERIES (TOPOGRAPHIC)
202
252
GREENFIELD QUADRANGLE
MASSACHUSETTS–FRANKLIN CO.
7.5-MINUTE SERIES (TOPOGRAPHIC)
251
201
MILLERS FALLS QUADRANGLE
MASSACHUSETTS–FRANKLIN CO.
7.5 MINUTE SERIES (TOPOGRAPHIC)
CONNECTICUT
RIVER
Pisgah Mtn
TURNERS FALLS
AIRPORT
SCALE 1:24 000
1 MILE
1000 0 1000 2000 3000 4000 5000 6000 7000 FEET
1 KILOMETRE
CONTOUR INTERVAL 20 FEET
HIGHLAND


GREENFIELD QUADRANGLE
MASSACHUSETTS - FRANKLIN CO.
7.5-MINUTE SERIES (TOPOGRAPHIC)
SCALE 1:24000
1 MILE
FEET
1 KILOMETRE
CONTOUR INTERVAL 20 FEET
CEMETERIES
250
CONNECTICUT
Barton Cove
Barton Island
Riverside
Turners Falls
South End Sch
Rawson Island
Smead Island
Maynard Pond
Brook
POWER CANAL
WATER TANK
TRAIL
ROAD
MAIN
ADAMS
BERNARDSTON
MOHAWK
Mountain
Willis Hill
MONTAGUE
GRAVEL PIT
Montague City

APPENDIX B

INFORMATION SOURCES

INFORMATION SOURCES

Miscellaneous: Piezometer Data with Graphical Representation
1) 12 rolls of river gage data
2) 13 rolls of piezometer data (FD-1 and FD-2)
3) piezometer charts compiled by COE
4) Table of recorder elevations at Area 51 for river gage and piezometer (FD-1 and FD-2)

Miscellaneous Water Surface Information
1) Surface - water stations in downstream order from a 1974 state report
2) Table of water surface recordings, with tabulation of drawdown and storage (July 1977 - Sept. 1977)
3) Water surface charts at Norwich, Vt., Ompompanoosec and South Newbury

Cross Section Information
1) Various maps of Turners Falls Pool cross sections and cross section location guide
2) Rolls of graphs for Turners Falls showing Cross Sections A'-Z', AA'-NN, PP-TT and #1-#30
3) Connecticut River cross section, Rockingham N., Springfield
4) 6 rolls of partial cross sections showing bank erosion over time

Bridge Information, including inventory card files, survey plans and profiles and boring logs for the following bridges:
1) Hinsdale Br. - (042/044 - N.H. Rte. 119)
2) Hinsdale Br. - (041/040 - N.H. Rte. 119)
3) Chesterfield Br. - (040/095 - N.H. Rte. 9)
4) Walpole Br. - (132/062 - N.H. Rte. 123)
5) Walpole Br. - (062/052 - N.H. Rte. 12)
6) Claremont Br. - (065/134 - N.H. Rtes. 12 and 013)
7) Cornish Br. - (064/108)
8) Hanover Br. - (026/056)
9) Lebanon Br. - (044/103 - 044/104 - Interstate Rte. 89)
10) Lebanon Br. - (053/127 - U.S. Rte. 14)
11) Lyme Br. - (053/112)
12) Orford Br. - (062/124 - N.H. Rte. 25-A)
13) Piermont Br. - (032/103 - N.H. Rte. 25)
14) Haverhill Br. - (063/162)
15) Haverhill Br. - (099/149)
16) No information available on Walpole Br. (058/044) or Charleston Br. (135/052)

Miscellaneous Maps:
1) 24 topographic maps of the Connecticut River
2) 7½ and 15 minute maps from Brattleboro to Woodsville
3) Maps of Areas 15, 301, 255, Reaches 147, 26 and 31
4) Turners Falls Pool Topographic
5) Detailed maps of the 4 pools behind dams
6) Hydrographic maps of Connecticut River Power Stations, 1945 and 1950.

Miscellaneous Correspondence
1) 7 letters between John Kalafut and the U.S. Army Engineering Division
2) 5 letters of declared interest in the project from various groups and individuals

Miscellaneous Data
1) Boat count and wave observations
2) USGS inflow data on Turners Falls
3) Raw data for study
4) Operation data for study period on Turners Falls Pool
5) Operation logs and pool stages for Wilder Pool
6) Turners Falls and Wilder Pool gradually varied flow output
7) 3 notebooks of Connecticut River erosion photos
8) Film showing wave action
9) River and bank water levels for Area 51, Hanover, NH
10) 49 survey record sheets at various locations

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APPENDIX C

BANK EROSION PROTECTION MEASURES SUGGESTED IN THE
INTERIM REPORT TO CONGRESS--U.S. ARMY CORPS OF ENGINEERS
SEPTEMBER, 1978

BANK EROSION PROTECTION MEASURES SUGGESTED IN THE INTERIM REPORT TO CONGRESS--U.S. ARMY CORPS OF ENGINEERS, 1978

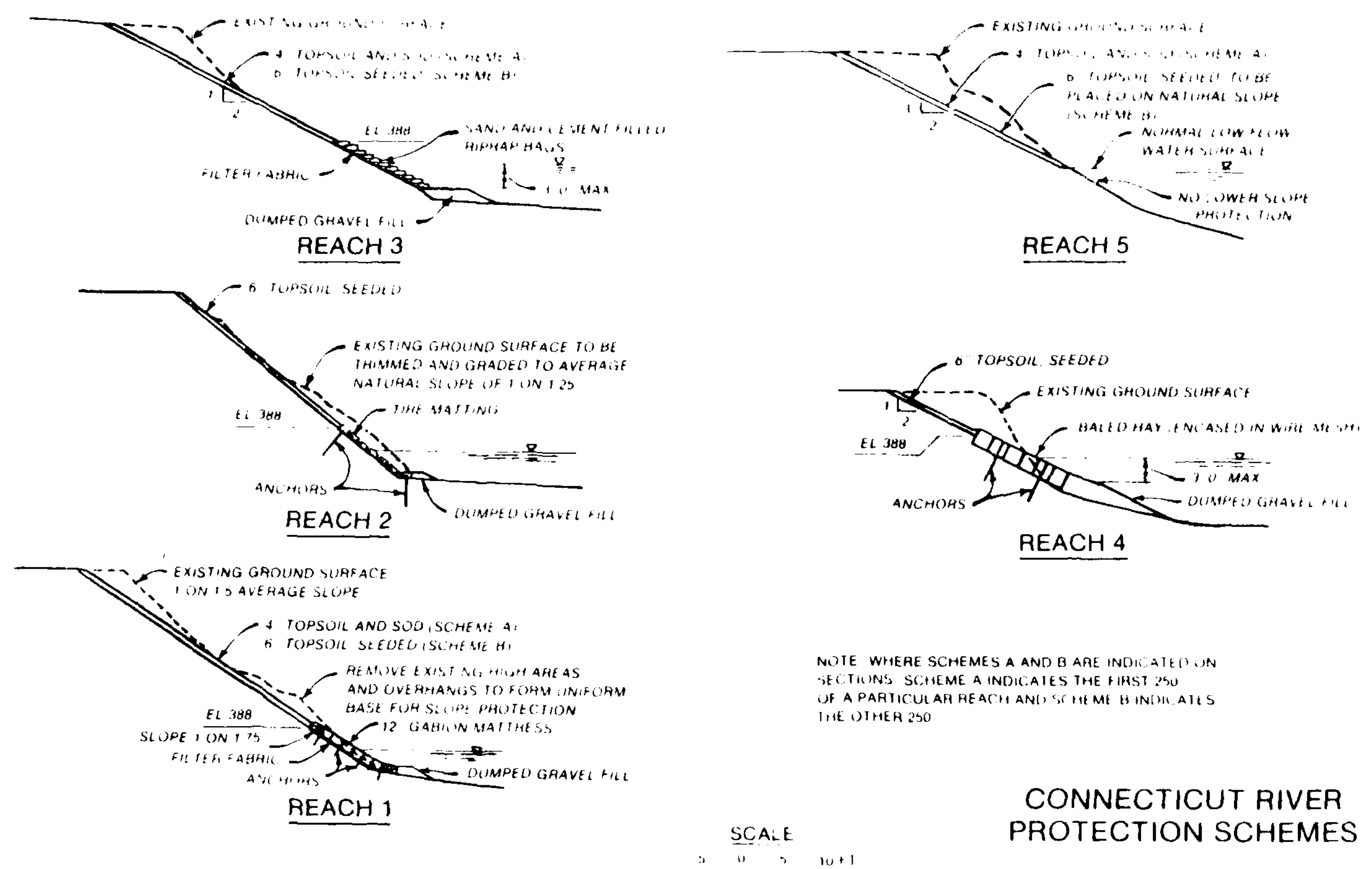
As mentioned in the introductory sections, the problem of bank erosion of channels is of growing national concern. In response to this concern the Section 32 Program tied to the Streambank Erosion Control Evaluation and Demonstration Act of 1974 was implemented. The broad objectives of the authorizing legislators are indicated by the work units comprising the program. The specific work units as reported in 1978 include:

1. Evaluation of extent of streambank erosion, nationwide.
2. Literature survey and evaluation of bank protection methods.
3. Hydraulic research on effectiveness of bank protection methods.
4. Research on soil stability and identification of causes of streambank erosion.
5. Ohio River demonstration projects.
6. Missouri River demonstration projects.
7. Yazoo River Basin demonstration projects.
8. Demonstration projects on other streams, nationwide.
9. Reconstruction at demonstration projects.
10. Reports to Congress.

Brief descriptions of these work units were given in the 1978 report.

The methodologies being tested at various sites nationally are of particular interest because several of them may be applicable to Connecticut River streambank erosion problems. Even though these methodologies and treatments have not yet been evaluated, it is worthwhile to consider them. The subsequent pages were taken from the Interim Report to Congress, September 30, 1978.

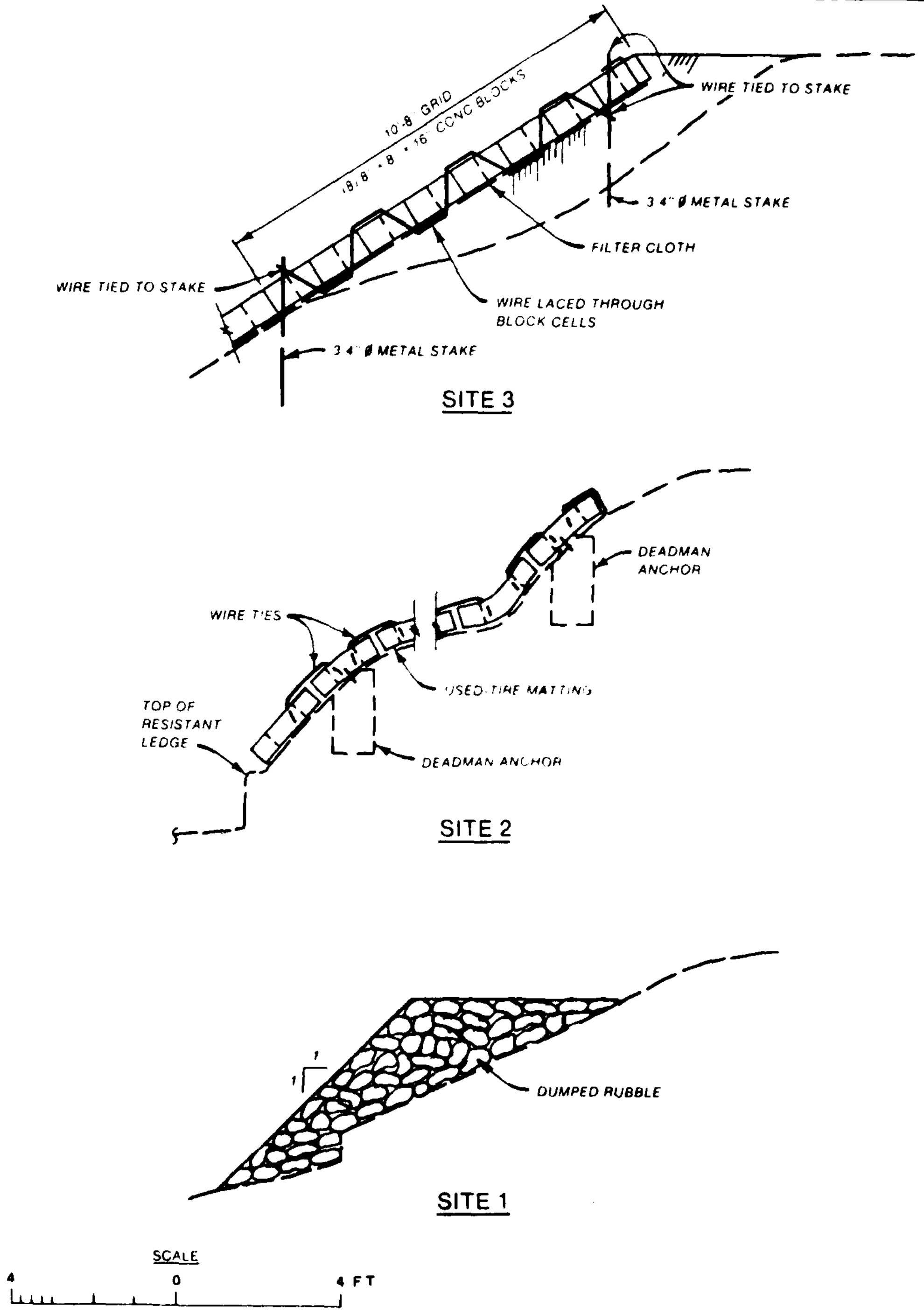
In general, about 15-20 percent of the length of most natural channels exhibit some degree of bank erosion and channel instabilities. Secondly, if bank erosion is eliminated, certain adverse effects may result as identified in the Interim Report to Congress.



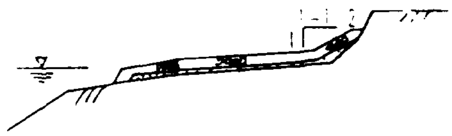
TYPICAL SCHEMES

CONNECTICUT RIVER
PROTECTION SCHEMES

HAVERHILL, NEW HAMPSHIRE

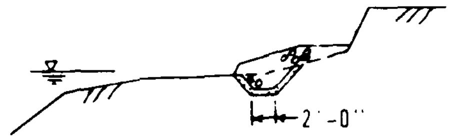


PEARL RIVER
MONTICELLO, MISS.
TYPICAL PROTECTION SCHEMES



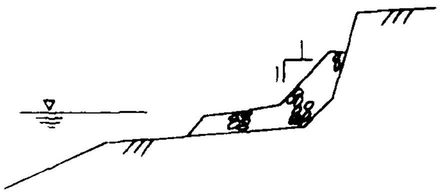
12-in.-THICK GRADED STEEL FURNACE SLAG BLANKET ATOP 6-in.-THICK GRADED SAND AND GRAVEL FILTER FROM EL 623.0 TO EL 628.0.

SCHEME 1



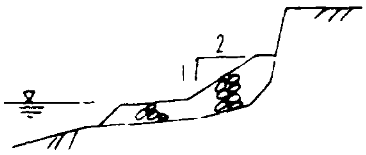
WEDGE OF GRADED STEEL FURNACE SLAG FROM EL 621.0 TO EL 626.0 SET IN TRENCH LINED WITH 6-in.-THICK GRADED SAND AND GRAVEL FILTER

SCHEME 2



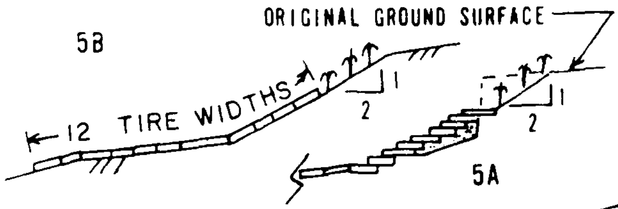
24-in.-THICK GRADED STEEL FURNACE SLAG BLANKET FROM EL 621.0 TO EL 630.0 WITHOUT FILTER

SCHEME 3



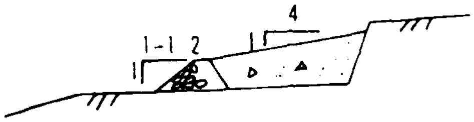
24-in.-THICK GRADED STEEL FURNACE SLAG BLANKET FROM EL 622.0 TO EL 627.0 WITHOUT FILTER

SCHEME 4



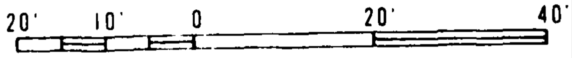
PLANT SHOOTS WITH MAT COVER FROM EL 628.0 TO TOP OF CUT SLOPE. SCHEME 5A IS STACKED RUBBER TIRE WALL FROM EL 623.5 TO EL 628.0 WITH GRAVEL-FILLED TIED TIRE BLANKET 6 TIRE WIDTHS BELOW EL 623.5. SCHEME 5B IS GRAVEL-FILLED TIED TIRE BLANKET 12 TIRE WIDTHS BELOW EL 628.0

SCHEMES 5A & 5B



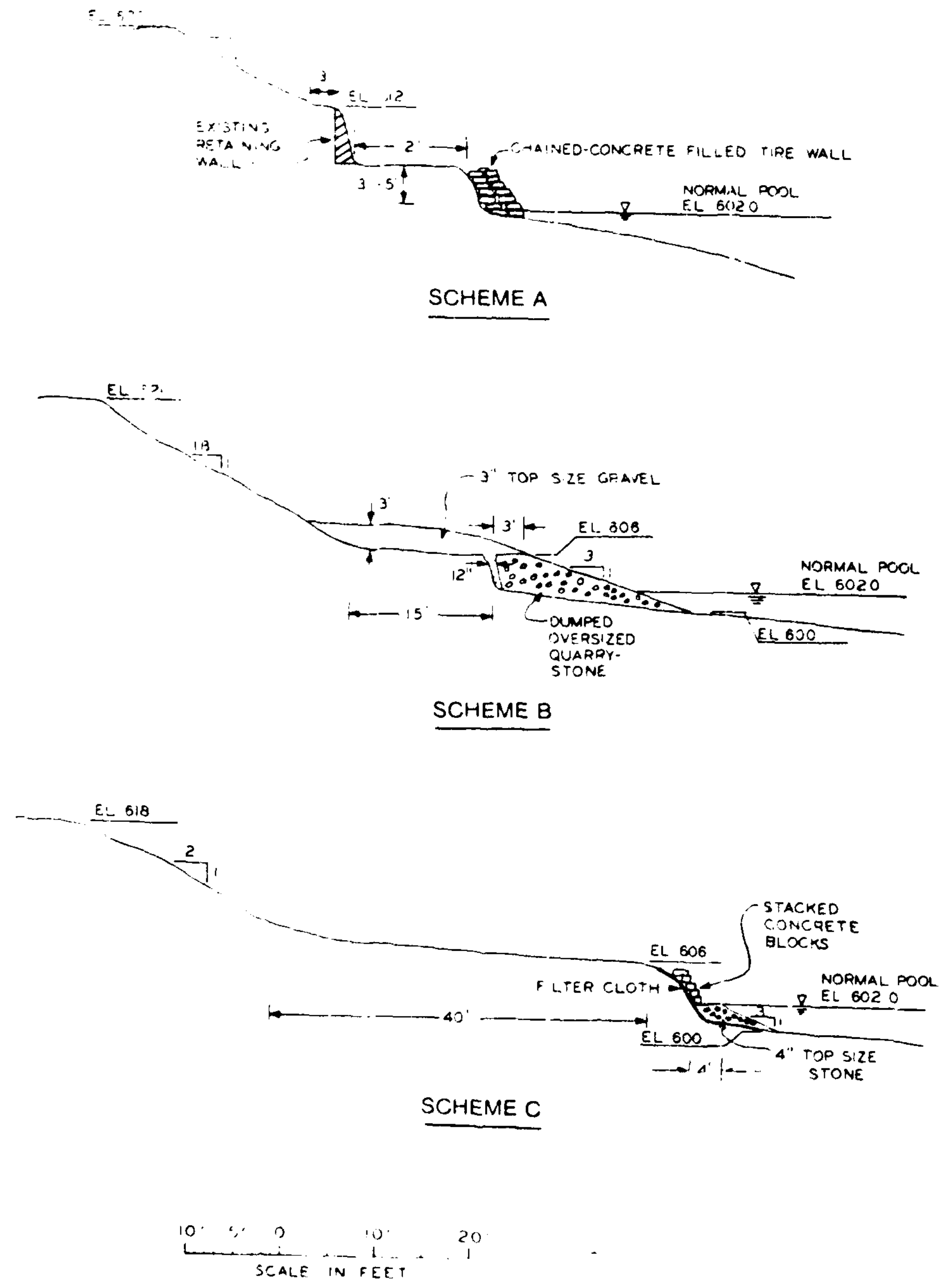
GRADED STEEL FURNACE SLAG WEDGE TO EL 625.0. SAND AND GRAVEL BACKFILL FROM EL 625.0 TO EL 628.0

SCHEME 6

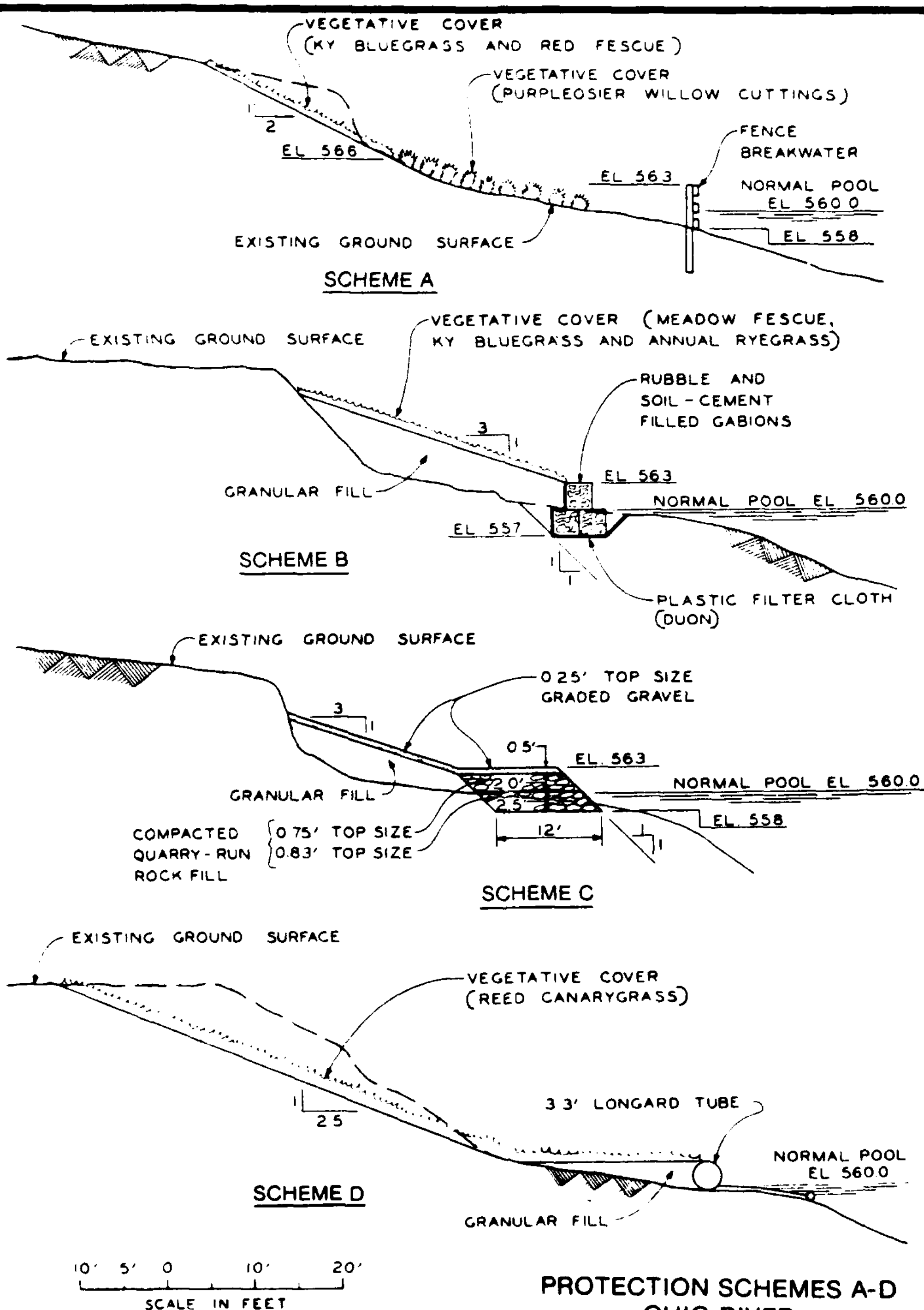


PROTECTION SCHEMES 1-6
POWHATAN POINT, OHIO SITE
OHIO RIVER

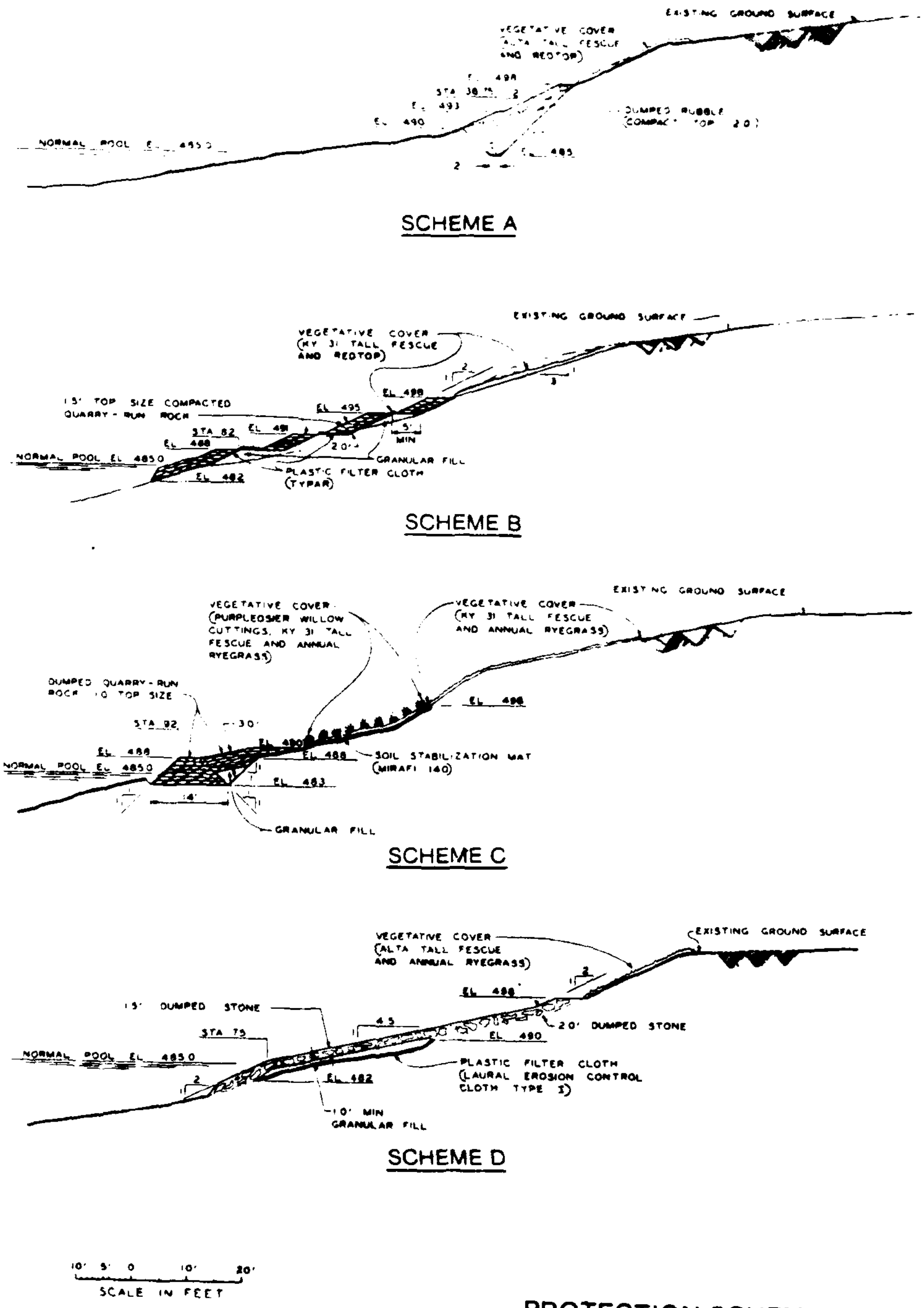
ORPED-GS



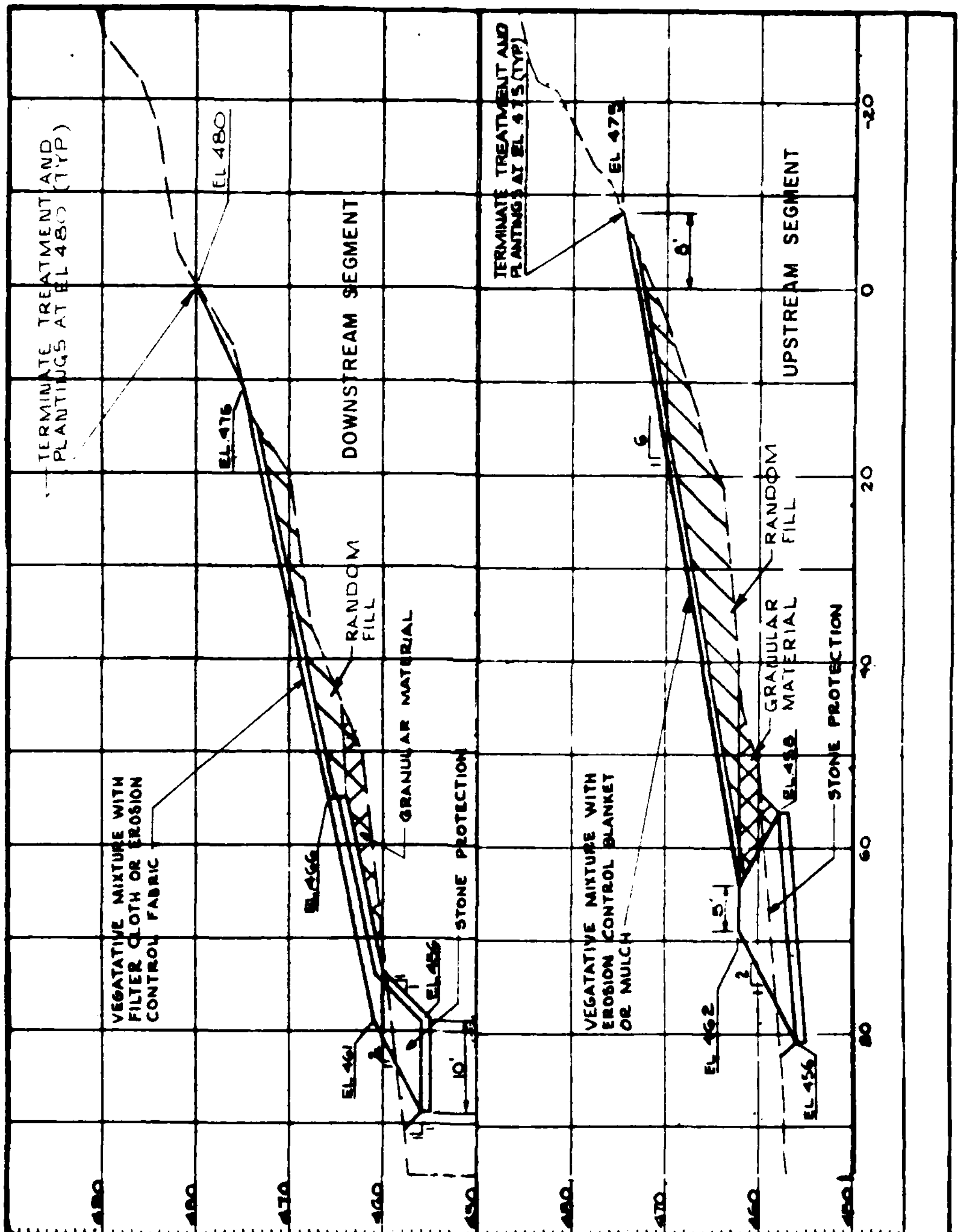
PROTECTION SCHEMES A-C
OHIO RIVER
ST. MARYS, W. VA.



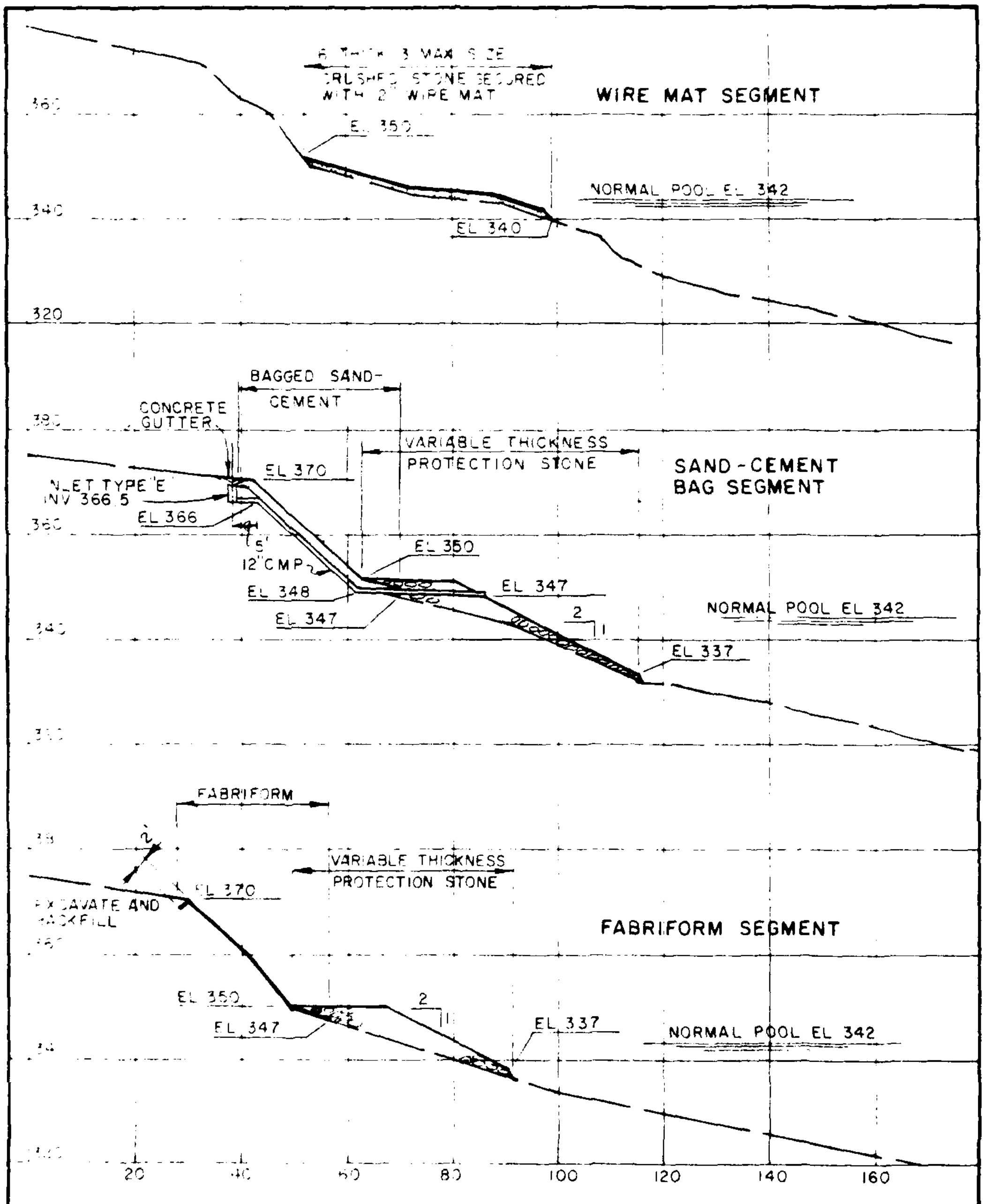
PROTECTION SCHEMES A-D
OHIO RIVER
RAVENSWOOD, W. VA.



PROTECTION SCHEMES A-D
OHIO RIVER
PORTSMOUTH, OHIO

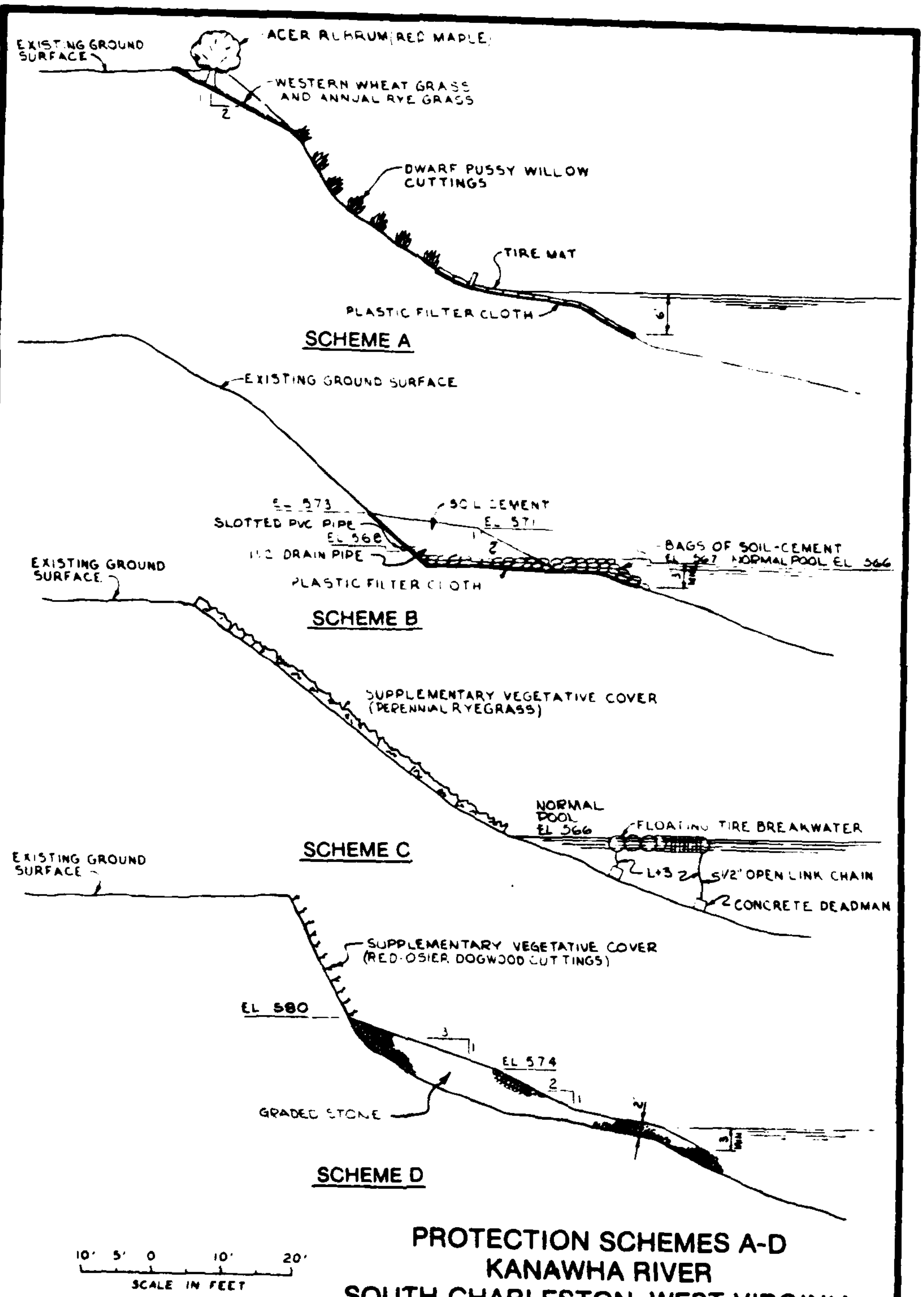


STREAMBANK EROSION CONTROL
DEMONSTRATION PROJECT
MOSCOW, OHIO
SECTIONS

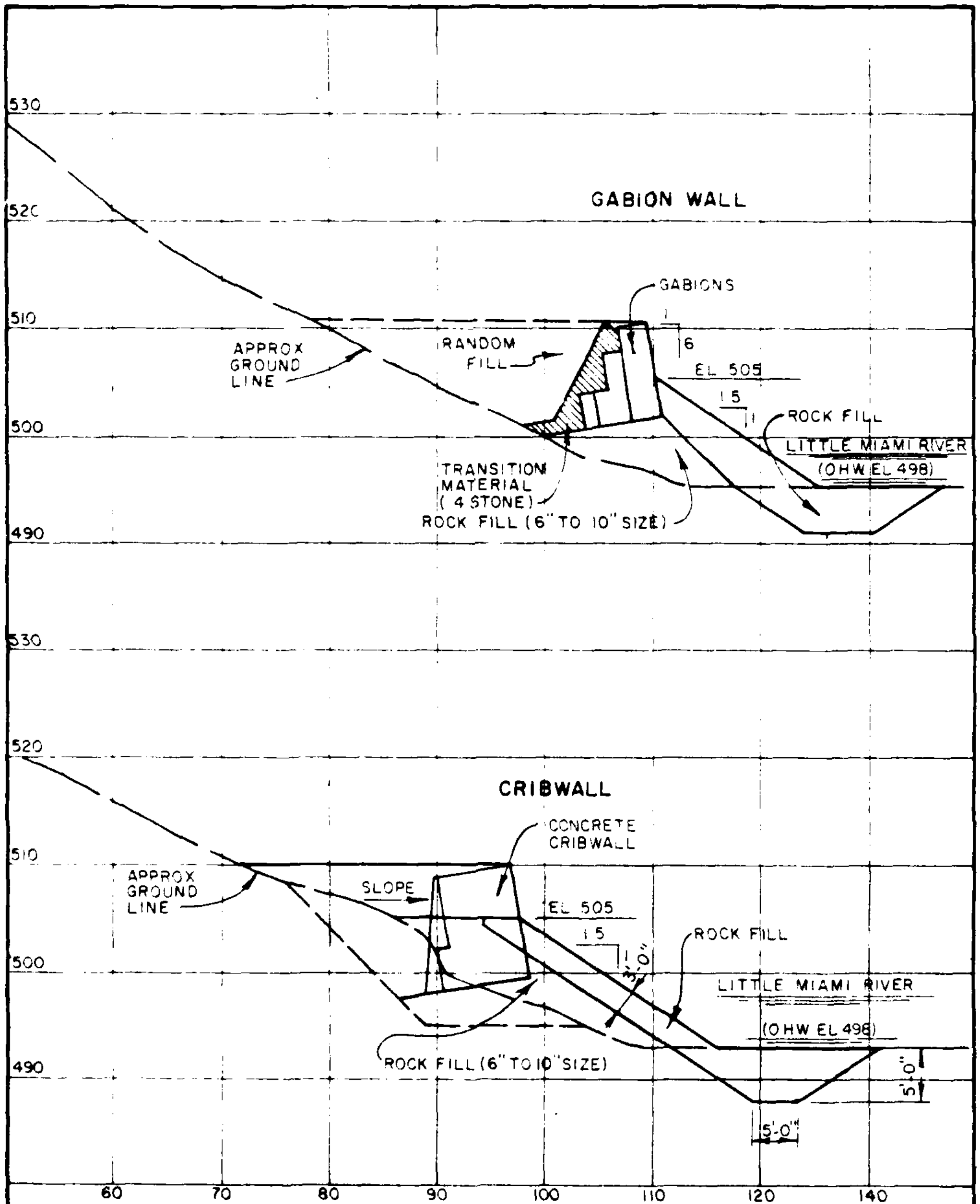


STREAMBANK EROSION CONTROL
DEMONSTRATION PROJECT
MOUNT VERNON, INDIANA

SECTIONS



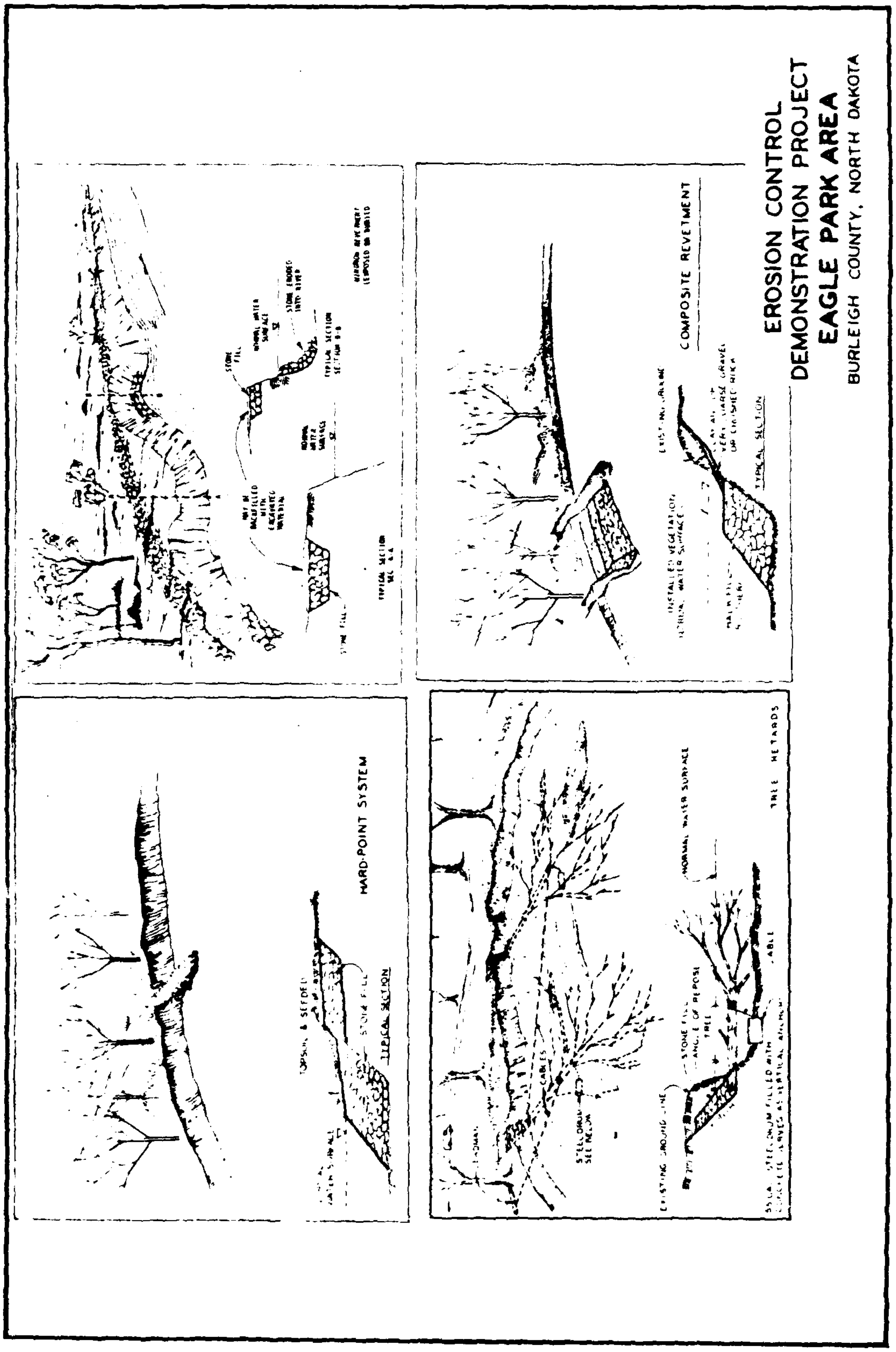
PROTECTION SCHEMES A-D
KANAWHA RIVER
SOUTH CHARLESTON, WEST VIRGINIA

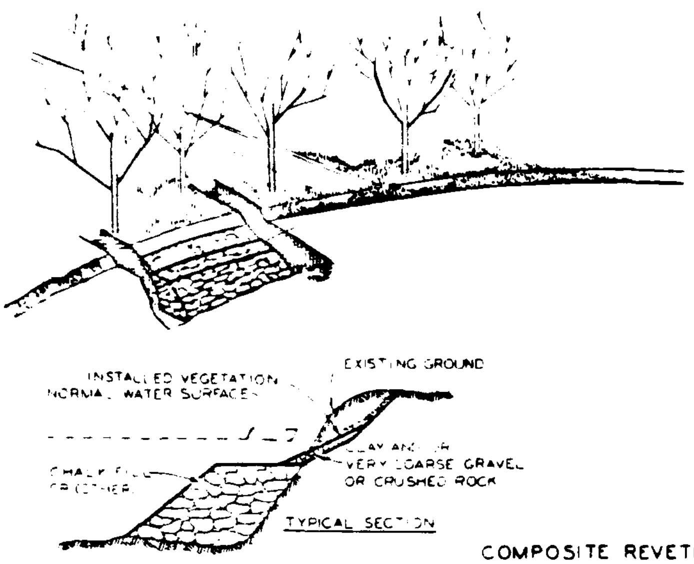


STREAMBANK EROSION CONTROL
DEMONSTRATION PROJECT

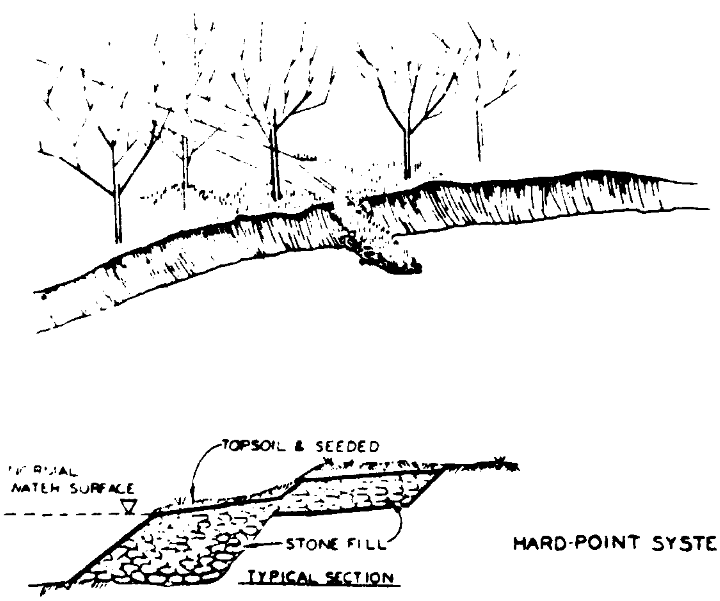
SECTION GABIONS—CONCRETE CRIBBING

MILFORD, OHIO


EROSION CONTROL
DEMONSTRATION PROJECT
EAGLE PARK AREA
BURLEIGH COUNTY, NORTH DAKOTA
COMPOSITE REVETMENT
TYPICAL SECTION
HARD-POINT SYSTEM
TYPICAL SECTION
STONE FILL
NORMAL WATER SURFACE
TREE RETARDS



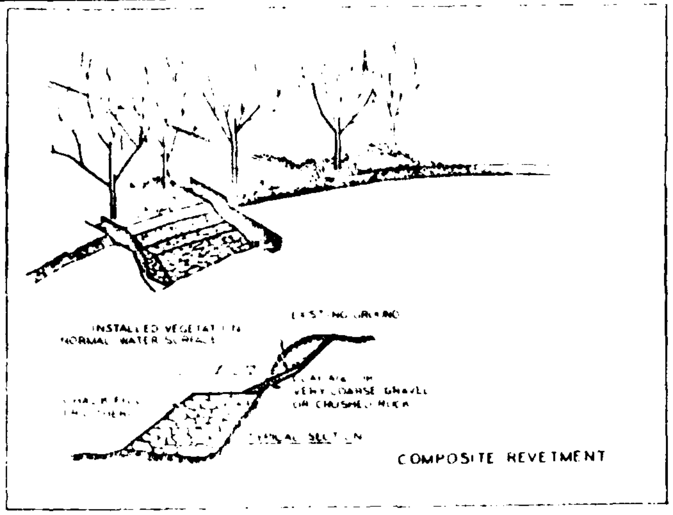
COMPOSITE REVETMENT

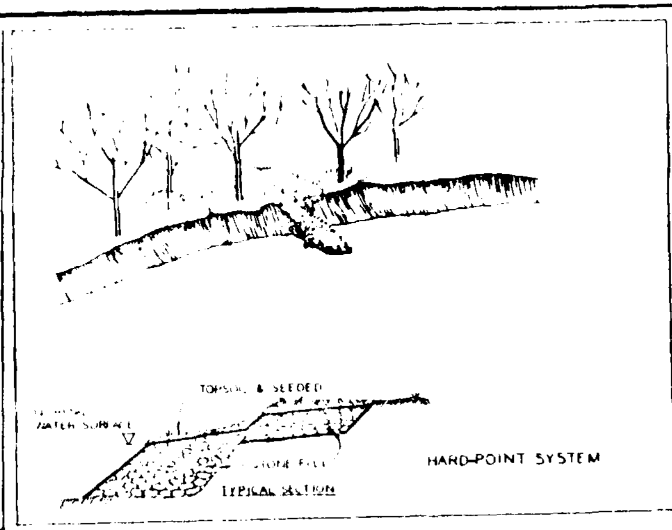


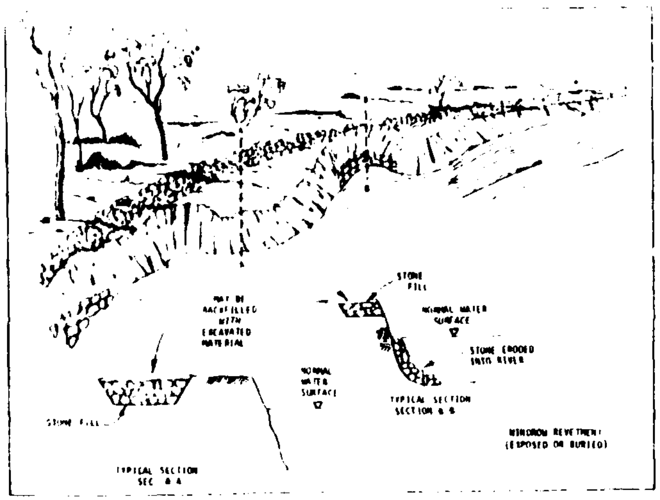
HARD-POINT SYSTEM

EROSION CONTROL
VERMILLION BOAT CLUB

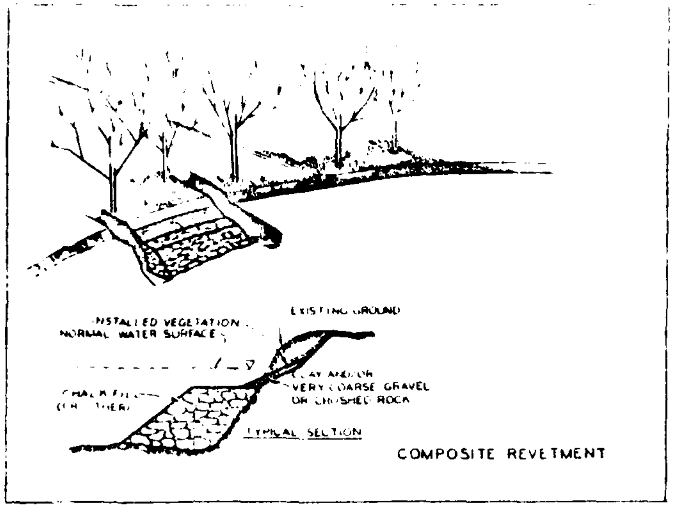
CLAY COUNTY, SOUTH DAKOTA







EROSION CONTROL
DEMONSTRATION PROJECT
BROOKY BOTTOM ROAD AREA
CEDAR COUNTY, NEBRASKA



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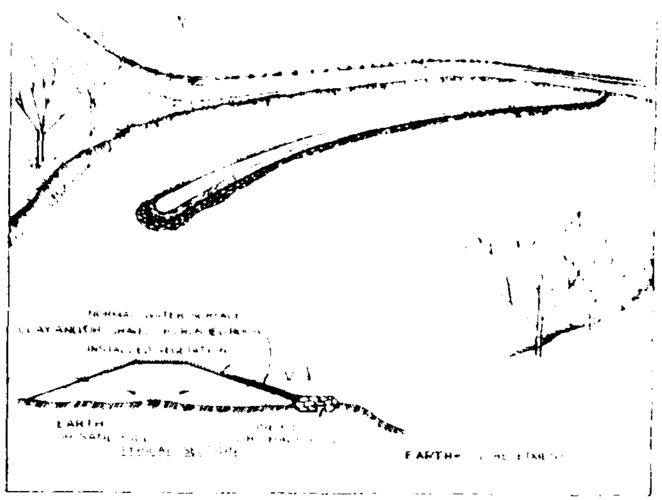
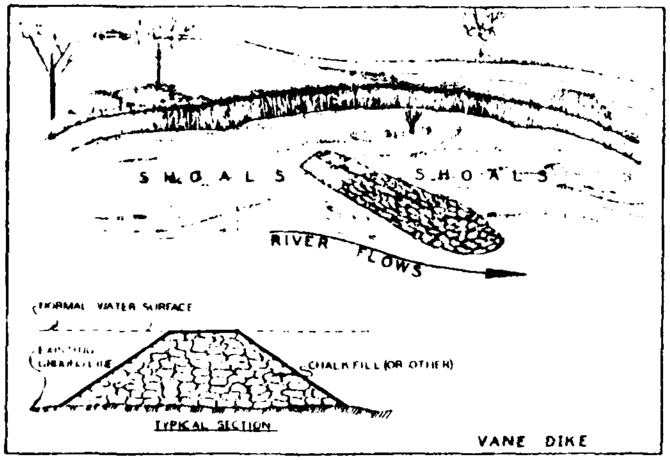


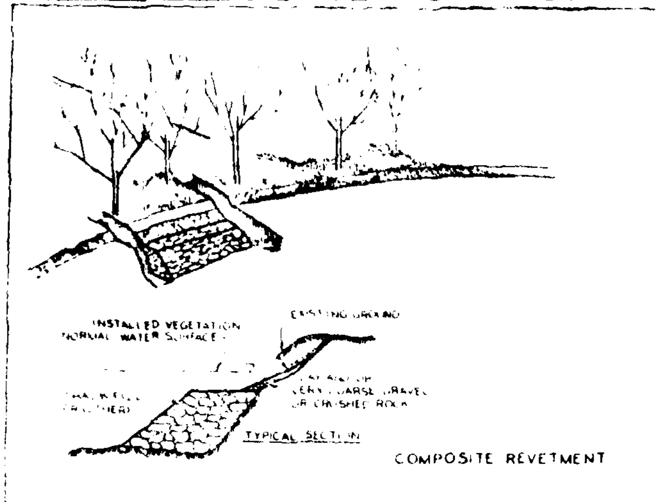


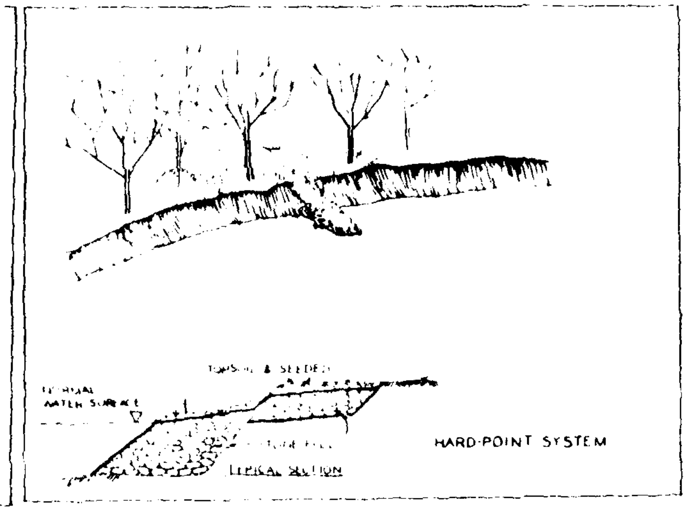
VANE DIKE

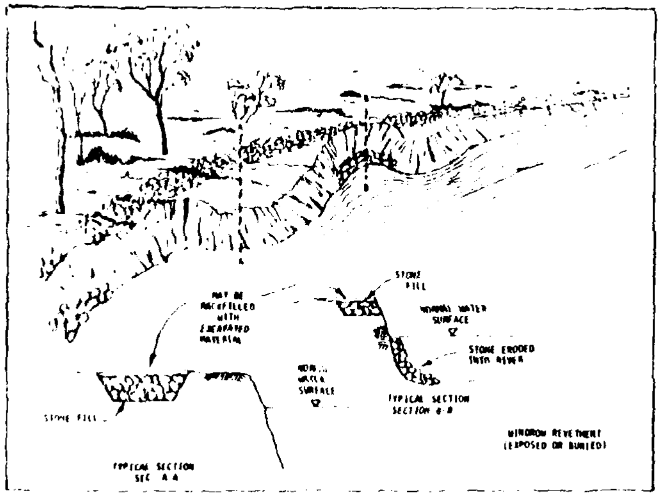
EROSION CONTROL
DEMONSTRATION PROJECT

MULBERRY BEND AREA

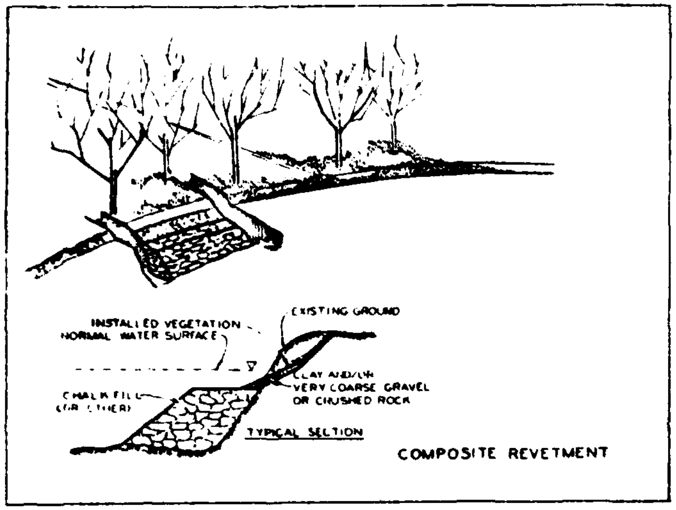
DIXON COUNTY, NEBRASKA

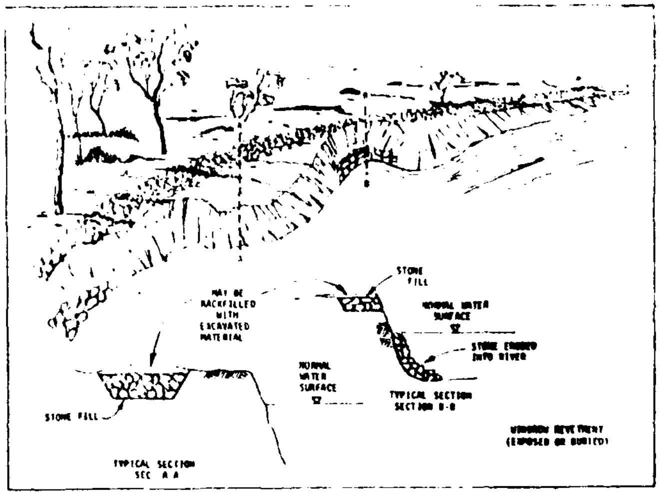


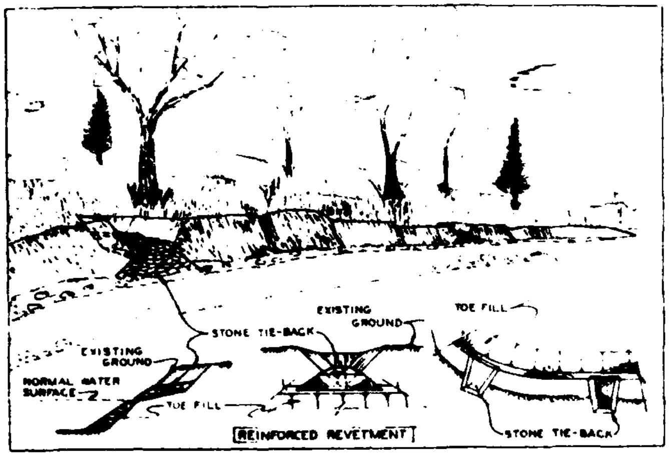




EROSION CONTROL
DEMONSTRATION PROJECT
VERMILLION RIVER CHUTE
CLAY COUNTY, SOUTH DAKOTA



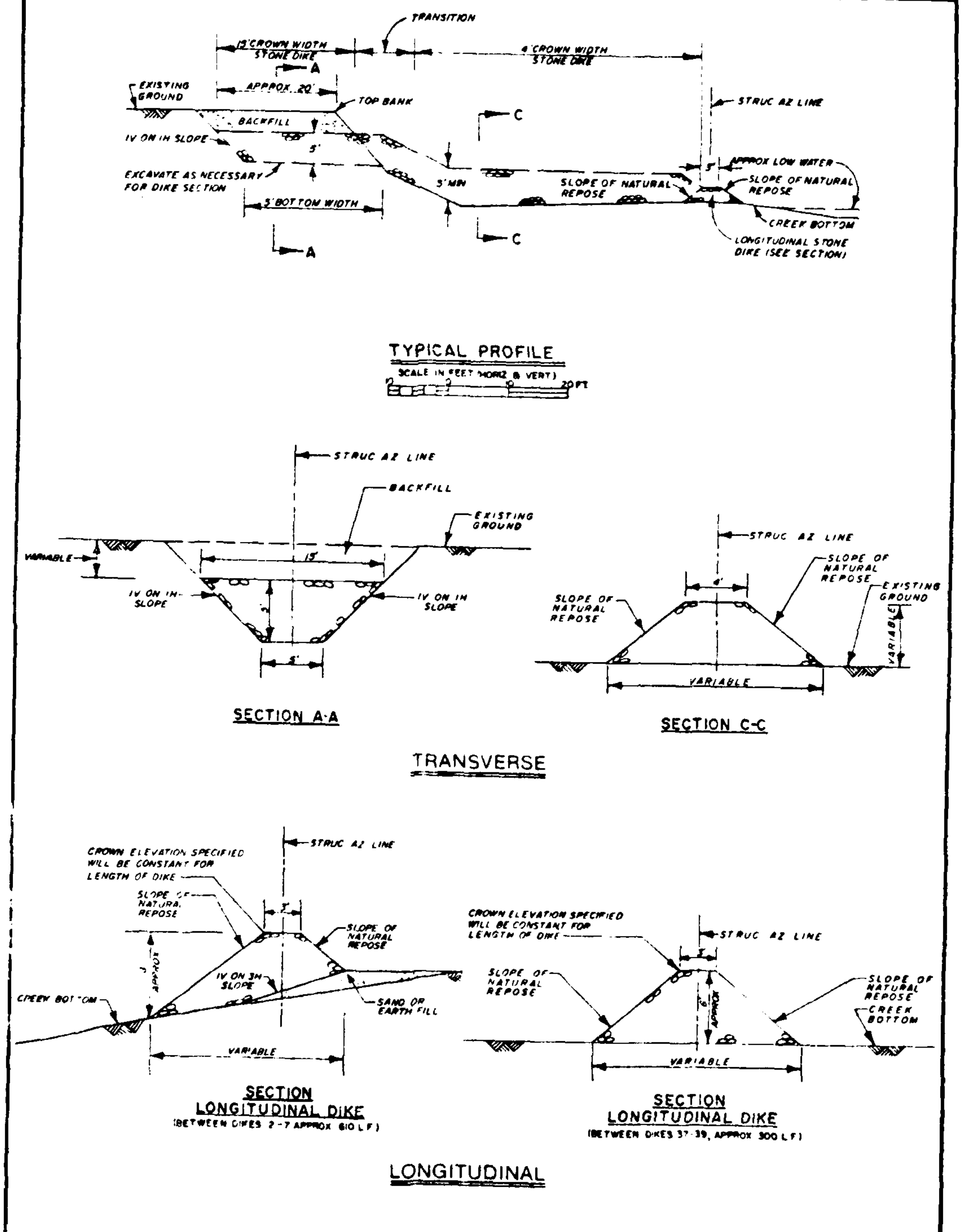




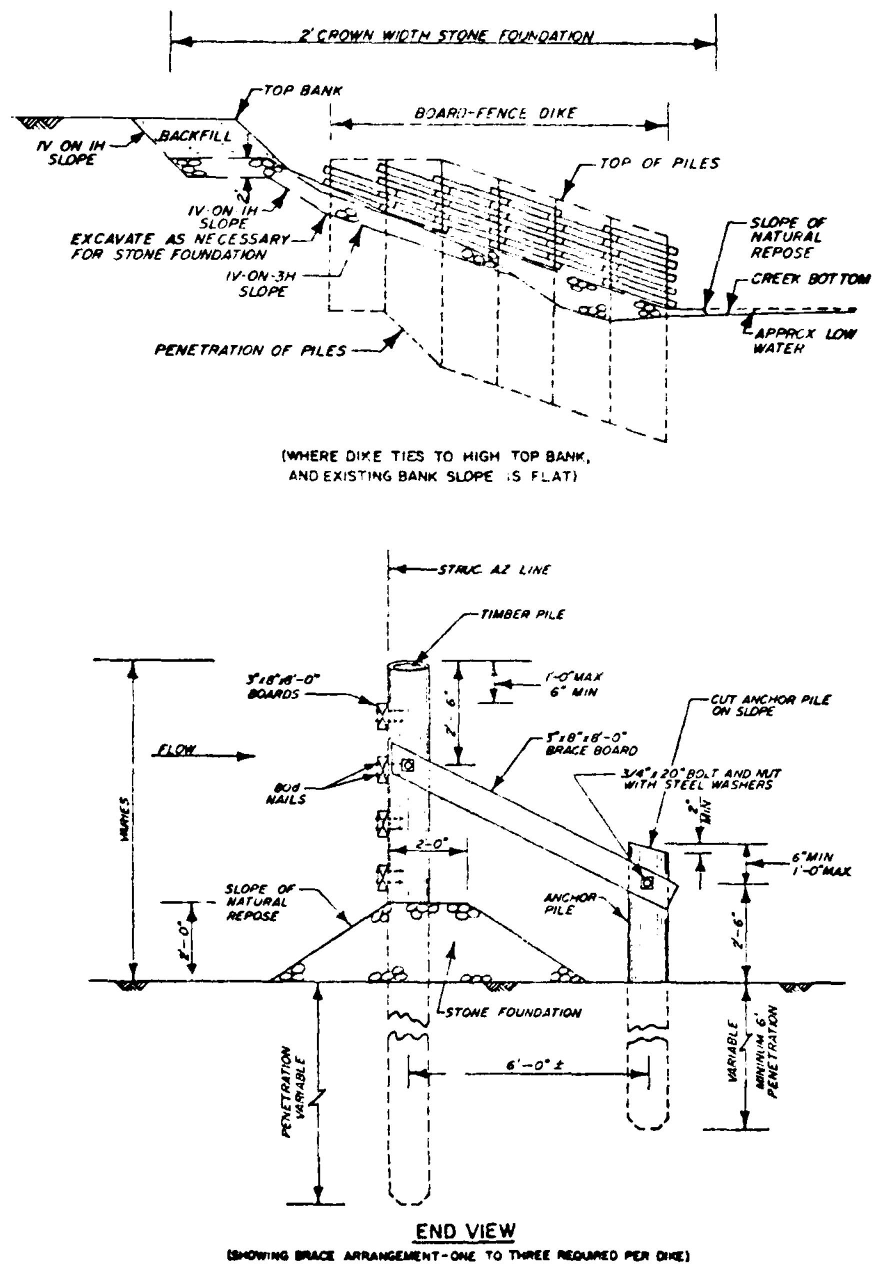
EROSION CONTROL
DEMONSTRATION PROJECT

RYAN BEND AREA

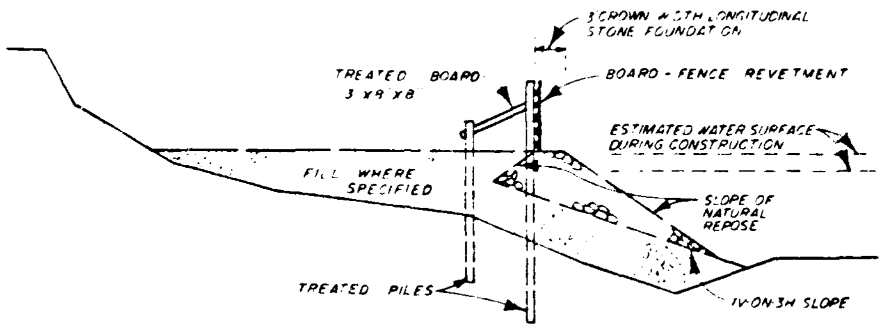
DIXON COUNTY, NEBRASKA



TYPICAL TRANSVERSE AND LONGITUDINAL STONE DIKES

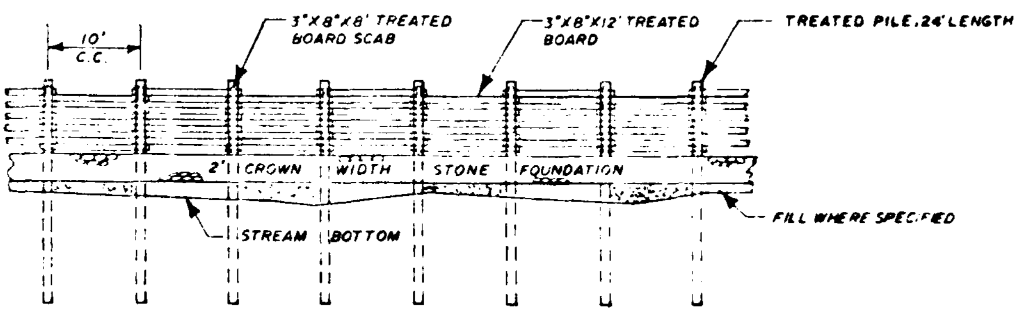


TYPICAL BOARD-FENCE DIKE



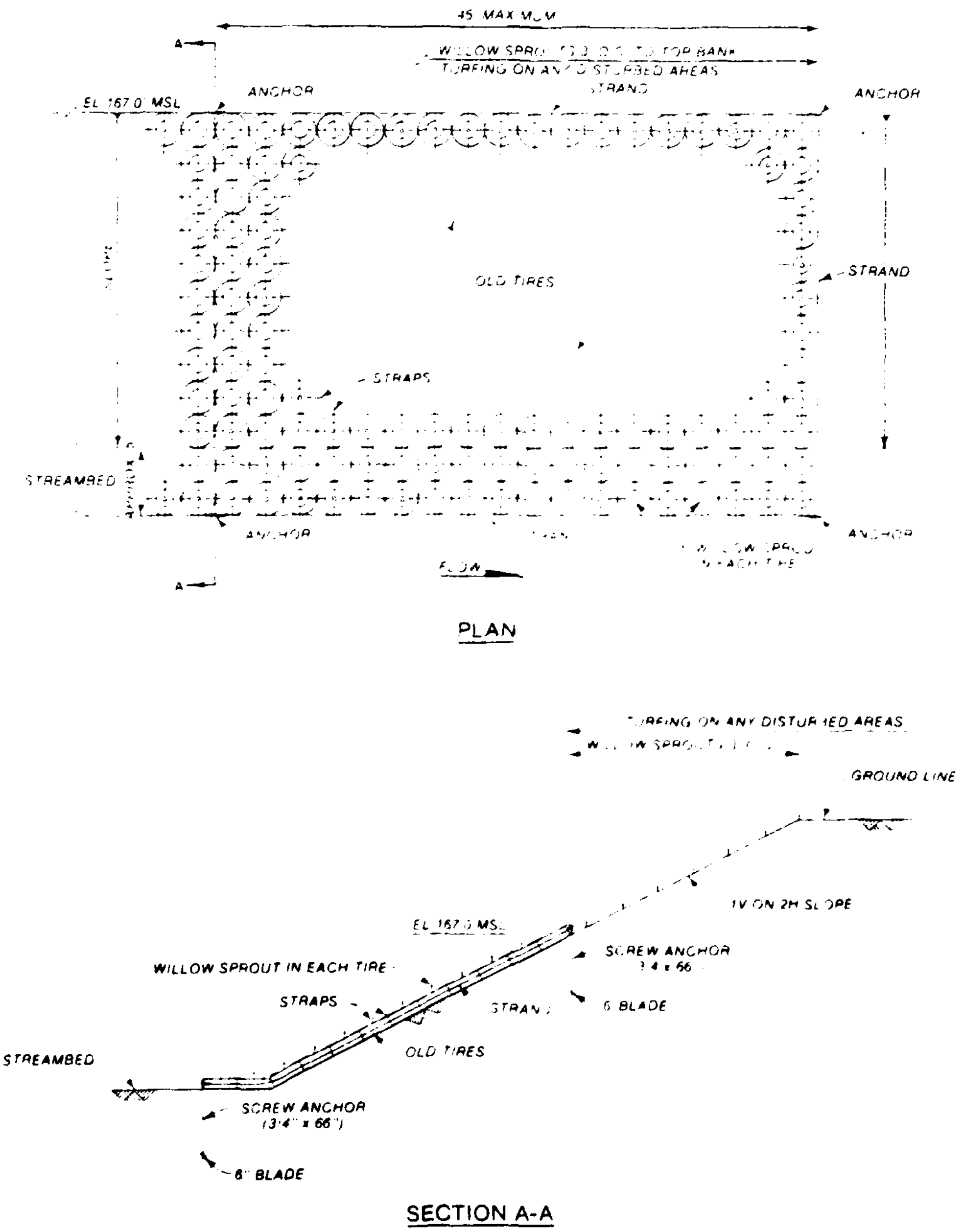
PROFILE

REVETMENT BETWEEN TIE-BACKS

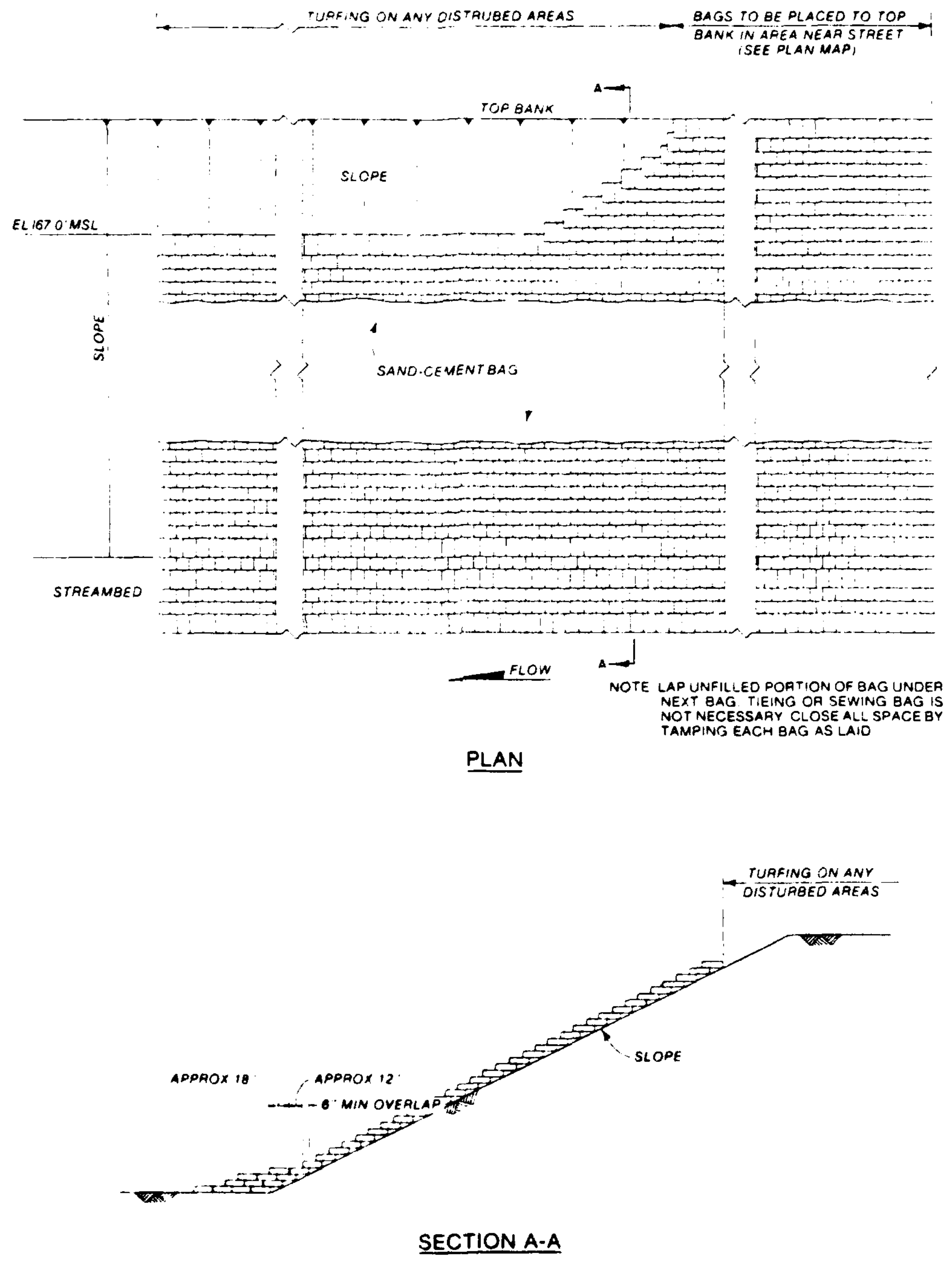


FRONT VIEW

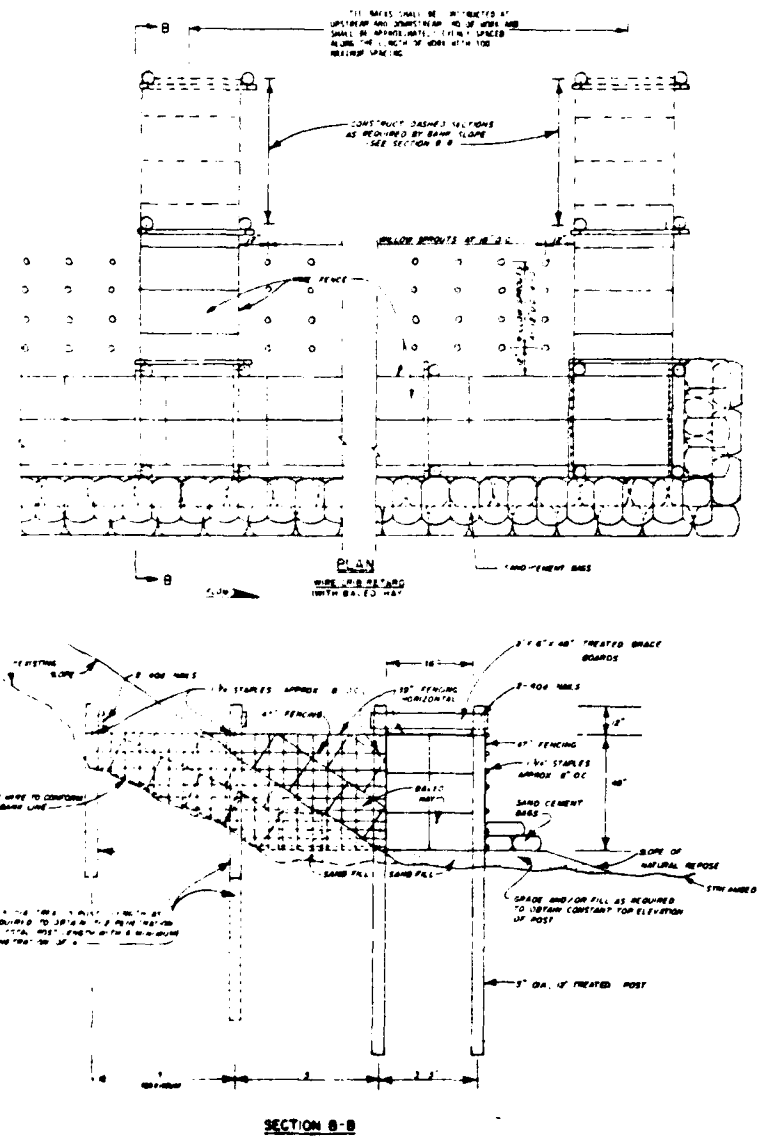
TYPICAL BOARD-FENCE
LONGITUDINAL REVETMENT



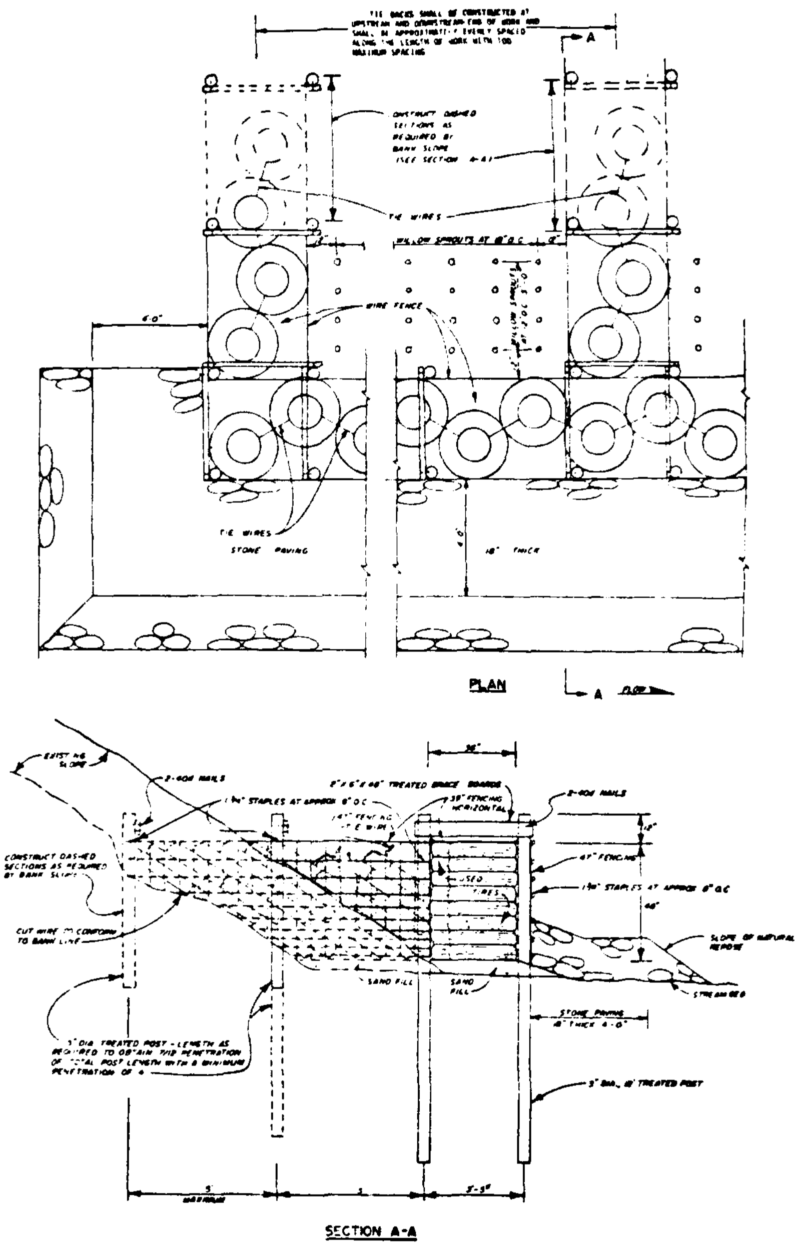
TYPICAL
RUBBER-TIRE REVETMENT



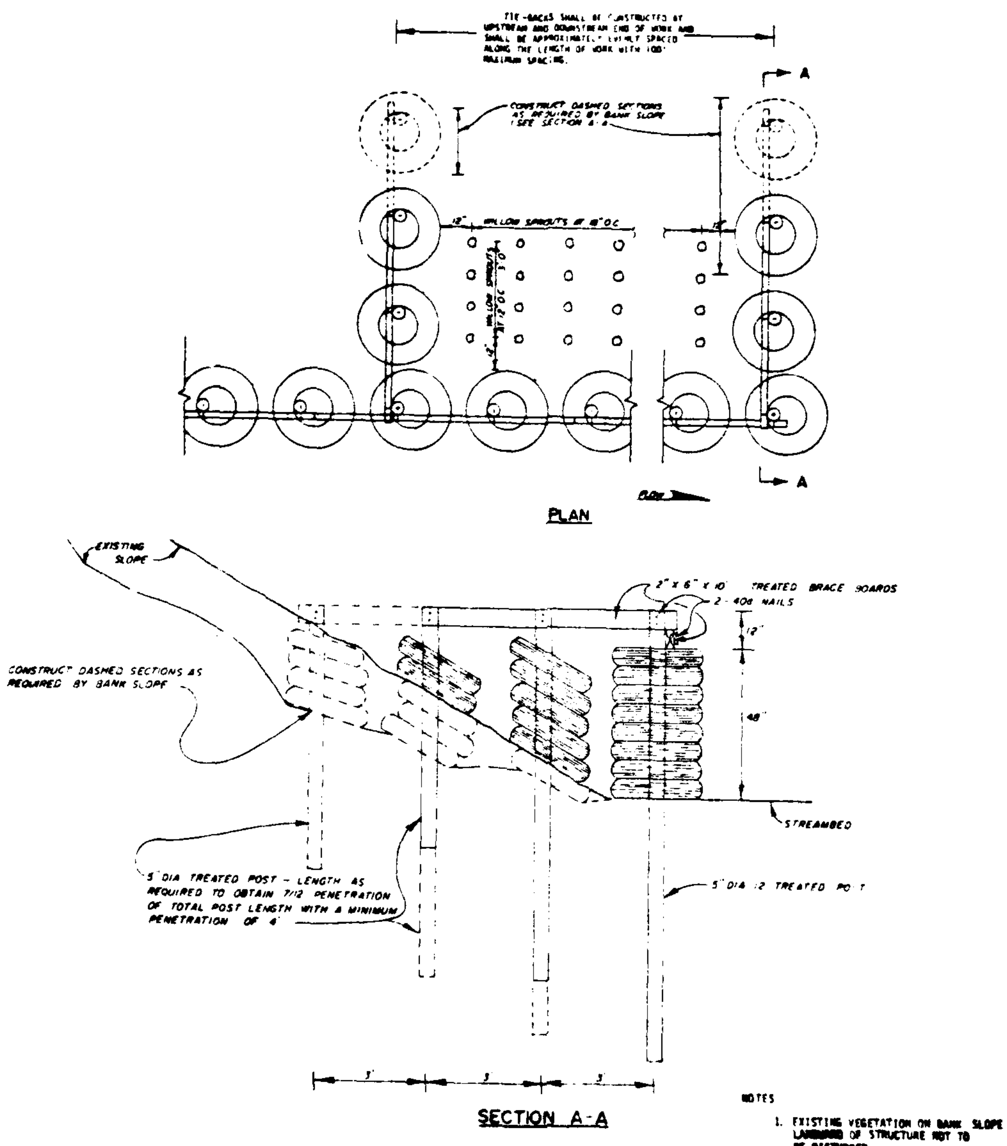
TYPICAL
SAND-CEMENT BAG REVETMENT



TYPICAL WIRE CRIB RETARDS
BALED-HAY-FILLED



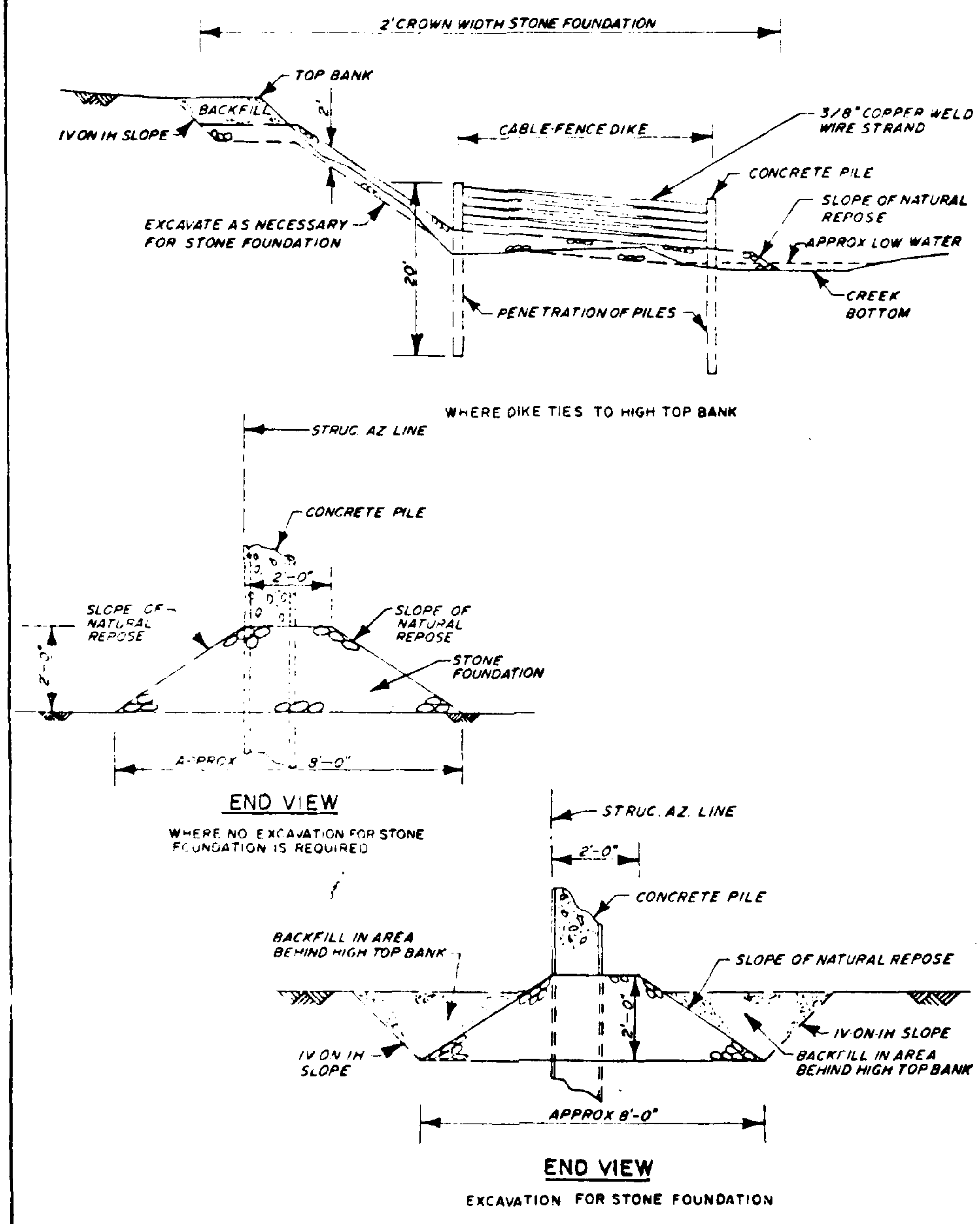
TYPICAL WIRE CRIB RETARDS
TIRE-FILLED



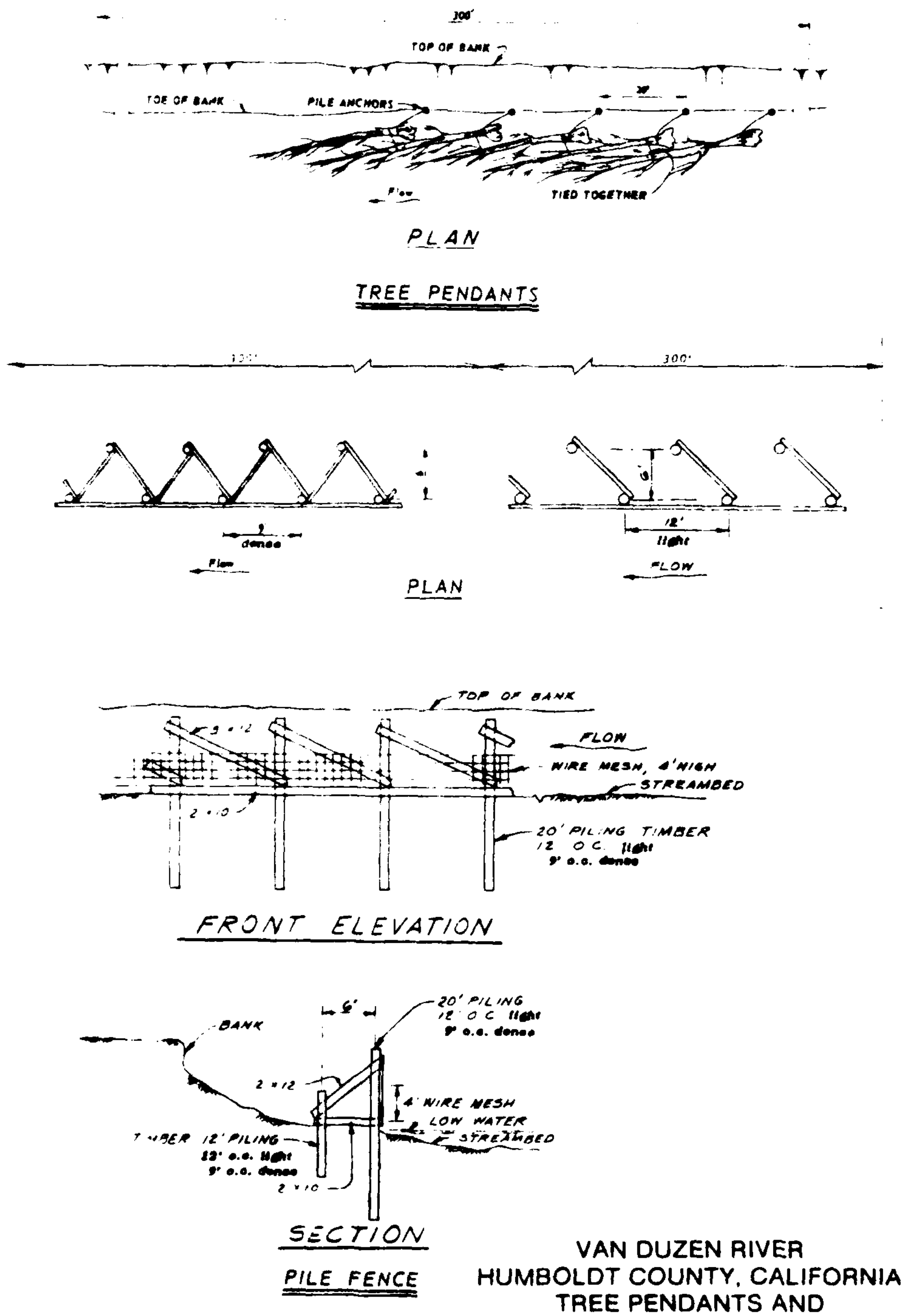
NOTES

1. EXISTING VEGETATION ON BANK SLOPE LANDWARD OF STRUCTURE NOT TO BE DISTURBED.

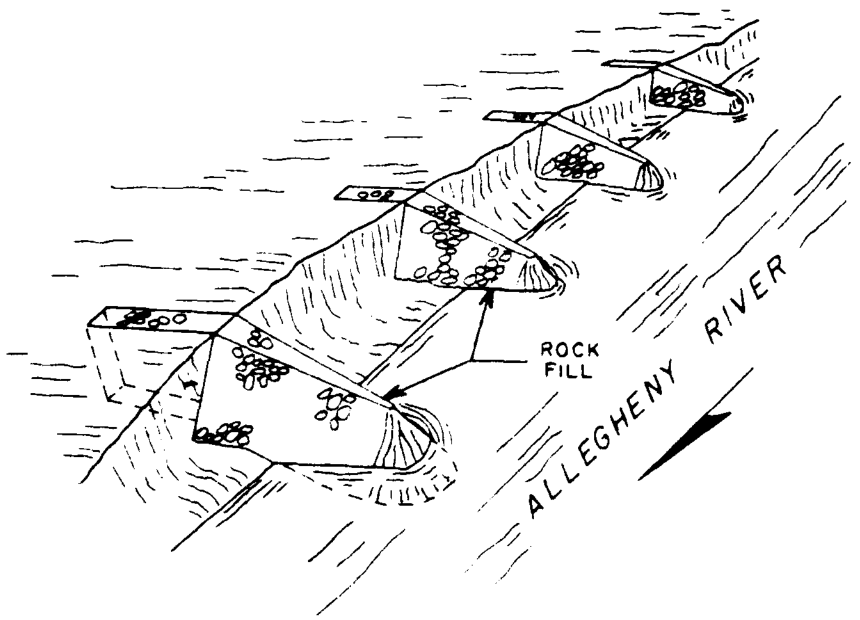
TYPICAL TIRE POST RETARDS



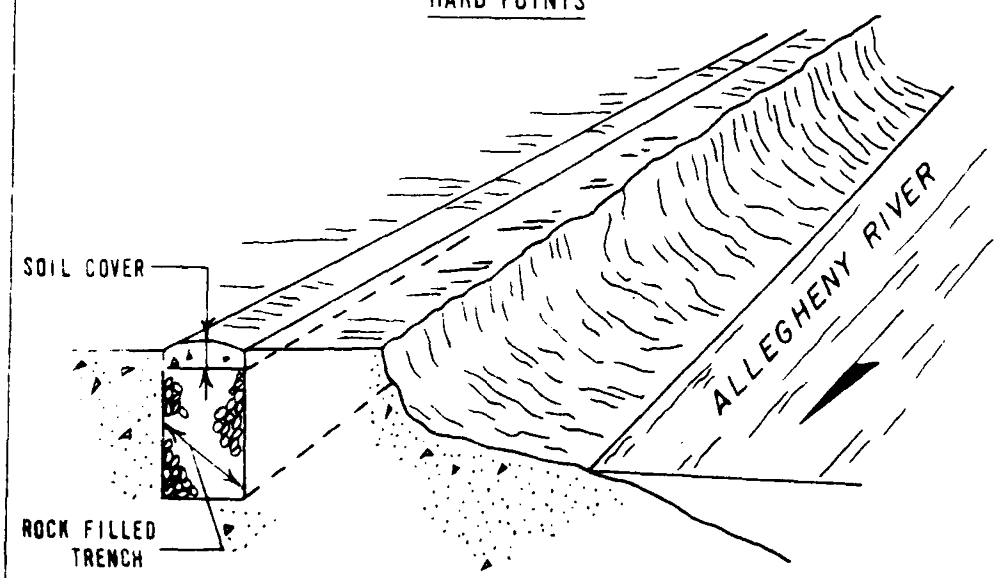
TYPICAL CABLE-FENCE DIKE



VAN DUZEN RIVER
HUMBOLDT COUNTY, CALIFORNIA
TREE PENDANTS AND
PILE-FENCE PROTECTION



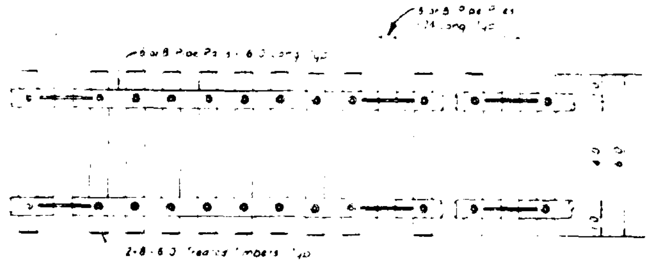
HARD POINTS



WINDROW

ALLEGHENY RIVER
WATTERSONVILLE, PENNSYLVANIA
POTENTIAL PROTECTION SCHEMES

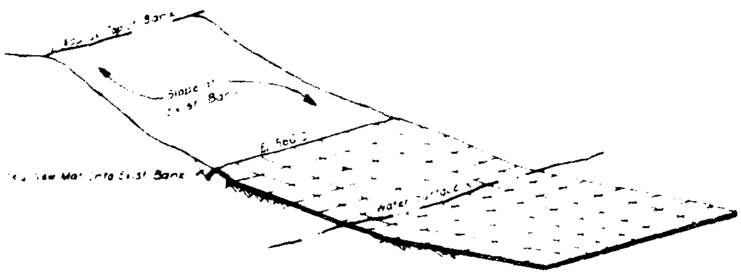
ORPED GS



ELEVATION

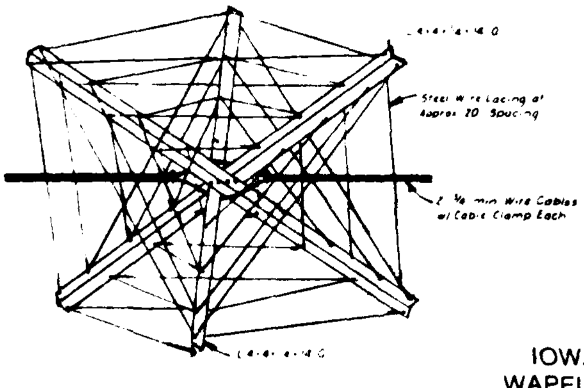
PERMEABLE TIMBER JETTY DETAILS

NO SCALE



CONCRETE-FILLED MAT DETAILS

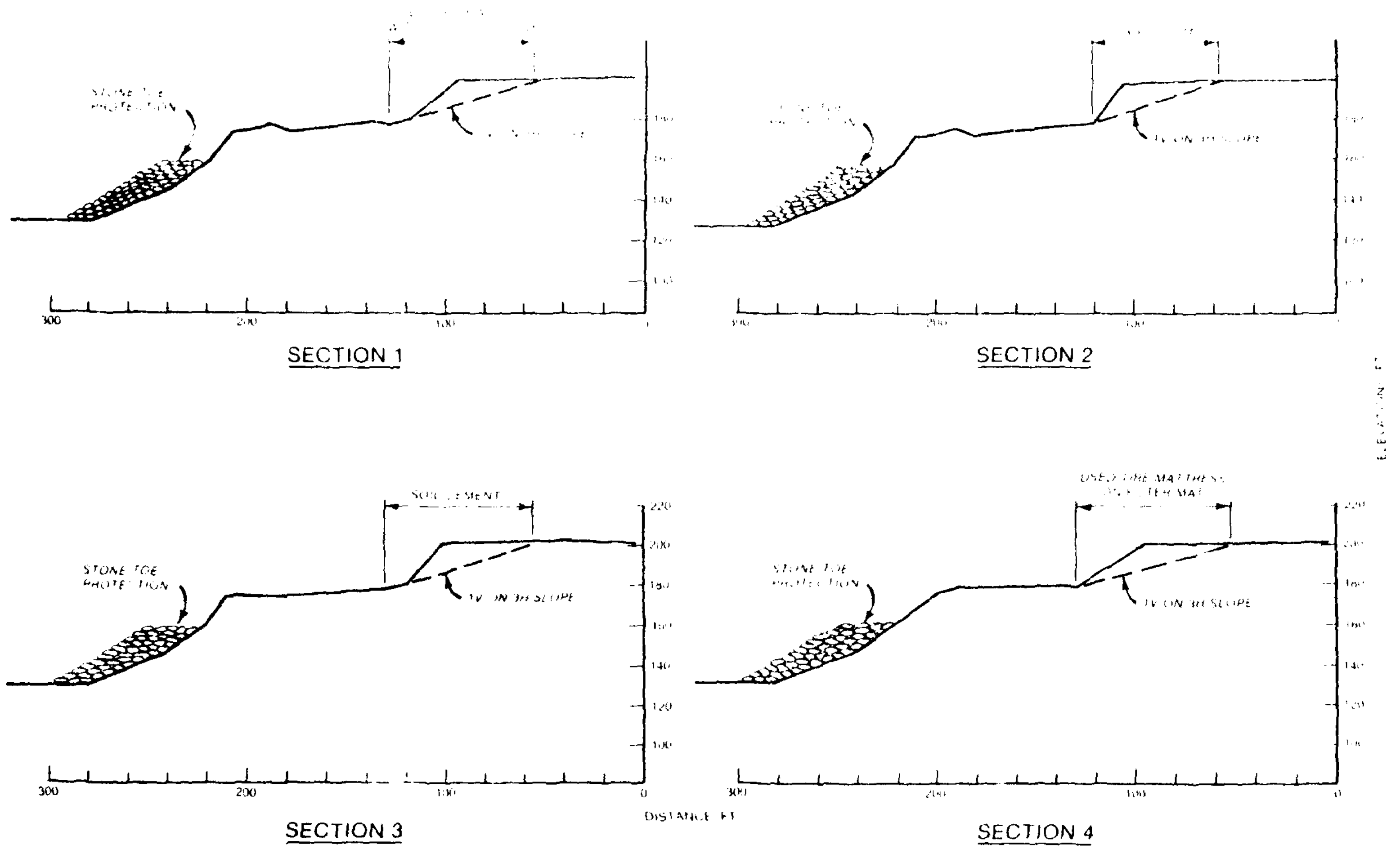
NO SCALE



STEEL JACK UNIT

NO SCALE

IOWA RIVER
WAPELLO, IOWA
PROTECTION PLAN DETAILS



WHITE RIVER
DES ARC, ARKANSAS
PROTECTION SCHEMES