A SUMMARY OF THE RIVER ENVIRONMENT

Prepared for

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FOREWORD

This report was performed under Contract No. 14-16-0003-30,617 titled "A Study of the Geomorphology of the Upper Mississippi River," between the U.S. Fish and Wildlife Service and Colorado State University.

The study was supervised by Mr. William E. Martin, Regional Supervisor, Division of Ecological Services, U.S. Fish and Wildlife Service. Dr. D. B. Simons of Colorado State University was the principal investigator. He was assisted by Dr. P. F. Lagasse, Dr. S. A. Schumm, Dr. Y. H. Chen, and Dr. M. A. Stevens. The period of agreement was from July 1, 1974 to January 31, 1976.

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Chapter 1

INTRODUCTION

1.1 Objectives

This report is supplemental to the reference document, The River Environment* (Simons et al., 1975). The purposes of this summary report are: (1) to provide a basic understanding of the river environment, (2) to serve as an index guide to the basic reference document, and (3) to illustrate the application of current knowledge and techniques to river problems.

1.2 Approach

The report illustrates the use of existing knowledge to analyze river problems. The material presented in The River Environment (TRE) is summarized and referenced.

Chapter 1 establishes the central theme of the summary report. That is, it is an absolute necessity of viewing rivers as dynamic physical/ecological systems, and this chapter outlines the ecological considerations and engineering requirements for river development programs. Research and training needs in the field of river mechanics also are highlighted. The basic concepts of river mechanics and river morphology are presented in Chapter 2. Chapter 3 briefly presents the techniques of river training and development works with emphasis on river modification using dikes and dredging. Response of the Mississippi River above Cairo, Illinois to development is briefly discussed in Chapter 4. Numerical examples to illustrate the application of

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*Simons, D. B., Lagasse, P. F., Chen, Y. H., and Shumm, S. A., "The River Environment," Colorado State University, Technical Report for the U.S. Fish and Wildlife Service, Twin Cities, Minnesota, 1975.

hydraulic and geomorphic concepts and theories to analyze a river system are presented in Chapter 5. Chapter 6 summarizes the data necessary to complete an analysis of the response of river systems to development, and it tabulates primary data sources.

1.3 Rivers as Dynamic Systems

An alluvial river continually changes its position and shape as a consequence of the hydraulic forces acting on its bed and banks. The river is further affected by biological forces that interact with these physical forces. Changes in a river may be slow or rapid and may result from natural environmental changes or from changes caused by man's activities. When a river channel is modified locally, the change frequently causes modifications of channel characteristics both up and down the river. The response of a river to natural and man-induced changes often occurs in spite of attempts to keep the anticipated response under control.

Rivers are dynamic and natural and man-induced changes frequently set in motion responses that can be propagated for long distances. Successful river development requires an understanding of the forces rivers are subjected to. It is necessary that river system analysis should consider knowledge of: (1) geologic factors, including soil conditions; (2) hydrologic factors, including possible changes in flows, runoff, and the hydrologic effects of changes in land use; (3) geometric characteristics of the stream, including the probable geometric alterations that may be activated by changes imposed on the channel; (4) hydraulic characteristics such as depth, slope, and velocity of streams and changes that may be expected in these characteristics in space and time; and (5) ecological/biological changes that will result from physical change and in turn will induce or modify physical changes. Therefore, in order to work with river engineering problems it is useful to utilize knowledge of: hydrology, hydraulics, erosion and sedimentation, river mechanics, soil mechanics, structures, economics, ecology, and related subjects.

Evidence from several sources demonstrates that most alluvial rivers are not static in their natural state (TRE 1.3.1):*

(1) Archaeological evidence: The number of archaeologic sites on floodplains decreases significantly with age simply because, as floodplains are modified by river migration, the earliest sites have been destroyed. Using such evidence, Lathrop (1968)** working on the Rio Ucayali in the Amazon headwaters of Peru estimates that on the average a meander loop begins to form and cut off in 5000 years. These loops have an amplitude of two to six miles and an average rate of meander growth of approximately 40 feet per year.

(2) Botanical evidence: A study of floodplain vegetation and the distribution of trees in different age groups led Everitt (1968) to the conclusion that about half of the Little Missouri River floodplain in western North Dakota was reworked in 69 years.

(3) Geologic evidence: Movement of the earth's crust is an important geologic agent causing modern river instability. The earth's surface in many parts of the world is undergoing measurable change by upwarping, subsidence or lateral displacement. For example, Schumm (1963) estimates that the San Andreas fault in California is moving at an average

* Section 1.3.1 in The River Environment.

** The references are given in Appenix A, TRE.

rate of 0.3 inch per year or three inches per decade. For many river systems, particularly large rivers, a change of slope of three inches would be very significant. (The slope of the energy gradient on the Lower Mississippi River is about three to six inches per mile.)

(4) Geomorphic evidence: A slow but implacable shift of a river channel is caused by erosion and deposition at bends, the lateral movement of channels forms chutes and islands, and such activities as cutoffs of meandering bends form oxbow lakes. In Figure 1-1, a section of the Mississippi River near Commerce, Missouri as it appeared in 1884 is compared with the same section as observed in 1968. In the lower six miles of this reach, the surface area has been reduced approximately 80 percent during this 84-year period. Some of this change has been natural and some has resulted from river development work. Additional evidence of river instability is (a) the Mississippi

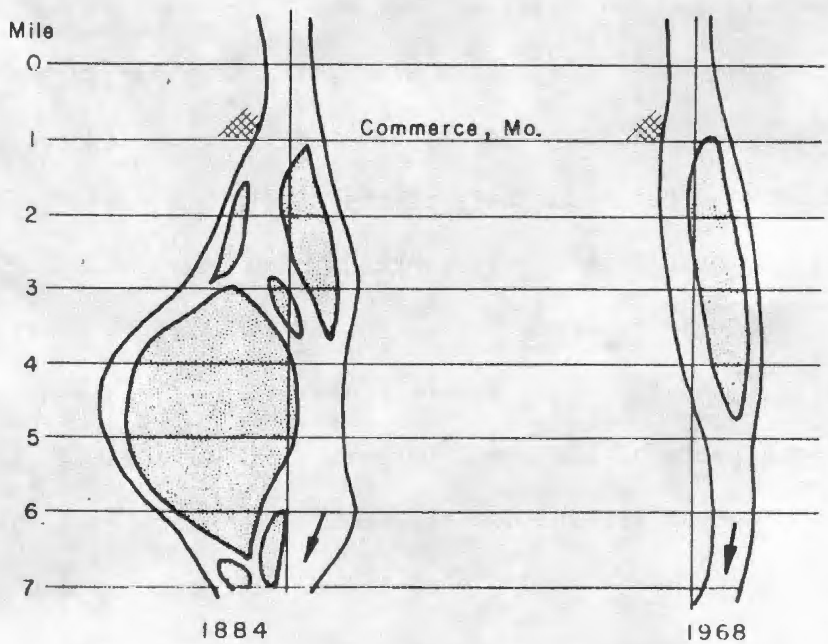

Figure 1-1 Comparison of the 1884 and 1968 Mississippi River Channel near Commerce, Missouri (Figure 1-1, TRE)

River near Rosedale, Mississippi. The river is migrating laterally 158 to 630 feet per year (Leopold and Wolman, 1957); (b) the Cimarron River of southwestern Kansas changed its width from 50 feet during the early 1900's to 1200 feet during the 1930's following a series of major floods, and then reduced its width to 500 feet in 1960 (Schumm and Lichty, 1963).

In summary, archaeological, botanical, geologic, and geomorphic evidence supports the conclusion that most rivers are subject to constant change as a normal part of their morphologic evolution. Stable or static channels are the exception in nature.

1.4 Environmental Considerations

River modification decisions should be made only after the potential environmental impacts have been completely identified and evaluated (TRE 1.4). This means that all feasible alternatives must be considered and the resulting environmental impacts must be understood and compared. Accordingly, an inventory of existing environmental resources (social, economic, and natural) is mandatory. Among other things, the inventory should identify areas of unique vegetation, wildlife and aquatic life, endangered or threatened species, etc. In order to place these factors in proper perspective all contributing disciplines must be considered to identify the interdependence and interrelationship between the life forms and to predict ecological response to stresses caused by exercising various alternatives. In addition, interdisciplinary studies are required to understand the social and economic impact upon the environment of the various project alternatives.

Once the impacts are understood for each of the project alternatives, cost estimates for the viable alternatives can be developed to include enhancement, mitigation, and compensation features that should be built into the project proposal. Finally, a preferred solution is identified and selected that will allow the project to proceed with the least damage and greatest benefit to the total environment.

1.5 Research and Training Needs

As knowledge of river hydraulics is reviewed, it is apparent that the current knowledge is available to cope with the majority of river problems. However, further research is required on the subjects that are summarized in TRE 1.6.2, and the research results should be presented in such a form that they can be easily understood and put to practical use.

On the other hand, the number of individuals who are cognizant of existing theory and can apply it successfully to the solution of river problems is limited. Particularly, the number of individuals involved in the actual solution of applied river mechanics problems is very small. There is a specific reason for this deficit of trained personnel. Undergraduate engineering educators in the universities in the United States, and in the world for that matter, devote only a small amount of time to teaching hydrology, river mechanics, channel stabilization, fluvial geomorphology, and related problems. It is not possible to obtain adequate training in these important topics except at the graduate level and only a limited number of universities and institutions offer the required training in these subject areas. There is a great need at this time to train people to cope with river problems.

The training of individuals could be accomplished by conducting seminars, conferences or short courses in institutions in the spirit of continuing education. Manuals, handbooks and reference documents should be prepared. Better use of informative films could be made. Television and videotapes can be economically prepared and utilized in instructional situations. Formal classroom training should be supported with field trips and laboratory demonstrations. Finally, larger numbers of disciplines should be involved in the training programs. Cooperative studies should involve research personnel, practicing engineers and people from the many different disciplines with an interest in rivers.

Chapter 2

BASIC CONCEPTS

2.1 Introduction

The basic theories on river mechanics and river morphology are summarized in this chapter.

2.2 The Hydraulics of Open Channel Flow

Flow conditions in open channels are complicated by the fact that the position of the free surface and the channel geometry are likely to change with respect to time and space, and also by the fact that the depth of flow, the velocity, the discharge, and the slope of the channel bottom and of the water surface are interdependent. A fundamental open channel problem is to determine the flow line (water surface profile) in a channel for a given discharge, i.e., to determine the flow velocity and depth. This information enables us to assess the available navigation depth and dredging requirements for maintaining adequate navigation channels, to design flood protection works, to evaluate vegetation, fish and wildlife habitats, and to plan land use and other related activities in the river basin. Natural or man-induced activities may alter the stage (water surface elevation) -discharge relationship and this in turn affects the river environment. Determination of the flow line requires an understanding of physical characteristics of open channel flow. The basic theories needed for the analysis of open channel hydraulic problems are briefly discussed.

Open channel flow can be: uniform or nonuniform flow, steady or unsteady flow, laminar or turbulent flow, and tranquil or rapid flow at the same time. In uniform flow, the depth and discharge remain constant with respect to space. Also the velocity at a given depth is the same everywhere along the channel. In steady flow, no change in discharge occurs with respect to time. In laminar flow, the flow field can be characterized by layers of fluid, one layer not mixing with its adjacent ones. Turbulent flow on the other hand is characterized by random fluid motion and rapid mixing of adjacent elements of the flow. Tranquil flow is distinguished from rapid flow by a dimensionless number called the Froude number (F_r) defined by the ratio of inertia forces to gravitational forces in the system or it may be defined by the ratio of the flow velocity to the velocity of a small surface gravity wave in the flow. If $F_r < 1$, the flow is tranquil and surface waves propagate upstream as well as downstream. If $F_r > 1$, the flow is rapid and surface waves can propagate only in the downstream direction. If $F_r = 1.0$, the flow is critical and upstream oriented surface disturbances remain nearly stationary in the flow. These terms, uniform or nonuniform, steady or unsteady, laminar or turbulent, rapid or tranquil, and the two dimensionless numbers (the Froude number and the Reynolds number) are more fully explained in TRE 2.1.2. In general, the flow in rivers is nonuniform, turbulent and tranquil. The river flow may be steady within a short period but unsteady within a long period of time. The analysis of steady, open channel flow will be briefly discussed. The hydraulic analysis is extended to include unsteady flow in TRE 2.4.4.

The basic flow equations that relate flow velocity, depth, discharge and slope are: (1) the conservation of mass (continuity equation), (2) the conservation of linear momentum (momentum equation), and (3) the conservation of energy (energy equation) (TRE 2.1.3).

The principle of conservation of mass states that matter can be neither created nor destroyed. For steady flow, if the mass density is constant (incompressible flow) then the flow discharge at each section in a river reach is identical (TRE 2.1.3.1).

The conservation of momentum is based on Newton's Second Law of Motion applied to the fluid, which states that the fluid acted upon by a resultant force receives an acceleration in the direction of the force that is proportional to the force and inversely proportional to the mass of the particle. The momentum equation thus derived states that the summation of all external forces (pressure force, body force and shear force) acting on the fluid is equal to the rate of efflux of momentum through the boundary of fluid (TRE 2.1.3.2).

Energy can exist in many forms (light, heat, chemical, mechanical, etc.) and every process involves conservation of energy from one form to another. The law of conservation of energy (First Law of Thermodynamics) states that energy can neither be created nor destroyed. This leads to the energy equation. For steady flow the total amount of energy (kinematic energy, pressure energy and potential energy) at any point in a stream is equal to the sum of the total energy at any downstream point and the loss of energy between the two points (TRE 2.1.3.3).

The continuity equation, momentum equation and energy equation can be used to determine the elevation of the water surface in an open channel for a given discharge. For steady flow, the momentum or the energy equation is rearranged in the following form to compute the water surface profile (TRE 2.1.6.3):

$$\Delta L = \frac{H_2 - H_1}{S_o - S_f} \qquad (2.1)$$

where ΔL* is the distance between Sections 1 and 2, H is the specific head, S_o is the bed slope and S_f is the friction slope or energy slope.

The <u>specific head</u> is defined by

$$H = \frac{V^2}{2g} + y \tag{2.2}$$

where V is the average velocity, g is the acceleration of gravity and y is the flow depth. When y is plotted as a function of H for a given unit discharge q, a specific head diagram (Figure 2-16, TRE) is obtained. There are two possible depths called alternate depths for any H larger than a specific minimum. The single depth of flow at the minimum specific head is called the <u>critical depth</u> y_c and the corresponding velocity the <u>critical velocity</u> $V_c = q/y_c$. The flow at minimum specific energy has a Froude number $(F_r = V/\sqrt{gy})$ equal to 1, which corresponds to the critical flow condition. This specific head diagram is very useful in estimating changes in water surface profiles resulting from changes in channel geometry.

The <u>friction slope</u> in Equation (2.1) is the slope of the energy line representing the elevation of the total energy of flow. The slope is related to the velocity and depth by the following empirical realtions (TRE 2.1.4.3):

(1) <u>Manning equation</u>

$$V = \frac{1.486}{n} R^{2/3} S_f^{1/2} \tag{2.3}$$

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*All notations are defined in the LIST OF SYMBOLS, TRE.

(2) Chezy equation

$$V = C R^{1/2} S_f^{1/2} \tag{2.4}$$

where R is the hydraulic radius which is equal to the cross-sectional area divided by the wetted perimeter. The Manning's roughness coefficient n and the Chezy's discharge coefficient C are empirical coefficients representing channel roughness. These coefficients can be estimated based on knowledge of the general nature of the channel boundaries. A catalog of values of Manning's n and Chezy's C is given in Table 2.1 (TRE), Barnes (1967) and Chow (1959).*

If the flow is uniform or nearly uniform the friction slope can be assumed equal to mean riverbed slope. This value can be used in Equation (2.3) or (2.4) to estimate the flow velocity and depth for a given discharge. The flow depth thus computed is known as the normal depth.

If the flow is nonuniform because of changes in slope of the bed, changes in the cross section (such as caused by construction of dikes), obstructions in flow (such as locks and dams), or imbalances between gravitational forces accelerating the flow and shear forces retarding the flow, the flow depth will be different from the normal depth and it can be computed from Equation (2.1). Examples of uses of Equations (2.1) and (2.3) to determine average flow velocity, depth and water surface profiles are given in Chapter 5.

Another method of determining average flow velocity and depth is by using the theoretical velocity equations. Use of these equations eliminates the necessity of estimating the empirical resistance

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*The references are given in Appendix A (TRE).

coefficients. One such well-known velocity equation is presented by Einstein (1950) (TRE 2.1.4.2).

Owing to the presence of a free surface and to the friction along the channel wall, the velocities in a channel are not uniformly distributed in the channel section. The shape of the section, the roughness and the presence of bends affect the velocity distribution. The measured maximum velocity in open channels usually occurs below the free surface at a distance of 0.05 to 0.25 of the depth; the closer to the banks, the greater the distance to the maximum velocity. In a wide, rapid and shallow stream or in a smooth channel, the maximum velocity may occur at the free surface. Then the velocity distribution, with respect to depth, can be approximated by the Karman-Prandtl velocity distribution (TRE 2.1.4.2) or by a power function.

In general, a river does not follow a straight course but consists of bends and crossings. The change in flow direction around bends produces centrifugal forces that cause a higher water elevation adjacent to the concave bank than adjacent to the convex bank. The total superelevation between the outer and inner banks is (TRE 2.1.5)

$$\Delta Z = Z_o - Z_i = \frac{V^2}{g r_c}(r_o - r_i) \tag{2.5}$$

provided that the velocity distribution at the cross section is uniform. The terms r_i, r_c and r_o are the radius at the inside, the center and the outside of the bend respectively; and Z_i and Z_o are the water surface elevations at the inside and at the outside of the bend. This superelevation in bends produces a transverse velocity distribution which results from an imbalance of radial forces on a fluid partical as it travels around the bend.

2.3 Flow in Alluvial Channels

Most streams flow on sandbeds for the greater part of their length and nearly all large rivers have sandbeds. In sandbed rivers, the interaction between the flow of the water-sediment mixture and the sandbed creates different bed configurations which change the resistance to flow, the velocity and rate of sediment transport. The bed forms that may occur in a sandbed alluvial channel are plane bed without sediment movement, ripples, ripples on dunes, dunes, plane bed with sediment movement, antidunes, and chutes and pools (TRE 2.2.3). Typical bed forms are shown in Figure 2-1. These bed configurations are listed as they occur sequentially with increasing values of stream power (τV).

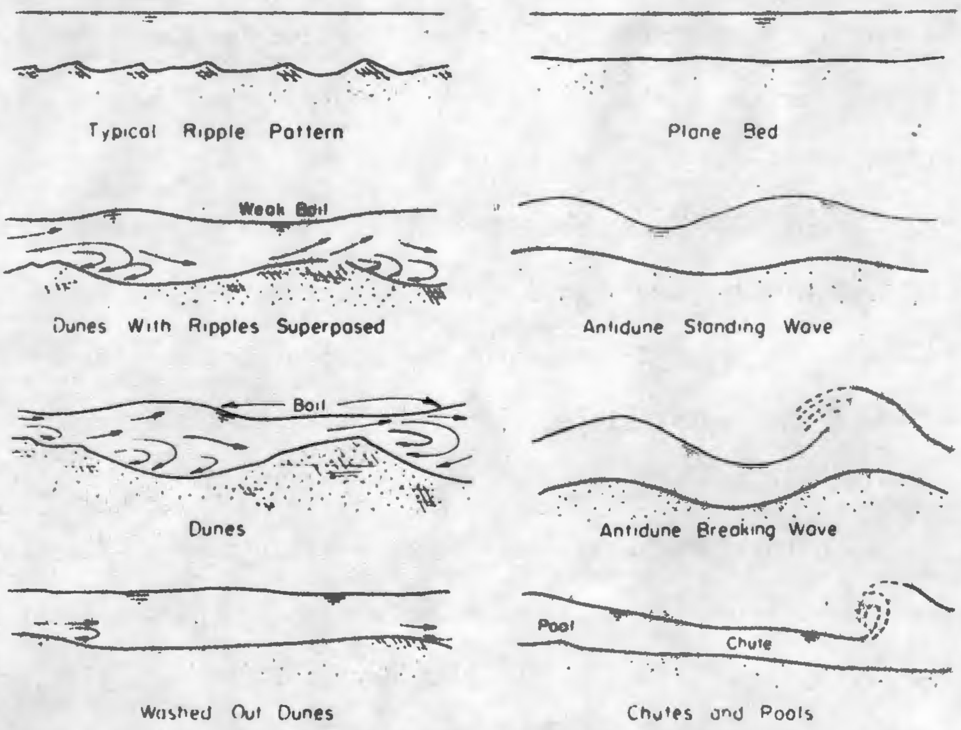


Figure 2-1 Forms of Bed Roughness in Sand Channels (Figure 2-21, TRE).

Using differences in these bed forms as a basic criteria, flow in alluvial channels is divided into two regimes of flow separated by a transition zone (TRE 2.2.3.3). These two regimes and their associated bed forms are: (1) lower flow regime (ripples, ripples superposed on dunes, and dunes) and (2) upper flow regime (plane bed with sediment movement, antidunes, and chutes and pools). In the lower flow regime resistance to flow is high, velocity is small and sediment transport is small. In the upper flow regime resistance to flow is low, velocity is large and sediment transport is large. As the bed configurations change through lower regime to upper regime, Manning's "n" changes from a typical value of 0.014 for plane bed without sediment motion to values as high as 0.04 for a dune bed. Increasing the stream power so that upper regime plane bed conditions develop can produce a decrease in Manning's roughness coefficient to values as low as 0.010 to 0.015. Further increases in the stream power produce an antidune flow and this slightly increases the value of Manning's n.

Simons and Richardson (1966) developed a graphical relation that utilizes stream power, median fall diameter, and bed form. The relation (Figure 2-2) indicates the form of bed roughness to be anticipated if the depth, slope, velocity, and fall diameter of the bed material are known.

In natural channels, bars are generally found. They are bed forms having lengths of the same order as the channel width or greater, and heights comparable to the mean depth of the flow (TRE 2.2.3.4). Several different types of bars occur. They are classified as (1) Point bars, (2) Alternate bars, (3) Transverse bars, and (4) Tributary bars. Bars may appear as small barren islands during low flow,

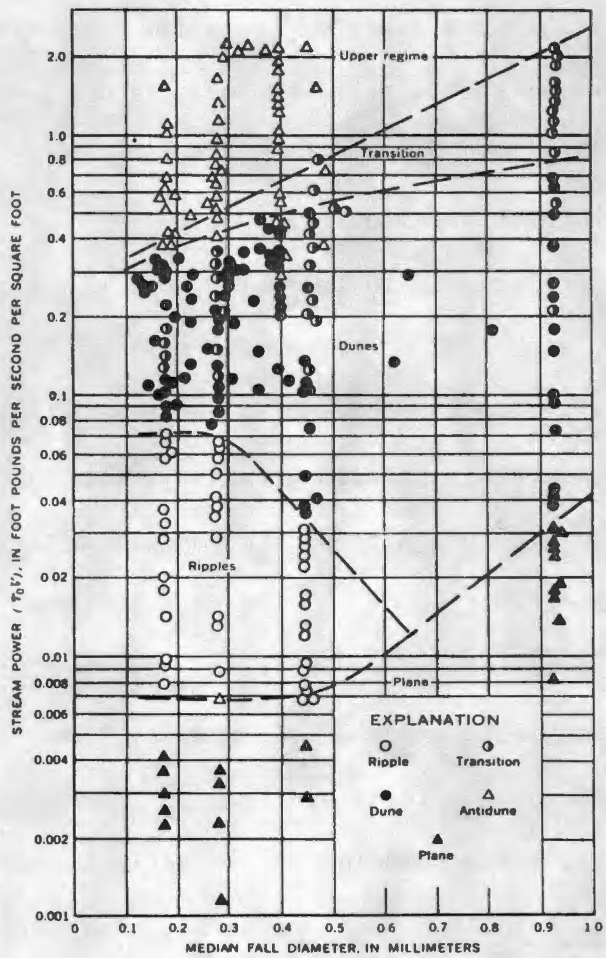


Figure 2-2 Relation of Stream Power and Median Fall Diameter to Form of Bed Roughness (Figure 2-23, TRE).

particularly if the width-depth ratio is large. Portions of the upstream slopes of bars are often covered with ripples or dunes and may be armored with gravel.

The flow in sandbed rivers usually transports sediment the size of which varies with the energy of the system. Every sediment particle which passes a particular cross section of the stream must satisfy two conditions: (1) it must have been eroded somewhere in the river basin above the cross section; (2) it must be transported by the flow from the place of erosion to the cross section. Each of these two conditions may limit the sediment rate at the cross section, depending on the relative magnitude of the two controls: the availability of the material in the river basin and the transporting ability of the stream (TRE 2.3). In most streams the finer part of the load, i.e., the part which the flow can easily carry in large quantities, is limited by its availability in the watershed. This part of the load is designated as wash load. The coarser part of the load, i.e., the part which is more difficult to move by flowing water moves at a limited rate that depends on the transporting ability of the flow between the source and the section. This part of the load is designated as bed-material load. The characteristics of the bed material are closely related to those of the bed material load. The total sediment load of a stream is the sum of the wash load and the bed-material load.

Sediment particles are transported as bed load (this material moves by rolling, sliding and saltation along the bed) and/or as suspended load (suspended material is supported by flow for an appreciable length of time)(TRE 2.3.2).

The determination of sediment discharge is mainly based on field measurement supplemented by theoretical methods developed to compute the unmeasured load, or it is determined by sediment transport functions that relate the sediment discharge to the river hydraulics. Numerous methods have been developed such as by Meyer-Peter and Muller (Sheppard, 1960), Einstein (1950) and Colby (1964) for determining mainly the bed-material discharge (TRE 2.3.3.1, 2.3.3.2, and 2.3.3.3). None have been thoroughly tested with field data. The suitability of these methods should be verified with field data when possible.

2.4 River Morphology

The configuration of the river as viewed on a map or from the air may include straight, meandering and braided channel patterns (TRE 3.2). Typical river patterns are shown in Figure 2-3.

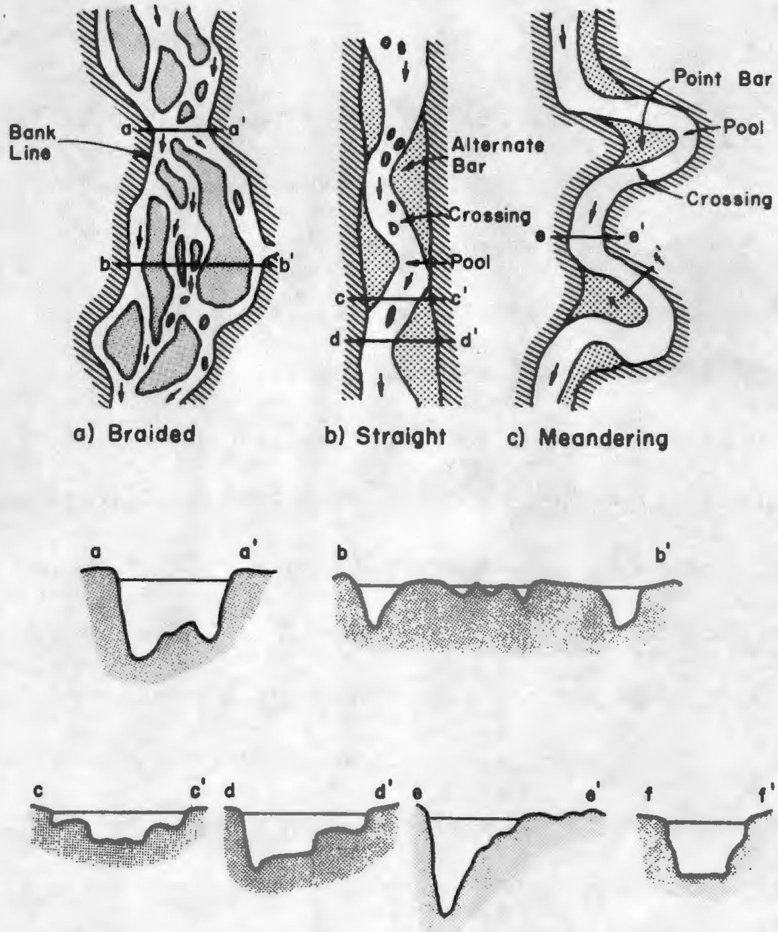


Figure 2-3 River Channel Patterns (Figure 3-1, TRE).

Reaches of a river that are relatively straight over a long distance are generally classed as unstable, as are divided flow reaches and those in which bends are migrating rapidly (TRE 3.2.1).

The braided river is difficult to work with in that it is unstable, changes its alignment rapidly, carries large quantities of sediment, is wide and shallow even at flood flow and is unpredictable. The main causes of the braided condition are an overload of sediment, steep slope and/or easily eroded banks (TRE 3.2.2).

The meandering river consists of a series of deep pools in the bends and shallow crossings in the short straight reach connecting the bends. This type of river is comparatively stable and can be developed and controlled at less cost (TRE 3.2.3).

Alluvial channels of all types deplore a straight alignment. The process of channel pattern development is given in TRE 3.2.4. Some important features involved in river and river valley development are the floodplain and delta (TRE 3.1.3), alluvial fans (TRE 3.1.4), natural levees and back swamps (TRE 3.2.5), and side channels (TRE 3.5.2). The main factors affecting channel patterns are hydraulic parameters such as channel slope S, discharge Q, and the characteristics of the bed and bank. Lane (1957) found that when $SQ^{1/4} \leq 0.0017$ most sandbed channels meander (TRE 3.2.6). Similarly, when $SQ^{1/4} \geq 0.01$ most rivers are braided. The region between these values of $SQ^{1/4}$ can be considered a transitional range where streams are classified as in transition. Many rivers of the United States fall in this transition category. If a river is meandering, but with a discharge and slope that borders on transitional, a relatively small increase in channel slope could initiate development of a transitional or braided character. The result may be a major setback considering river stabilization, river development and the river environment.

Because prediction of river behavior is necessary, methods have been developed to help establish the response of channel systems to changing conditions and/or development.

A very useful relation for predicting river system response is

$$QS \sim Q_s D_{50} \qquad (2.6)$$

where Q_s is the sediment discharge and D_{50} is the median diameter of sediment particle for which 50 percent of the material is finer. Applications of this equation for qualitative prediction of channel response to natural or imposed changes are given in Section 5.5.3 and in TRE 3.3.2, 3.3.4, and 5.2.

In addition to the long-term morphological changes of rivers created by system modification, short-term periodic effects such as variation of stage are imposed on the crossings and pools (TRE 3.4.2). The crossing and pool sequence is common to both meandering reaches and straight reaches (Figure 2-3). Each has a thalweg (the deepest part of a riverbed) that meanders through a system of alternate bars. The characteristics of the crossing and pools are described in TRE 3.4.1.

Changes in stage radically alter the morphology of the crossing and pool sequence by changing the direction of flow of both water and sediment and by modifying deposition and scour patterns. At low stage the thalweg impinges on the concave bank of the bendway. The higher velocities and greater momentum of the high-stage flow tend to "short circuit" the meander pattern. The high stage thalweg skirts the convex bank and cuts across the tip of the point bar, opening, in some cases, a chute channel across the bar (Figure 2-4). Moreover, at high flow, the crossing tends to fill with sediment and the pool tends to scour because of the larger velocity and steeper slope through the pool. At low flow the processes are reversed. Because of the greater sediment activity during high flow, low flow scour on a crossing (further

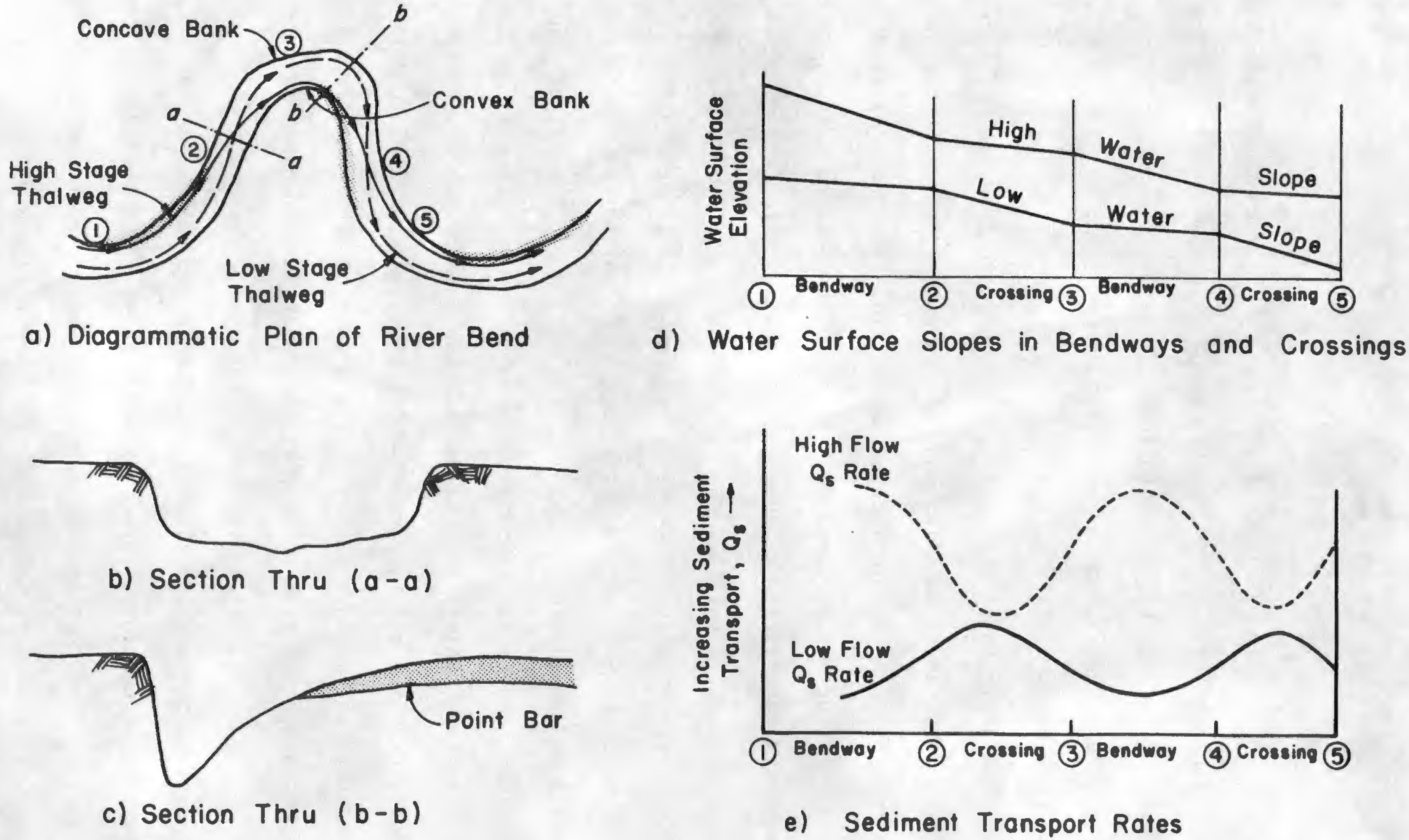


Figure 2-4 Characteristics of a River Bendway (Figure 3-16, TRE).

limited by armoring) is generally not sufficient to remove the material deposited during high-stage flows.

Because of the influence of stage on channel morphology, an alluvial river will behave differently for different river flows. This dual personality of alluvial rivers poses serious problems for development programs such as channel maintenance for navigation. The system of channels formed by low flow conditions at low-stage are altered or destroyed by the flows at high-stage. As stage falls again, the low-flow stream must reform its preferred alignment through a maze of islands, bars, and channels that do not conform to the alignment preferred by reduced flows.

2.5 Modeling of Rivers

Many complex problems in hydraulic engineering can be solved by the use of models which duplicate the complicated geometry and in which the resulting flow patterns can be observed directly (TRE 2.4). The models may be physical models; that is, small-scale physical replications. Also, they may be mathematical models consisting of mathematical abstractions of the physical phenomena. The boundaries of models may be rigid or mobile. Models are used to test the performance of a design or to study the details of a phenomenon such as propagation of a flood and river response to river development. The performance test of proposed structures can be made at moderate costs and small risks on models. Similarly, the interaction of a structure and the river environment can be studied in detail.

A model study of a river system starts with a decision as to type of model (physical model with rigid or movable boundary or mathematical model), model scale, phenomena to be modeled, and time and cost required to accomplish the objectives of the model study. The physical model of free surface flow is constructed based on the Froude model law. That is, the Froude number of the prototype must equal that of the physical model to achieve the prototype-model similarity. For mathematical models the equations that govern the flow phenomena are generally solved by using numerical methods and a computer. After construction of the model, it is necessary to calibrate the model to assure that it simulates prototype behavior. The model construction and calibration procedures are described in TRE 2.4.2, 2.4.3 and 2.4.4. The model is then operated to achieve the objectives of the model study. The model results are documented, analyzed and interpreted. Performance of alternative systems can also be tested. By comparing the performance of different systems, the most suitable system can be selected.

Quite often, a model is subjected to scale effects resulting from overreduction of prototype dimensions or from inadequacy of mathematical representation of the physical phenomena that changes the behavior of dominating parameters. The significance of scale effects on the similarity between prototype and model should be carefully evaluated. Proper action should be made to reduce scale effects to an acceptable level.

A comparison of physical models and mathematical models reveals that physical models are best for detailed studies of short reaches of river, but not for long reaches because of the model scale and the space limitation of model facilities. The physical model is also expensive to operate. Mathematical models can be applied to a general study of a long or short reach, and are relatively cheap to operate. However, it is difficult to utilize the mathematical model to study local phenomena that cannot be properly described by mathematical relations.

Chapter 3

RIVER TRAINING AND DEVELOPMENT WORKS

3.1 Introduction

In Section 3.2, purposes of the channel stabilization, the various types and uses of stabilization works, and guides for selecting methods and devices for river stabilization are summarized. In Section 3.3, the role and methods of dredging, applications of dredging, hydraulics of dredged cuts, and dredged material disposal problems are discussed.

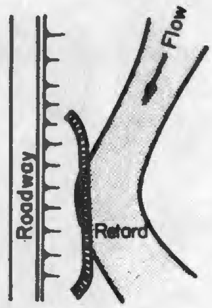
3.2 Channel Stabilization

Basically, there are three fundamental reasons for employing channel stabilization works: 1. protection, i.e., protection of property from erosion and floods, 2. improvement of channels for navigation, and 3. enhancement of the environment. For protection from erosion and floods, channel realignment, revetments, dikes, groins, levees, retards, and bankheads are employed. For navigation purposes similar measures and structures are used to provide improved channel alignment and improved channel dimensions, and to deepen the shallow crossings. To improve the river environment and adjacent wetlands similar and/or special structures may be used to accomplish specific objectives. The types and uses of stabilization works are summarized. An extensive treatise on the subject of bank and shore protection was prepared by the California Division of Highway (1970). Lindner (1969) also provides an excellent summary of the objectives and common methods of regulating and stabilizing alluvial channels. However, environmental considerations are lacking.

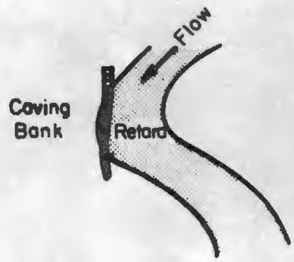
Various devices and structures have been developed to control river flow along a preselected path and to stabilize the banks. Most have been developed through a trial and error process, aided in some instances by hydraulic model studies. Dikes, retards, and jetties are the three types of devices most commonly used for river training.

In general dikes extend outward from the bank into the channel at right angles or they are angled thereto, depending upon the circumstances and particular success achieved in past experiences (TRE 6.2.3.1). There are two principal types of dikes, permeable and impermeable. Permeable dikes permit flow through the dikes but at reduced velocities, thereby preventing further erosion of the banks, encouraging deposition of suspended sediment particularly at low flow. Also, they guide the flow along a desirable path. Impermeable dikes deflect the flow away from the bank, train the flow and protect the bank. Jetties form a jetty field along the river banks to add roughness to a channel or overbank area to train the main stream along a selected path and to protect the banks from excessive erosion (TRE 6.2.3.2). Retards are permeable devices placed parallel to embankments and river banks to decrease the stream velocities and prevent erosion (TRE 6.2.3.3). The use of dikes, jetties and retards is illustrated in Figure 3-1.

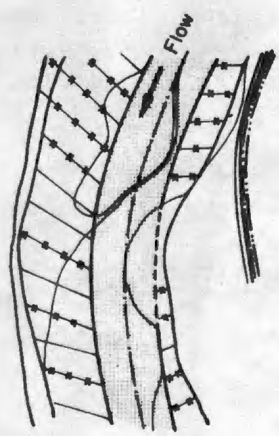
The type of channel improvement and devices used for training and bank stabilization depend upon river width, depth and discharge; type of river, that is, meandering, braided or straight; length of river to be protected; availability of construction materials; environmental considerations; aesthetics; legal aspects; river use with regard to navigation, recreation, agriculture, municipal and industrial purposes; and perhaps other factors. Table 6-1 (TRE) is presented as a guide to assist in considering alternatives and formulating decisions for channel improvement and selection of type of bank protection and river training



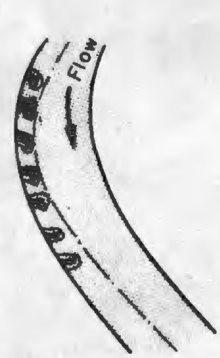
(a) Retards to protect highway embankment



(b) Retard to prevent further bank caving



(c) Jetties to train the flow and protect the bank



(d) Dikes to train the flow and protect the bank

Figure 3-1 Retards, Jetties and Dikes to Protect Embankments and Train Channel Flow (Figure 6-2, TRE).

works. Some applications of stabilization works are presented in TRE 6.2.5.

The stabilization works cause changes in both river geomorphology and hydraulics. These changes should be assessed before and after the construction of stabilization works to identify and determine the environmental impacts and to improve our knowledges of effects of structures on river response. These experiences aided by physical or mathematical model studies are very useful for planning and decision making considering other similar projects. This procedure helps maximize positive impacts and minimize negative impacts caused by modification of a river system. The basic concepts of physical and mathematical modeling are presented in TRE 6.3 and 6.4.4 respectively to indicate river response to dike construction.

3.3 Dredging

Dredging is defined as a process by which undesirable sediments are removed from the bottom of streams, lakes, and coastal waters, transported by ship, barge, or pipeline, and discharged in open water or on land.

The dredging devices which have been used on the Mississippi River can be grouped into: stirring and scraping devices, current deflectors, mechanical devices, jets, and the suction dredge. The dredge development on the Mississippi River is given in TRE 7.2.1. Currently, two types of dredges are used for maintenance and improvement dredging in the riverine environment of the Mississippi: the cutterhead hydraulic dredge and the dustpan hydraulic dredge. The dredge devices and their operations are described in TRE 7.2.2 and 7.2.3.

Basically dredging is applied in three situations: development dredging, gravel mining and maintenance dredging.

Development dredging provides a means of rapidly altering channel configuration and morphologic processes in support of river development programs. Examples of development dredging are provided by the Arkansas River project and the Mississippi River project (TRE 7.4.1). In the Arkansas River, dredging was used to develop the navigation channel just downstream for each lock and dam to accelerate the anticipated natural degradation. In the Lower Mississippi River, dredging was used between

1929 and 1945 to greatly accelerate the cutoff process. The result was a 96.7 mile decrease of distance between the mouth of the White River and of the Red River.

Gravel mining dredging is application of dredging in rivers to obtain gravel for construction and related uses. This activity may significantly influence a river system. The impact is closely related to the role played by the coarser fraction of the bed material in controlling sediment transport and stabilizing channel patterns and bed forms. Removal of the gravel armor from riverbed and banks can lead to erosion and loss of control. As a result, meandering reaches may tend to develop a braided character, velocity and bed-material transport may increase, and localized changes may contribute to the deterioration of adjacent reaches. Some examples of gravel mining in the Mississippi River and its associate impacts are discussed in TRE 7.4.2.

Dredging in many instances is applied to maintain a desirable hydraulic system. Quite often dredged cuts are filled up in a short time period so that recurrent dredging is required. For those cases maintenance dredging is viewed as a temporay solution which treats the symptoms but not the disease. However, temporary solutions have the advantage of being relatively simple and direct in application, and they are flexible and can meet unexpected or changing requirements.

Maintenance dredging of a channel through a crossing or shoal is considered successful if the dredged cut meets two criteria (TRE 7.3.1). First, the cut should achieve the required depth with a minimum of excavation, and second, the cut should not require redredging during a proper time period. These two criteria are influenced by channel alignment, depth and width of cut. In general, no single alignment will coincide with both the high-stage and low-stage patterns of flow over a crossing (TRE 3.4.2 and 7.3.4). A decision to align the dredged cut with the low-stage pattern of the river accepts in general that the cut is malaligned for high-stage conditions, and thus, cannot be expected to survive more than a few high-flow cycles. Consequently, dredged cuts are considered stable if they provide the required depths during a single season. An analysis of the effect of channel depth on dredged cut stability is given in TRE 7.3.1, and an example is given in TRE 7.3.2.

The volume of material dredged from a reach of river is affected not only by geomorphic or hydraulic parameters but also by funding, dredge plant capability, availability and policies (such as overdredging). A direct cause and effect relationship is difficult to establish. Some efforts have been spent in analyzing dredging records in the Upper and Middle Mississippi River. The analysis reveals a close correlation between trouble spots in the Upper Mississippi River that require frequent dredging with the following factors: location of locks and dams, location of pool primary control point, straight reaches and divided reaches. The modification of a river can reduce dredging requirements (TRE 7.5).

Dredged material disposal may significantly change the riverine environment. Disposal of material on marshland, sloughs and in backwater areas may change the local morphology of the river by reducing discharge, depth and velocity, changing rates of sediment and decreasing dissolved oxygen levels. Disposal may eliminate valuable habitat; change the nature of the substrata; increase turbidity and affect water quality. When dredged material is deposited along an open bankline, it is subjected to severe erosion by wind and water. The eroded material can return to the channel, increasing turbidity and affecting water quality and the disposed material can directly eliminate valuable habitat.

Consequently, deposition practices are considered harmful to the riverine environment when dredged material is deposited (TRE 7.7.1):

1. In such a manner as to cause the filling of chutes and side channels.
2. In or near inlets and outlets between the river and sloughs or backwater areas.
3. On submerged wing dams and closing structures.
4. At the upstream end of the islands.
5. In such a manner that the outwash covers a substantial area of aquatic vegetation in the backwater sloughs and lakes.
6. Without due consideration for established or contemplated public use areas.

When considering geomorphic and hydraulic consequence of open water disposal, the most desirable general location is a region where deposition would occur naturally (TRE 7.7.2). At these locations, the flow has less power to erode and transport the disposed material back to the main channel where it can produce additional damage. However, locations that are desirable based on a hydraulic and geomorphic analysis are quite often unacceptable from a biologic point of view. To avoid detrimental effects on fish and wildlife, recreation, and navigation, the following general recommendations are made considering future disposal sites and other uses of dredged material:

1. Deposit dredged material on existing islands having a low value for timber and wildlife habitat.
2. Consider the construction of sand islands in the lower end of wide, flat pools. The principal advantage of such islands would be (1) reduction of wave action, (2) provide areas for wildlife, (3) better define the navigation channel, and (4) create additional recreation areas.
3. Develop sand beaches at state, county and municipal parks bordering the river.
4. Provide fill for proposed public access and parking areas sponsored by Federal and State programs.
5. Create dikes in large shallow water areas for waterfowl and aquatic fur animal management.
6. Provide fill in lowland areas of little wildlife value adjacent to communities wishing to have additional space for industrial expansion or other purposes.
7. Provide sand and gravel for construction purposes.

Where compromise solutions between physical and biological considerations cannot be found, alternatives to the current practice of open water disposal of dredged material must be sought. A summary of possible alternatives is presented in Figure 7-31 (TRE). An alternative to disposing dredged material in the main channel is discussed in TRE 7.7.3. This alternative is based on the geomorphic and hydraulic characteristics of the river system, which avoids the direct biologic impacts of bankline and island disposal, and at the same time is within the capabilities of existing dredge plants.

A geomorphic analysis of the crossing and pool sequence supported by mathematical modeling of a particularly troublesome reach of Pool 25 on the Upper Mississippi indicates that dredging from a crossing and disposing the dredged material in a downstream pool can constitute a feasible alternative to the disposal problem. The process involves a degree of risk of impacting the integrity of the navigation channel downstream from the pool, particularly if dredging is followed by a dry year with low flow. However, the risks incurred would be outweighed by the potential environmental benefits at many locations. Based on data currently available, the direct biologic impacts of disposal in the main channel as well as possible secondary impacts from turbidity generated by the disposal process appear minimal. In addition, the serious ecological problems associated with open water disposal on marshlands and near chute channels, sloughs, and backwater areas are avoided by the process of thalweg disposal. Although conditions downstream of a proposed disposal site may preclude thalweg disposal at certain locations, in many cases disposing only a portion of the dredged material along the thalweg would still result in reduced environmental impacts. Still another alternative involves temporarily placing the dredged material conveniently along the banks during low flow and then returning it to the main channel during periods of high flow when the river has a large capacity to transport sediment.

Chapter 4

RESPONSE OF THE MISSISSIPPI RIVER TO DEVELOPMENT

4.1 Development Program

The major objectives of developments along the Mississippi River have been to provide flood protection to people and property on the floodplain and to provide sufficient depth of water for commercial transport during low flows.

In the 195-mile reach of river between the mouth of the Missouri River above St. Louis to the mouth of the Ohio River at Cairo, the Mississippi River is known as the Middle Mississippi. Above the mouth of the Missouri the river is called the Upper Mississippi River (Figure 4-1, TRE).

Improvement of the Mississippi River for navigation has been underway for almost 150 years. The first undertaking to improve conditions was to remove snags hazardous to navigation. Later on, dikes were constructed to confine the low flows to a narrower channel thus increasing the depth of flow. The dikes constructed in the Middle Mississippi River were high dikes with crests close to bank-full stage, whereas the dikes in the upper river were relative low structures with crest at a level six feet above the 1864 low water. After 1930 the Corps of Engineers continued utilizing additional contraction dikes and dredging on the Middle Mississippi to achieve a 9-foot channel (TRE 6.4.1). On the Upper Mississippi, however, the basic approach was flow regulation using a system of locks and dams, supplemented by dredging (TRE 5.3.1).

In addition, levees have been used in the Mississippi River basin to protect the people and floodplain property from floods for more than a century.

4.2 Historical River Response to Development

River development for flood protection and navigation has produced a new river morphology and different river behavior. The history of channel positions, riverbed areas, cross-sectional areas, channel bed elevations and the variations in water and sediment discharge, stages, and stage versus discharge indicate how the river behavior has been changing.

4.2.1 Geomorphic Changes

The geomorphic response of the Mississippi River to the combined activities of contraction works, dredging, and construction and operation of locks and dams is summarized below.

An analysis of river position based on a time-sequenced comparison of river banklines leads to the conclusion that the Mississippi between St. Paul and Cairo has not changed its position appreciably in the last 150-200 years (TRE 5.3.2.1). Using the change in river position as an indicator of stability, both the upper and middle rivers are quite stable. In terms of degree of stability, several significant local changes in position on the Middle Mississippi, such as the Kaskaskia cutoff, support the characterization of the Middle Mississippi as somewhat less stable than the Upper Mississippi, but certainly far more stable than parts of the Lower Mississippi.

The use of dikes to create an improved navigation channel produced a slight decrease in river width (distance between the vegetated banks taken normal to the general flow direction) between 1890 and 1930 on the upper river (TRE 5.3.2.2) and a major decrease in width between 1890 and the present on the middle river (TRE 6.4.2.1) because vegetation held on the high dike field but not on the low dike field which was quite often submerged. Although detailed conclusions relative to geomorphic and hydraulic change in specific reaches on the Upper Mississippi subsequent to 1940 require an analysis of the particular pool in question, general trends in Pools 4, 24, 25 and 26 appear reasonably representative of changes on the upper river following lock and dam construction. The immediate response of the river to lock and dam construction was an increase in surface width throughout each pool, however, the long-term response has been a decrease in width immediately below each lock and dam and near the confluence of tributaries carrying heavy sediment load, and an increase in width just above each lock and dam.

The entire Mississippi above Cairo has experienced considerable within-channel change. These changes are reflected in variations in river surface area, island area and riverbed area. (The surface area of a river is between the vegetated riverbanks including the islands. Islands are defined as areas with land-type vegetation that are separated from the mainland by the main channels and side channels. Riverbed area is defined as the surface area less the island areas.) Because the length of the Mississippi above Cairo has not changed appreciably, surface area change has generally mirrored the change in river width. Dike construction on the Middle Mississippi has produced significant decreases in island area and in riverbed area because of attachment of islands to the floodplain. On the Upper Mississippi, again, the response was more complex and was a function of position in a pool. Higher water levels immediately upstream of a lock and dam have produced a decrease in island area, while a lowering of bed elevations downstream from a lock and dam due to degradation has resulted in lower stages and increased island area (TRE 5.3.2.3).

Bed elevations on the Middle Mississippi have degraded throughout the period of construction of high dikes (TRE 6.4.2.3). In a 14-mile reach selected by the Corps of Engineers for detailed study, the riverbed lowered almost 11 feet between 1889 and 1966. This degradation reduced the dredging requirement for maintaining the navigation channel as indicated in Section 4.2.3. The period of construction of low dikes on the reach that includes Pools 24, 25 and 26 (1890-1930) was one of slight aggradation of the riverbed (TRE 5.3.2.4). The limited effectiveness of the low dikes constructed on the upper river, and the concentration of construction effort toward the end of the era of dike construction, coupled with a natural tendency toward aggradation on the Upper Mississippi contributed to this pattern of increasing bed elevations. However, in the Pool 4 reach below the Chippewa, the riverbed degraded in the period 1897-1929 because of increased dike construction. These differences in river response show that riverbed elevation changes depend upon the degree of flow constriction.

The trends of aggradation and degradation were reversed after 1930. In the period between 1930 and 1973, the riverbed degraded in most of Pools 24, 25 and 26 mainly because much of the sediment that would have normally been delivered to the study reach was being trapped in upstream pools. In the reach that includes Pool 4 below the Chippewa, the riverbed aggraded between 1938 and 1972, because Lock and Dam 4 raised the water level of low and medium flows, reduced the flow velocity, and in turn decrease the channel ability to transport sediment delivered from the Chippewa basin. These river responses to construction of locks and dams indicate that dredging activities in the upper pools cause changes in lower pools and that a decrease in sediment inflow to a river reach causes degradation or reduces aggradation in the reach. These findings agree with predictions of Equation (2-6). Local exceptions to these trends are the aggradation immediately above locks and dams and the local scour immediately below them.

Viewing the river in cross section provides an integrated picture of the effects of changes in width, surface area, and bed elevation. In particular, the response to dike construction is clearly evident in the cross-sectional view. The river flow area downstream of St. Louis on the Middle Mississippi has progressively decreased. Now it is only two-thirds that of the natural river (TRE 6.4.2.2). A similar decrease has occurred all along the Middle Mississippi wherever the channel has been contracted with high dikes. On the Upper Mississippi the flow area generally decreased during the period of dike construction and increased following lock and dam construction in response to the changes caused in surface width and bed elevation (TRE 5.3.2.5).

4.2.2 Hydraulic Changes

Geomorphic response of the Mississippi above Cairo to man's activities is reflected in the hydraulic parameters of discharge and stage. Annual peak flood discharges on the upper river have remained, on the average, unchanged through the period of record (TRE 5.3.2.2). On the middle river present day peak floods are, on the average, slightly lower than in the past, reflecting the construction of storage dams on the Missouri River. Minimum flows have increased slightly both above and below the mouth of the Missouri (TRE 6.4.3.2).

The effect on river stage has been more significant. Downstream of St. Louis, the decrease in both flow area and overbank storage has contributed to an increase in the annual maximum flood stage. Although present day floods on the Middle Mississippi produce flood stages higher than similar discharges produced in the past, levees prevent flood damage when the river exceeds bank-full stage. Under natural conditions, flood damage occurred whenever the river exceeded bank-full stage.

On the Upper Mississippi minimum stages have been strongly influenced by river development. In general minimum stages decreased at locations immediately below locks and dams and increased above locks and dams immediately after pools were established. On the average, the annual maximum stages and discharges at Alton and Keoluk have remained unchanged in the last 100 years. This indicates that present day floods on the upper river produce flood stages similar to those of the past.

Sediment data supports the characterization of the Upper Mississippi as a clear water stream and the Mississippi below the Missouri as a heavy sediment carrier (TRE 5.3.3.3). A little over 10 percent of the suspended sediment load at St. Louis is contributed by the Upper Mississippi. While 15 percent of the suspended load at St. Louis is sand, the sediment data indicates that very little sand is moving in suspension in Pools 4, 24, 25 and 26 during periods of low flow.

Sediment records on the Mississippi above Cairo are not of sufficient length to permit an accurate determination of the effect of river development on sediment load. Available data does suggest that sediment loads have been decreasing in recent years. The sediment trapped by the pools of the Upper Mississippi lock and dam system and dredging are certainly factors that have contributed to the observed general degradation in the lower pools of the system.

4.2.3 Maintenance Dredging

The importance of maintenance dredging to the program for extension and improvement of a navigable waterway in the Mississippi River has been recognized in the authorizing legislative action.

In Pools 4, 24, 25 and 26 of the Upper Mississippi the large dredged volumes of the 1933-1938 period can be related primarily to dredging associated with the transition from the 6-foot channel project to the 9-foot channel project, and to the extreme low water of the 1930's. The reduction in dredging in the Pool 24, 25 and 26 reach between 1939 and 1948 can be attributed, in part, to the heavy dredging of the 1930's and to the war years. The increased dredging in Pools 4, 24, 25 and 26 during the 1960's can be partially attributed to a period of unusually high flow and to the policy of overdepth and overwidth dredging (TRE 7.5.1).

To establish a more direct relationship between dredging requirements and geomorphic factors in the Upper Mississippi, the dredging data are reported in terms of dredging frequency and volume by location. The results of analysis indicate that the pools of Locks and Dams 24, 25 and 26 radically altered the distribution of dredging requirements by concentrating dredging at a few trouble spots (TRE 7.5.2). The analysis reveals a close correlation between identified reaches that require dredging in Pools 4, 24, 25 and 26 with the following factors: (1) location of the locks and dams, (2) the location of the primary controls of the pools, (3) major tributaries that carry heavy sediment loads, (4) straight reaches, and (5) divided reaches. The most serious dredging problems occur in straight reaches which are located above the pools primary control point and below a major tributary carrying heavy sediment loads that are divided by alluvial islands.

On the Middle Mississippi River annual dredging quantities have declined from a peak of 8,131,000 cubic yards in 1965 to a minimum of 2,056,000 cubic years in 1973. This decline can be related to effectiveness of contraction works constructed to develop the 9-foot channel. Experience has shown that sufficient contractive effort can eliminate maintenance dredging requirements by inducing sufficient degradation of the riverbed. However, contraction works on the Middle Mississippi have been responsible, in part, for a significant increase in stage at the higher discharges. When permanent improvements such as contraction of a river to obtain navigation depths produce undesirable environmental, hydraulic or geomorphic response, dredging provides an alternate means of maintaining a river for navigation.

4.3 Mathematical Modeling of River Response

A mathematical model for water and sediment routing in rivers is developed by describing the unsteady flow of sediment-laden water with the one-dimensional partial differential equations representing the conservation of mass for sediment, and the conservation of mass and momentum for sediment-laden water (TRE 2.4.4). The model is applied to study the response of rivers to natural and man-induced activities.

4.3.1 Mathematical Modeling of the Upper Mississippi River

A mathematical model of Pools 24, 25 and 26 has been constructed and a mathematical model study of Pool 4 below the Chippewa is being developed to assess future geomorphic changes. What will Pools 4, 24, 25 and 26 (Upper Mississippi, Lower Chippewa and Lower Illinois Rivers) look like 50 years from now? Will the side channels be filled with sediment? Will the riverbed aggrade making the maintenance of the 9-foot channel project more expensive? Are there any viable alternatives to the present-day operations that would enhance the environmental aspects of the pools and at the same time maintain the required navigation channel? The application of a mathematical model to answer these and other questions relative to the Pool 24, 25 and 26 reach is described in TRE 5.4 and is summarized below. A similar study of Pool 4 has been initiated. Impacts on river geomorphology by reducing sediment inflow from the Chippewa to the Pool 4 reach is being studied.

Considering possible future changes in the delivery of water and sediment to the reach that includes Pools 24, 25 and 26, the time sequence in which these amounts are delivered and man's activities within the reach, five major 50-year simulations were conducted to assess future geomorphic changes. They are: (1) continue with the present scheme of operation to maintain the 9-foot channel, (2) increase pool levels 1 foot above normal pool, (3) hold pool levels 1 foot below normal pool, (4) consider zero sediment inflow into Pool 24, and (5) consider maximum sediment inflow into Pool 24. Simulations 1 through 3 were conducted to assess the effects of different operational schemes for the locks and dams on the geomorphology of the study reach during the next 50 years. The effects of other alternative operational schemes can be investigated in a similar way. Simulations 4 and 5 were performed to estimate the effects caused by the upper and lower limits of changes in the delivery of sediment to the study reach. From these two simulations and Simulation 1, the impact of changes in the delivery of sediment to the study reach on the morphology of rivers and adjacent lands in the future was assessed. The effects of the pools on the behavior and form of the Illinois River were also estimated.

On the basis of past geomorphic changes in the river reach and with the mathematical simulation of future river response, it is concluded that the river scene in the Pool 24, 25 and 26 reach 50 years into the future will be essentially as it is today. The present-day manner of operation does not have serious detrimental effects on the geomorphology or hydraulics of the river system in the study reach. Holding the normal pool level one foot higher or lower does not create significant sedimentation problems. The impact of changes in the delivery of sediment to the study reach will not go beyond Lock and Dam 24 for at least 50 years. It is anticipated that future geomorphic changes in other pools would be similar to those predicted in the lower three pools. However, for pools that receive major tributaries that carry heavy sediment load (such as the Pool 4 reach joined by the Chippewa River), the geomorphic changes may be significantly different. In these pools, varying the normal pool level may significantly affect the river geomorphology. Reducing the sediment load delivered from the tributary to the pool may cause degradation and possibly channel instability in the river reach. These changes frequently result in modification of channel characteristics both up and downstream for long distances. A study of the responses of the whole river system to development is thus required.

Other man-induced activities in the study reach such as dredging and dike construction were also considered.

The future geomorphic changes, for longer time periods in the study reach, may be assessed by operating the mathematical model, provided that the input flow discharge hydrographs can be adequately synthesized.

The limitation of the mathematical model relates primarily to its one-dimensional character (longitudinal dimension along the main flow direction). The model was effective in studying the short-term and long-term river responses to development in a long river reach. With the aid of two- or three-dimensional theories of sediment and velocity distributions and the concepts of lateral flows, mathematical models can be employed to assess the impacts of all factors considered in this study. However, since the space increments are chosen to be relatively large to operate the mathematical model efficiently and economically, and since sediment is assumed to be uniformly distributed over the channel width, only the general pattern of the river geomorphology was considered in this study. To study a particular reach of river in detail the reach may be subdivided into a large number of segments to apply the mathematical model (e.g., study of a local dredging problem) or a combination of physical models and mathematical models could be used.

Since there is no width predictor included in this mathematical model, the changes in channel width with time should be considered to be a known quantity or the potential variation of width should be evaluated using qualitative geomorphic concepts.

4.3.2 Mathematical Modeling of Response to Dikes

A multiple-channel model has been developed to represent prototype conditions including a divided flow reach or a braided reach. The multiple channel model provides a better representation of response of a reach contracted by dikes along one bankline than could be achieved with a single channel model. The formulation of a multiple channel model and its application to the problem of river response to contraction works is described in TRE 6.4.4. The model is summarized below.

A hypothetical channel (Figure 6-27, TRE) was utilized to approximate the patterns of erosion and deposition in a meandering stream. The channel reach is divided into Stream 1 and Stream 2 along the channel centerline. The mathematical model routes both water and sediment through the two adjecent channels of the hypothetical reach and at the same time accounts for an interchange of the water and sediment between the two streams by utilizing appropriate lateral inflow or outflow parameters.

To test the feasibility of using contraction dikes to correct the adverse conditions created by deposition on the crossings, the construction of a series of dikes along the left bank of the modeled reach was simulated. A one-year stage hydrograph is routed through the channel reach. By comparing the geomorphic and hydraulic changes in the modeled channel with and without contraction works, the effect of the contraction structures may be observed.

The typical result of constructing contraction dikes is a deepening of the channel at that location and an increase in stage during high flow. Although an increase in the water surface elevation may be acceptable in view of the advantages gained from channel modification, the increased potential for inundating additional portions of the river valley must be examined and evaluated.

Chapter 5

APPLICATION EXAMPLES

5.1 Introduction

The theory and knowledge presented in the River Environment can be used to determine:

1. The flow line for a given discharge, by using Equations (2.1) and (2.3). This information is useful to assess available navigation depths and dredging requirement for maintaining navigation channels, to design flood protection works, to evaluate biological habitats, and to plan land use and other related activities in the river basin. Also, effects of natural or man-induced activities (such as construction of dikes and locks and dams) on the stage-discharge relationships can be determined.
2. Superelevation of the water surface in a bend can be determined using Equation (2.5). This superelevation produces a transverse velocity which is an important mechanism that effects scour and deposition in bends. During flood stages when the flow spreads over the floodplain, this additional head may generate sufficient velocities over the floodplain to suspend fine sediment previously deposited and increase the turbidity level in the backwater area. If flood conditions are accompanied by high winds the resulting generated wind waves may cause significant fine sediment suspension. The increase in suspended sediment may cause severe damage to the

aquatic community in affected marsh and other wetland areas as well as erosion of the floodplain.

3. The flood discharge through culverts or drainage tunnels can be evaluated using the energy equation (TRE 2.1.3.3). These drainage structures allow water to flow through dams, highway and railroad embankments mainly to reduce or avoid flood damage or to provide additional water to a water-short portion of the system.

4. The roughness caused by bed configurations in sandbed rivers can be evaluated by using Figure 2-2. The bed forms affect the roughness of riverbed and control the stage-discharge relation.

5. The sediment transport rate in a river reach can be estimated by using theoretical methods such as those developed by Lane and Borland (1951), Meyer-Peter and Muller (Sheppard, 1960), Einstein (1950), and Colby (1964) (TRE 2.3.3), or by using empirical relations established from field data. The capability of the flow in the channel to transport sediment is primarily responsible for the degradation-aggradation, the dredging requirements for maintaining navigation channel, the water quality, and channel stability.

6. Potential changes in channel patterns can be evaluated by using Lane's $SQ^{\frac{1}{4}}$ relationship given in Section 2.4. Natural or man-induced activities may cause significant changes. For example a stable meandering river may be changed to a braided river by minor channel improvements if threshold conditions exist.

7. The power relations defining hydraulic geometry define channel width, depth, velocity, bed-material load, slope, roughness coefficient to the flow discharge (TRE 3.3.3). Knowledge of hydraulic geometry is essential for design of stable alluvial canals.

8. The qualitative geomorphic response of river to development can be evaluated using Equation (2-6). This initial step in analyzing long-term channel response problems is useful, because it warns of possible future difficulties in designing channel improvement and flood protection works and provides a good first-order estimate of response to development.

9. The quantitative response of river to development can be determined by using a physical or a mathematical model (Section 2.5).

To illustrate the use of developed concepts and theories twelve river problems are analyzed in this chapter. They include evaluation of:

1. River Mechanics
    a. Stage-discharge relation (Section 5.3.1)
    b. Backwater profile (Section 5.3.2)
    c. Superelevation of the water surface in a bend (Section 5.3.3)
    d. Culvert hydraulics (Section 5.3.4)
2. Sediment Transport
    a. Bed forms (Section 5.4.1)
    b. Beginning of motion (Section 5.4.2)
    c. Sediment transport rate (Section 5.4.3)
    d. Resuspension of bed material (Section 5.4.4)

3. River morphology

   a. Channel patterns (Section 5.5.1)

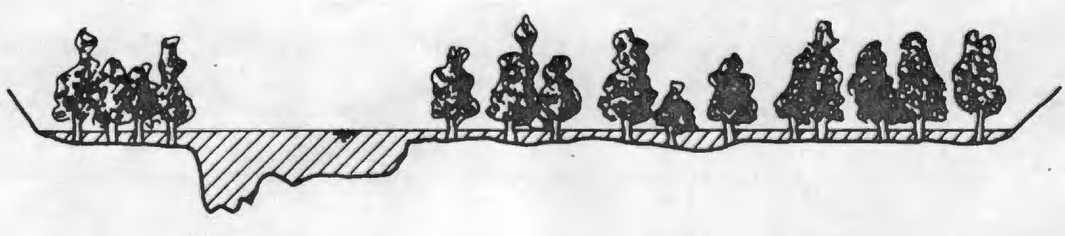
   b. Hydraulic geometry (Section 5.5.2)

   c. Qualitative response of river systems (Section 5.5.3)

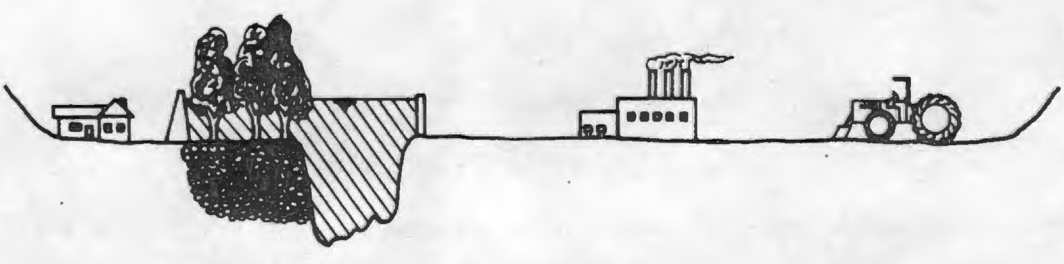
   d. Quantitative response of river systems (Section 5.5.4)

5.2 Characteristics of the Example Reach

To explain application of the available theories to study behavior of flow, an example river reach which has a uniform section (fixed cross-sectional geometry) is adopted and is shown in Figure 5-1. The natural cross section of the example river reach is



(a) The Natural River Cross Section



(b) The Developed River Cross Section

Figure 5-1. The Cross Section of the Example Reach.

shown in Figure 5-1a. It is assumed that this river reach has been developed for flood control and navigation by construction of contraction dikes and levees as shown in Figure 5-1b. Significant degradation has occurred in the river reach as a consequence of contraction. The development process the river has been subjected to in this reach is quite similar to conditions improved on the Middle Mississippi River.

The example river reach has the following properties:

(1) Bed slope $S_o = 0.6$ ft/mile. (For a nonuniform channel, an average bed slope of the study river reach is used.)

(2) Manning's roughness coefficient (estimated considering bed and bank roughness or calculated from field data):

In the main channel: $n_m = 0.035$.

On the floodplain: $n_f = 0.5$.

where the subscripts m and f denote the main channel and the floodplain respectively.

(3) For the geometric properties of a channel section in the example reach see Table 5-1.

This information on the channel properties is the basic data required to determine the hydraulic and geomorphic properties of the river reach and the river response to development. These data are used in the numerical examples presented in the following sections. Detailed data needs and data sources are given in Chapter 8 (TRE) and are summarized in the following chapter.

5.3 River Mechanics Applied to the Example River Reach

5.3.1 Stage-Discharge Relation

The determination of stage-discharge relations is based on the Manning equation (2.3). The discharge in the developed reach example

Table 5-1 Geometric Properties of the Channel at a Section in the Example Reach

Water Surface Elevation h(ft)	Cross-Sectional Area (sq ft) Main Channel A_m	Cross-Sectional Area (sq ft) Floodplain A_f	Wetted Perimeter (ft) Main Channel P_m	Wetted Perimeter (ft) Floodplain P_f
Before Development (Bed elevation z = 380 ft)				
385	15,000	0	3,260	0
390	31,750	0	3,460	0
400	66,730	0	3,580	0
410 (bank full)	103,110	0	3,800	0
420	140,610	120,000	3,810	12,000
After Development (Bed elevation z = 360 ft)*				
380	28,800	0	1,560	0
385	36,750	0	1,650	0
390	44,950	0	1,730	0
400	62,630	0	1,900	0
410 (bank full)	82,160	0	2,180	0
420	103,160	17,000	2,190	1,710

*Twenty feet of degradation in the main channel is assumed (Δz = 380 - 360 = 20 ft).

corresponding to h = 420 feet at the section (Table 5-1) is computed.

In the main channel

A_m = 103,160 sq ft (square feet)

P_m = 2,190 ft (feet)

$R_m = A_m/P_m$ = 47.11 ft

n_m = 0.035 (estimated)

$S_f \simeq S_o$ = 0.6 ft/mile = 0.000114

$$V_m = \frac{1.486}{n_m} R_m^{2/3} S_f^{1/2} = 5.91 \text{ fps (feet per second)}$$

$Q_m = A_m V_m$ = 610,000 cfs (cubic feet per second)

On the floodplain

$$A_f = 17{,}000 \text{ sq ft}$$

$$P_f = 1{,}710 \text{ ft}$$

$$R_f = A_f/P_f = 9.94 \text{ ft}$$

$$n_f = 0.5$$

$$s_f \approx S_o = 0.000114$$

$$V_f = \frac{1.486}{0.5} (9.94)^{2/3} (0.000114)^{1/2} = 0.147 \text{ fps}$$

$$Q_f = A_f V_f = 2500 \text{ cfs}$$

where A is the cross-sectional area, P is the wetted perimeter (length of the line of intersection of the channel wetted surface with a cross-sectional plane normal to the flow direction), R is the hydraulic radius, n is Manning's roughness coefficient, S_f is the friction slope, V is the average flow velocity, Q is the water discharge, h is the stage (water surface elevation) and subscripts m and f denote the main channel and the floodplain respectively. The total discharge $Q = Q_m + Q_f = 612{,}500$ cfs.

In a similar manner, the discharges for other stages are computed. The results are presented in Table 5-2 and Figure 5-2.

Figure 5-2, similar to Figure 6-24 (TRE), shows the usual effect of contraction works and levees on river hydraulics. For all flows above 420,000 cfs stages are higher after river development, while for flows below 420,000 cfs stages are lower after development than for the natural (undeveloped) river. For example, the computed results show that the discharge of 612,500 cfs produces a stage of 420 ft in the developed river reach. Before development, the same

Table 5-2 Computation of Stage-Discharge Relation

h (ft)	Main Channel V_m (fps)	Main Channel Q_m (cfs)	Floodplain V_f (fps)	Floodplain Q_f (cfs)	Q (cfs)
Before Development					
385	1.25	18,800	0	0	18,800
390	1.99	63,100	0	0	63,100
400	3.19	212,900	0	0	212,900
410	4.09	421,800	0	0	421,800
420	5.02	706,600	0.147	17,700	724,300
After Development					
380	3.17	91,300	0	0	91,300
385	3.59	131,900	0	0	131,900
390	3.98	178,900	0	0	178,900
400	4.66	291,900	0	0	291,900
410	5.10	419,000	0	0	419,000
420	5.91	610,000	0.147	2,500	612,500

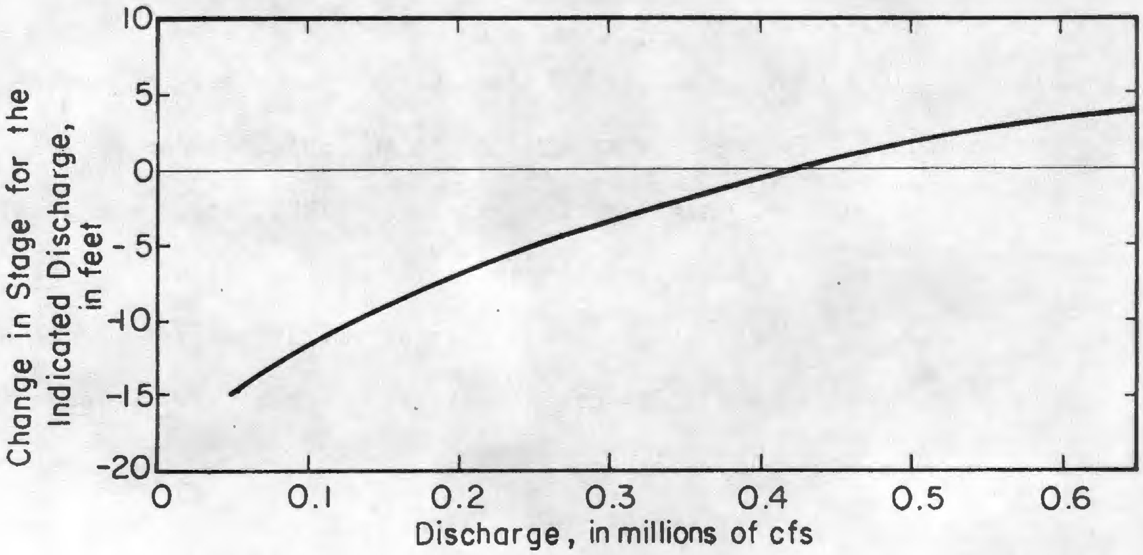


Figure 5-2. Changing Stages in the Example Reach.

discharge produced a stage of 416.5 ft. The increase in stage is 3.5 ft, and is plotted as +3.5 ft at 0.613 million cfs in Figure 5-2.

The computed stage-discharge relation is based on the approximation that $S_f \approx S_o$. Because of unsteady flow effects during passing of a flood, in general the value of S_f is slightly greater than S_o when stage is rising and S_f is slightly smaller than S_o during the falling stage. This produces a looped stage-discharge rating curve. Moreover, in alluvial channels the changes of stage alter the bed configuration, the resistance to flow and affect the stage-discharge relation. The interaction between the flow, the riverbed configuration and the complex channel geometry coupled with the unsteady flow effects produces scatter in the points used to establish the stage-discharge relation.

5.3.2 Backwater Profile

If a dam is constructed across the developed reach of river (preceding problem) and the water surface elevation above the dam is controlled at 395 feet (the corresponding flow depth is equal to 35 ft) for discharges smaller than 200,000 cfs, the dam causes a backwater effect raising the water surface profile upstream of the dam. (This condition is similar to Pool 4 where the pool level at the dam is held at Elevation 666.50 for discharges smaller than 89,000 cfs. The stage at all other points in the pool is allowed to rise.) This backwater curve can be computed using Equation (2.1).

$$\Delta L = \frac{H_2 - H_1}{S_o - S_f} \tag{5.1}$$

where ΔL is the distance between Sections 1 and 2 in the river reach and H is the specific head at Sections 1 and 2.

The procedure is to start with the known depth at the dam, then assume another depth a short distance upstream (because the flow is tranquil; for rapid flow assume a downstream depth), and compute the distance ΔL to the assumed depth. To demonstrate the computation procedure, let's compute the distance upstream to where the depth is 34 feet. For $Q = 200{,}000$ cfs; i.e.,

$y = 34$ ft

$A = 52{,}020$ sq ft (by linear interpolation from Table 5-1)

$P = 1{,}800$ ft

$R = A/P = 28.90$ ft

$V = Q/A = 3.84$ fps

$$S_f = \left[\frac{nV}{1.486\ R^{2/3}}\right]^2 = 0.0000922$$

$$\frac{V^2}{2g} = 0.23$$

$$H = y + \frac{V^2}{2g} = 34.23 \text{ ft}$$

At the dam,

$y = 35$ ft

$A = 53{,}790$ sq ft

$P = 1{,}815$ ft

$$R = \frac{53{,}790}{1{,}815} = 29.64 \text{ ft}$$

$$V = \frac{200{,}000}{53{,}790} = 3.72 \text{ fps}$$

$$S_f = \left[\frac{0.035 \times 3.72}{1.486\ (29.64)^{2/3}}\right]^2 = 0.0000837$$

$$\frac{V^2}{2g} = 0.21 \text{ ft}$$

$$H = 35 + 0.21 = 35.21 \text{ ft}$$

Between these sections where $y = 35$ feet and $y = 34$ feet the average friction slope is

$$S_{f_{ave}} = \frac{0.0000837 + 0.0000922}{2} = 0.0000880$$

The distance between these two sections is

$$\Delta L = \frac{35.21 - 34.23}{0.000114 - 0.0000880} = 37{,}700 \text{ ft}$$

Hence, the section where the depth is 34 feet is 37,700 feet upstream of the dam.

In a similar manner, the distance between the sections where the depths are 33 feet and 32 feet (arbitrary choice) is computed. The analysis is given in Table 5-3. Note that the dam causes a backwater effect that extends approximately 54 miles upstream of the dam. Upstream of this location the flow is normal, i.e., unaffected by the dam. However, subsequent deposition of sediment in the backwater area could change the results of this analysis.

Table 5-3 Computation of the Backwater Curve

y ft	h ft	A sq ft	P ft	V fps	S_f 10^{-4}	$S_{f_{ave}}$ 10^{-4}	$V^2/2g$ ft	H ft	ΔL miles	L miles
35	395.00	53,790	1,815	3.72	0.837	-	0.21	35.21	-	0
34	398.28	52,020	1,800	3.84	0.922	0.880	0.23	34.23	7.14	7.14
33	406.80	50,254	1,781	3.98	1.023	0.973	0.25	33.25	15.86	23.00
32	424.24	48,486	1,764	4.12	1.135	1.079	0.26	32.26	30.74	53.74

If the river reach is nonuniform, it is adequate to compute the backwater curve by solving a simplified form of the energy equation:

$$h_1 + \frac{V_1^2}{2g} = h_2 + \frac{V_2^2}{2g} + S_f \, \Delta L \tag{5.2}$$

The computations procedure is to choose a section upstream of the dam, measure the distance ΔL, and compute the stage h_1 at this section by an iteration method.

5.3.3 Superelevation of the Water Surface in a Bend

Considering natural river conditions in the example reach and a bend with a radius of 10,000 feet, compute the total superelevation between the outer and inner bank using Equation (2.5).

At $h = 410$ feet, the corresponding top width is 3750 feet and the velocity is 4.09 fps (Table 5-2). For these conditions the total superelevation is

$$\Delta Z = \frac{V^2}{g r_c} (r_o - r_i)$$

$$= \frac{4.09^2}{32.2 \times 10{,}000} (3{,}750) = 0.195 \text{ ft}$$

The terms r_i, r_c, and r_o are the radius at the inside, the center and the outside of the bend respectively.

5.3.4 Culvert Hydraulics

The culvert is usually a covered channel of comparatively short length installed to allow water to flow through dams, highway and railroad embankments. The culvert may flow partly full as an open channel, or may flow full as a pipe. With flood conditions the outlet of a culvert may be submerged causing the culvert to flow full. A field examination frequently will be required to estimate the water levels at the inlet and the outlet. The difference in water levels is the head ΔH which controls the culvert discharge.

When culverts flow full, the discharge can be determined from the energy equation:

$$\Delta H = H_e + H_f + H_o \tag{5.3}$$

where H_e, H_f and H_o are energy losses at inlet, through the culvert and at outlet respectively.

Consider a culvert with a diameter $D = 10$ feet and length $L = 100$ feet. During a flood, the culvert flows full and the head causing flow is $\Delta H = 3$ feet. The culvert discharge is determined for this condition.

The energy losses can be expressed as a function of velocity in the culvert, i.e.,

$$H_e = K_e \frac{V^2}{2g} \text{ (entrance loss)} \tag{5.4}$$

$$H_f = L\, S_f = L\left(\frac{nV}{1.486\, R^{2/3}}\right)^2 \text{ (friction loss)} \tag{5.5}$$

$$H_o = K_o \frac{V^2}{2g} \text{ (exit loss)} \tag{5.6}$$

where the coefficients $K_e = 0.5$, $n = 0.02$ and $K_o = 1.0$ are estimated, and the hydraulic radius $R = D/4$. From Equations (5.3) through (5.6)

$$\Delta H = V^2 \left(\frac{K_e}{2g} + L \frac{n^2}{2.21\, R^{4/3}} + \frac{K_o}{2g}\right)$$

$$3 = V^2 \left(\frac{0.5}{64.4} + 100 \frac{0.02^2}{2.21(10/4)^{4/3}} + \frac{1}{64.4}\right)$$

$$3 = 0.0286\, V^2$$

and

$$V = 10.24 \text{ fps}$$

Then

$$Q = AV = \frac{\pi}{4} (10)^2 \times 10.24 = 804 \text{ cfs}$$

When culverts flow partly full, the culvert discharge should be determined using nonuniform flow theories developed for open channel flow conditions (TRE 2.1.6, Chow (1959)).

5.4 Sediment Transport

5.4.1 Bed Forms

Assume the example reach (Figure 5-1) has a median size of bed material $d_{50} = 0.4$ millimeter. The bed form in this river reach can be determined from Figure 2-2. The estimated bed form is presented in Table 5-4. It is found that narrowing the river to improve the navigation channel effects the bed configuration. For example, at stage $h = 385$ feet the predominate bed form in the natural channel is dunes, but after river development the bed form changes because of increased depth and hence increased stream power to transition. This change in bed configuration changes the resistance to flow and river stage. The effect of the channel change on bed roughness is explained in Figure 5-11 (TRE) and Section 2.3.

5.4.2 Beginning of Motion

Water flowing in an alluvial channel exerts forces on the bed material that tend to move or entrain the particles. When these forces have reached value that, if increased even slightly will put the sediment particles into motion, critical or threshold conditions exist. With critical conditions the hydrodynamic forces acting on a grain of

Table 5-4 Determination of Form of Bed Roughness and Sediment Discharge

h ft	R ft	τ lb/ft^2	V fps	τV lb/ft-sec	Bed Form	Q_s tons/day
Before Development						
385	4.60	0.033	1.25	0.041	Dunes	-
390	9.18	0.066	1.99	0.130	Dunes	3,770
400	18.64	0.133	3.19	0.423	Dunes	50,500
410	27.13	0.193	4.09	0.789	Upper regimes	182,000
420	36.91	0.263	5.02	1.318	Upper regimes	335,000
After Development						
380	18.46	0.131	3.17	0.416	Dunes	22,000
385	22.27	0.158	3.59	0.569	Transition	44,200
390	25.98	0.185	3.98	0.736	Upper regimes	76,500
400	32.96	0.234	4.66	1.093	Upper regimes	171,000
410	37.69	0.268	5.10	1.367	Upper regimes	237,000
420	47.10	0.335	5.91	1.980	Upper regimes	373,000

Note: R and V were obtained from Tables 5-1 and 5-2, respectively.

Shear stress $\tau = \gamma R S_f \simeq \gamma R S_o$.

The bed-material load Q_s is determined from Colby's method.

bed material is just balanced by the resisting force of the particle. This hydrodynamic force acting on the effective surface area of the particle is called the <u>critical shear stress.</u>

The critical shear stress can be determined from Figure 2-20 (TRE). For large shear Reynolds number, the relation is

$$\tau_c = 0.047(\gamma_s - \gamma)d \tag{5.7}$$

For a sediment particle of size $d = 0.4$ mm and unit weight $\gamma_s = 165$ lbs/cu ft, the critical shear stress is

$$\tau_c = 0.047(165 - 62.4)0.4/304.8 = 0.0063 \text{ lb/sq ft.}$$

By examining Table 5-4, the shear stress for different flow conditions in the example reach exceeds the critical shear stress. This indicates that the bed material is in motion. The rate of transport of the bed material is determined in the following section.

5.4.3 Sediment Transport

In TRE 2.3.3.3, Colby's and Einstein's methods for computing bed-material discharges are presented. Colby's method is recommended for rivers with flow depths less than 10 feet but works well over an even larger range of depths. Einstein's method is suggested for rivers for which the bed load is a significant part of the total bed-material load.

In the example reach, Colby's method is used to compute the bed-material discharges at various stages. The procedure is to first read the uncorrected sediment discharge, q_n, corresponding to the known V and d_{50} for two of the depths indicated in Figure 5-3 which brackets the desired mean depth. Then interpolate on a logarithmic graph of depth versus q_n, to obtain the bed-material discharge per unit width for the desired D, V and d_{50}. The true sediment discharge, q_t, is determined from Equation (2.84) (TRE), where the coefficients k_1, k_2 and k_3 can be found from the charts developed by Colby (1964). An example computation follows.

At $h = 390$ feet, the section (Table 5-4) in the natural example reach has $D \simeq R = 9.18$ feet and $V = 1.99$ fps. Then from Figure 5-3, at

$$D = 1.0 \text{ ft}$$

$$V = 1.99 \text{ fps}$$

$$d_{50} = 0.4 \text{ mm (millimeter)}$$

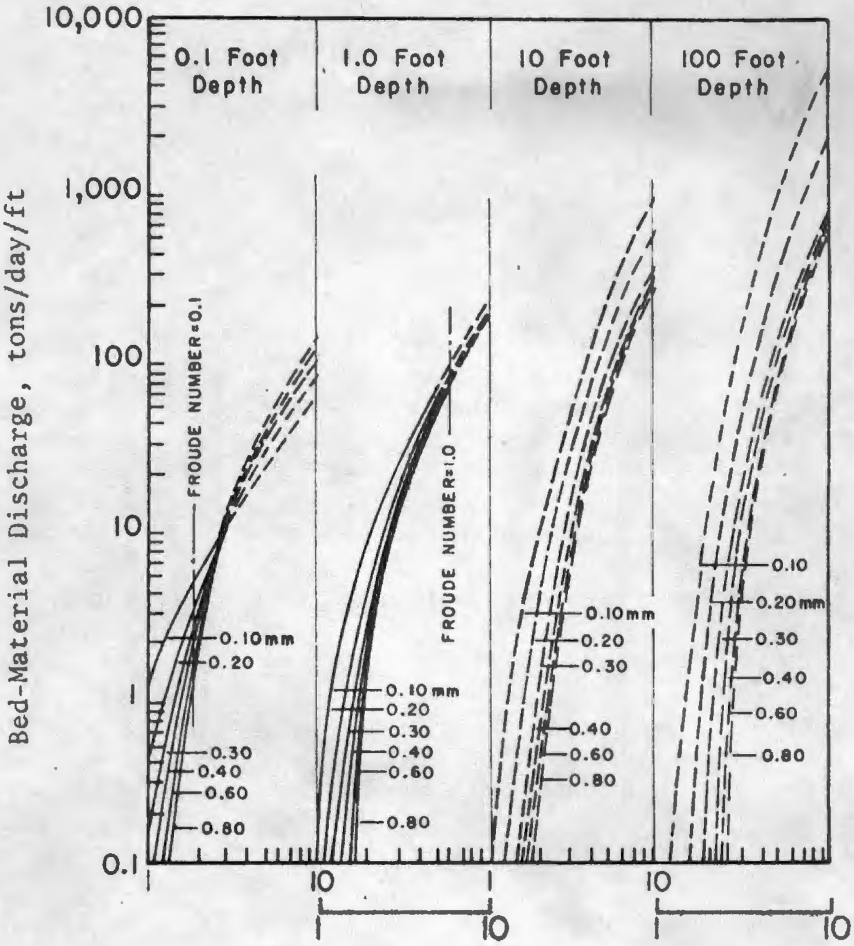


Figure 5-3 Relation of Discharge of Sands to Mean Velocity for Six Median Sizes of Bed Sands, Depth of Flow, and a Water Temperature of 60°F (after Colby, 1964; Figure 2-31, TRE)

We have

$$q_n = 2.18 \text{ tons/day/ft}$$

Similarly, at

$$D = 10 \text{ ft}$$

$$q_n = 1.06 \text{ tonw/day/ft}$$

After interpolating on a logarithmic paper, the value of q_n for $D = 9.18$ feet is 1.09 tons/day/ft. From Colby's charts, $K_1 = 1.0$ at 60°F, $k_2 = 1.0$ for negligible fine sediment and $k_3 = 100$ for $d_{50} = 0.4$ mm. Therefore, by applying Equation (2.84)(TRE),

$$q_t = [1 + (k_1\ k_2 - 1)(0.01)k_3]q_n = 1.09 \text{ tons/day/ft.}$$

The top width $W \simeq P = 3,460$ feet (Table 5-1). Then the total bed-material load

$$Q_T = Wq_t = 3,770 \text{ tons/day.}$$

In a similar manner, the bed-material load can be determined for other flow conditions. The computed results are presented in Table 5-4 and Figure 5-4. This figure shows that the sediment discharge in the developed channel is larger than for the natural channel for the same discharge. This increase in sediment transport rate caused by contraction works causes degradation of the river reach.

5.4.4 Resuspension of Bed Material

Any disturbance of the streambed either through increased velocity, or turbulence generated by passage of boats suspends material that previously settled. The turbulence that is generated by a boat decays and the resuspended material settles. The time required for settling of suspended material depends on the velocity and turbulence of the flow and fall velocity of the suspended particles.

Consider a simple case and use a simplified approach to show the effects of the fall velocity of the particles on settling of resuspended sediment. Assume in a uniform reach the flow has a depth $D = 12$ feet and a velocity $V = 4$ fps. A ship travels downstream, generating sufficient turbulence to resuspend fine sediment ($d = 0.1$ mm). Immediately after passage of the ship, the resuspended sediment is assumed to be uniformly distributed over that segment of channel cross section affected by the ship. Assume a fine sediment concentration of $C = 1000$ ppm. The sediment travels with the flow velocity. Because of gravity effect, each particle is settling with a fall velocity of $\omega = 0.025$ fps when the effect of turbulence fluctuations on settling of sediment is negligible.

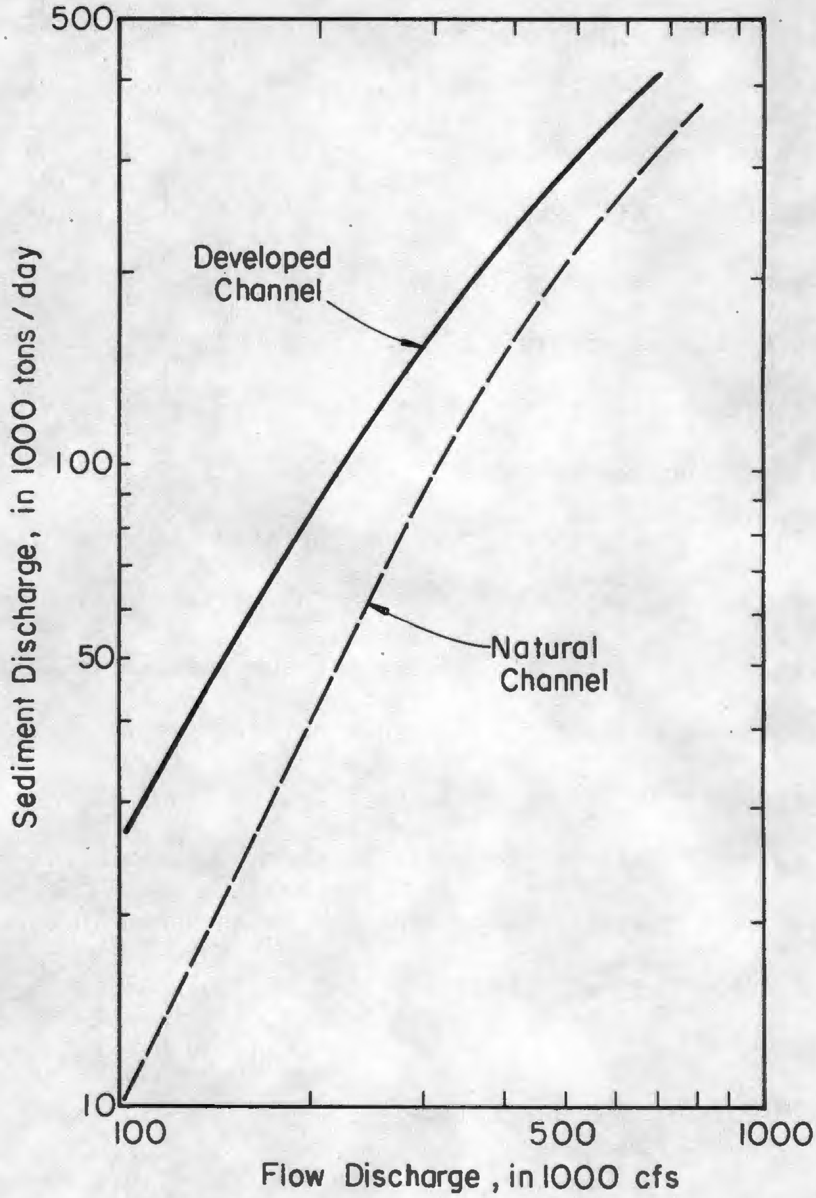


Figure 5-4 Flow and Sediment Discharge Relationship in the Example Reach.

Consider a cross section in the river reach. The sediment resuspended by the ship over this cross section is carried downstream by the flow. The sediment resuspended at a station $L = 400$ feet upstream travels downstream and reaches this particular section at $t = L/V = 400/4 = 100$ seconds after the passage of the boat. A portion of this resuspended sediment has settled out before reaching this section. Since a particle has settled a distance of approximately $y = \omega t = 0.025 \times 100 = 2.5$ feet in 100 seconds, that portion of resuspended particles within 2.5 feet of the bed would have settled out. The mean concentration of fine particles still in suspension 400 feet downstream is then about $C(D-y)/D = 1000 \times (12 - 2.5)/12 = 792$ ppm.

In a similar manner, the concentration of resuspended sediment over the cross section is assessed for different time periods (Figure 5-5). All of the sediment resuspended more than 1,920 feet upstream would have settled before arriving at this section.

If the resuspended sediment is still finer (has a diameter of 0.06 mm and a fall velocity of 0.010 fps) the turbidity level is higher and sediment stays in suspension longer as shown in Figure 5-5. This explains why the bed material of the Illinois River is more susceptible to resuspension due to boat effects than the bed material of the Upper Mississippi River which is coarser.

The example is simplified. In natural rivers the settling of suspended particles is complicated by the turbulence of the flow and the interaction between the flowing water and the suspended particles. Better results can be obtained by using the dispersion equation to simulate the settling process.

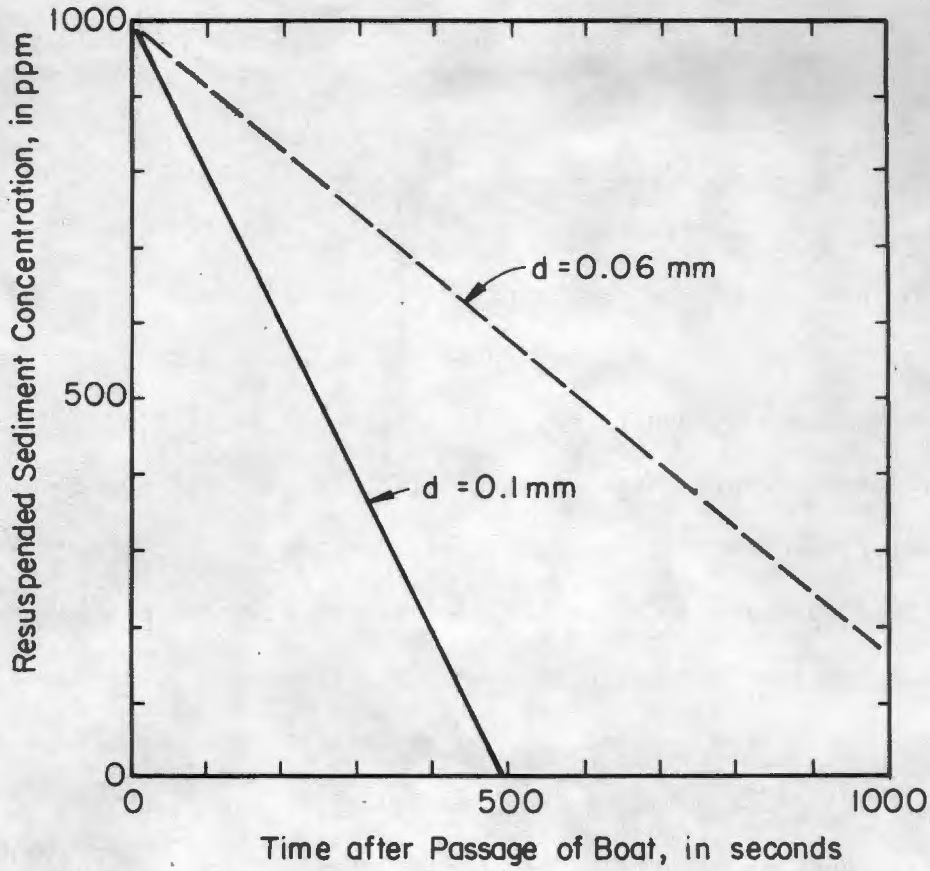


Figure 5-5 Resuspension of Bed material Caused by Passage of Boat

5.5 River Morphology

5.5.1 Channel Pattern

The example reach (Figure 5-1) has a bed slope of 0.6 ft/mile. At bank-full stage the flow discharge is 420,000 cfs. Therefore

$$S\,Q^{1/4} = 0.0029$$

Comparing this value to the limits 0.01 and 0.0017 (when $SQ^{1/4} \leq 0.0017$ a sandbed channel tends toward a meandering pattern; when $SQ^{1/4} \geq 0.01$, a river tends toward a braided pattern, Section 2-4), the example reach falls in the transitional range.

If the reach is straightened by channelization or cutoffs, increasing the slope to 2.5 ft/mile

$$S\ Q^{1/4} = 0.012 > 0.01$$

the reach will tend to braid.

5.5.2 Hydraulic Geometry

Alluvial stream dimensions can be estimated using quantitative hydraulic geometry relations. In general, these relations apply to channels within a physiographic region and can be derived from data available on gaged rivers. It is understood that hydraulic geometry relations express the integral effect of all the hydrologic, meteorologic, and geologic variables of a drainage basin (TRE 3.3.3).

Leopold and Maddock (1953) have shown that in a drainage basin, two types of hydraulic geometric relations can be defined: (1) those relating W, D, V and Q_T to the variation of discharge at a station, and (2) those relating these variables to the discharge of a given frequency of occurrence at various stations in a drainage basin. For a uniform channel reach, these two relations are identical.

The hydraulic geometry relations for the example reach (Figure 5-1) are determined by plotting A, W, D and V versus Q as shown in Figure 5-6. The values of A, and V and Q are given in Tables 5-1 and 5-2 respectively. The values of W and D are determined based on the following relations:

$$P \approx W + 2D \tag{5.8}$$

and

$$A = WD \tag{5.9}$$

In Figure 5-6, the points present the hydraulic properties determined previously, and straight lines are fitted to the points statistically or by eye to best represent the general relation. The curves are drawn in such a way as to be consistent. If a vertical

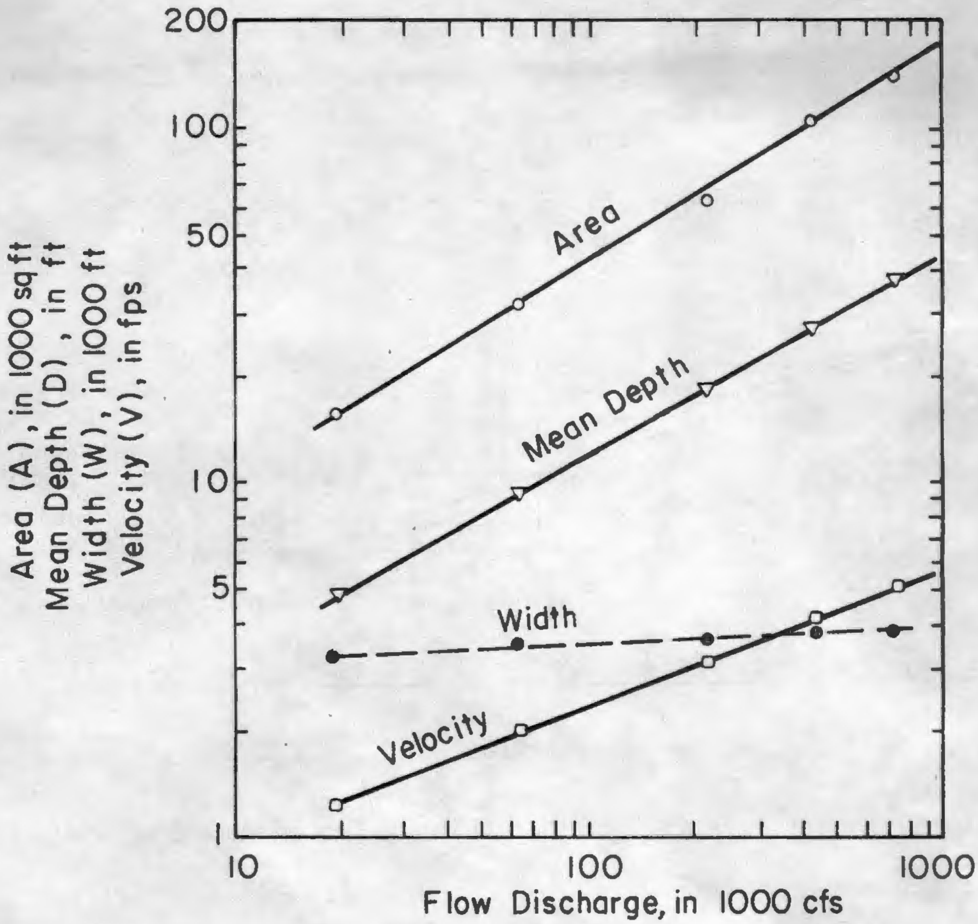


Figure 5-6 Hydraulic Geometry Relations in the Example Reach

section is taken at a given discharge, the values from the curves must satisfy two physical laws: (1) the product of width and depth equals area, WD = A; and (2) the area times velocity equals the given discharge, AV = Q. The resulted hydraulic geometry relations are

$$A = 34.1\ Q^{0.619} \tag{5.10}$$

$$D = 0.0178\ Q^{0.566} \tag{5.11}$$

$$W = 1917\ Q^{0.053} \tag{5.12}$$

$$V = 0.0293\ Q^{0.381} \tag{5.13}$$

5.5.3 Qualitative Response of River Systems

The geomorphic response of river to development can be evaluated qualitatively using Equation (2.6). To use a classic example, consider the downstream response of a river to the construction of a dam (Figure 5-7). Aggradation in the reservoir upstream of the dam will

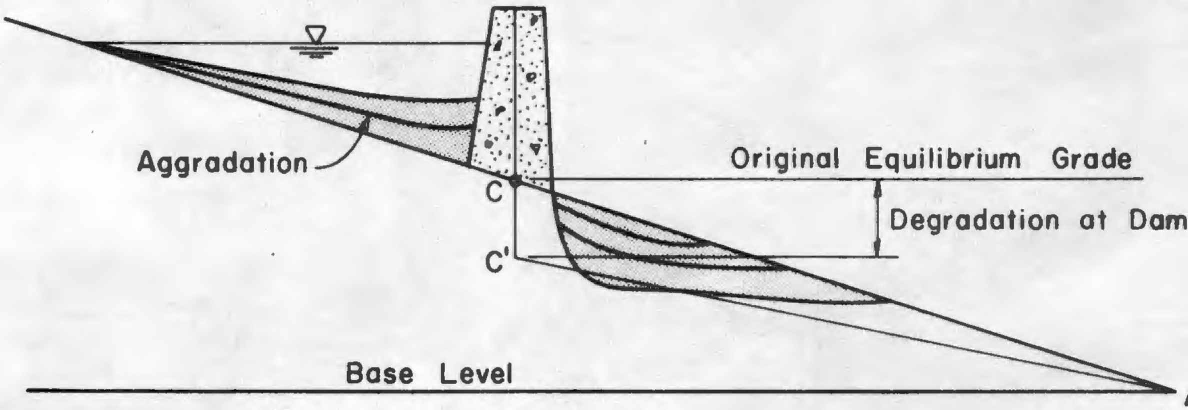


Figure 5-7 Channel Adjustment above and below a Dam (Figure 3-12, TRE)

result in relatively clear water being released downstream of the dam, that is, Q_s will be reduced to Q_s^- downstream. Assuming particle size of the bed material and water discharge remain constant, slope must decrease downstream of the dam to balance the proportionality of Equation (2.6)

$$Q_s^- D_{50}^o \sim Q^o S^- \tag{5.14}$$

In Figure 5-7 the original channel gradient between the dam and a downstream geologic control (line CA) will be reduced to a new gradient (line C'A) through gradual degradation below the dam. With time, of course, the pool behind the dam will fill and sediment would again be available to the downstream reach. Then, except for local scour, the gradient C'A would increase to the original gradient CA to transport the increase in sediment load. Upstream, the gradient would eventually parallel the original gradient, offset by the height of the dam. Thus, dams with small storage capacity may induce scour and then deposition over a relatively short time period.

As another example, Figure 5-8 illustrates a more complicated set of circumstances. In this case a reach of river is affected by Dam A constructed upstream as well as Dam B constructed downstream. As shown in Figure 5-7, Dam A would cause significant degradation in the main

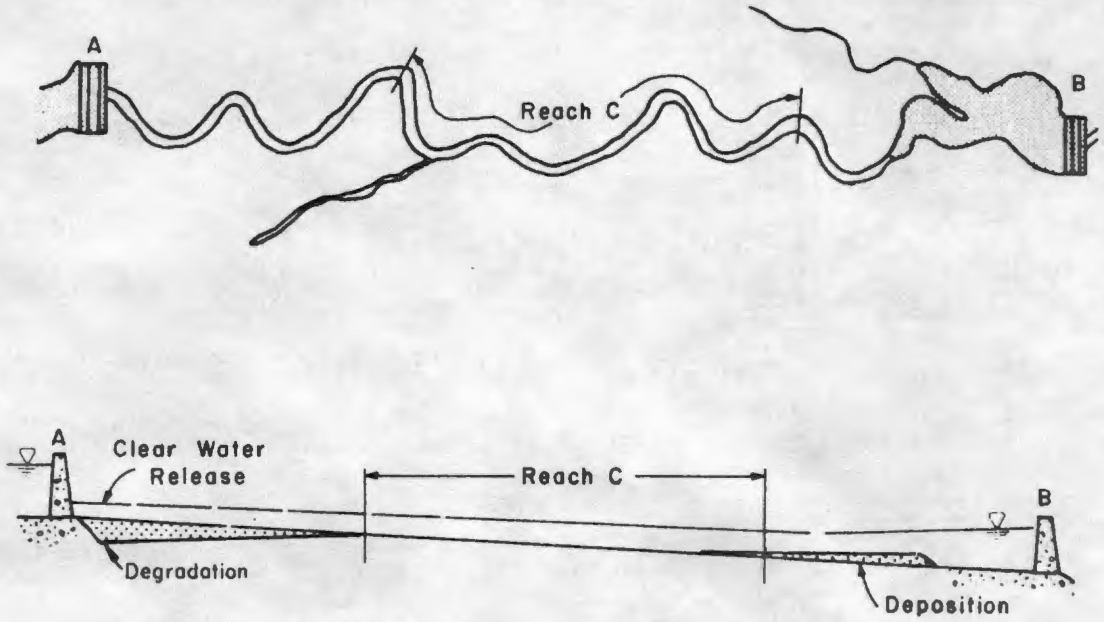


Figure 5-8 Combined Increase of Base Level and Reduction of Upstream Sediment Load (Figure 5-8, TRE)

channel. The final condition in Reach C is estimated by summing the affects of both dams on the main channel and the tributary flows. The scour below Dam A would make some sedimentary material available for deposition in the reservoir above Dam B, further complicating the situation. Normally, this analysis requires water and sediment routing techniques studying both short- and long-term effects of the construction of these dams.

Additional examples that demonstrate qualitative analysis of river response to river development are presented in TRE 3.3.2 and 5.2. They include

1. Changes in channel slope in response to an increase in sediment load (Figure 3-13, TRE).
2. Lowering of base level for tributary stream (Figure 5-1, TRE).
3. Raising base level for tributary stream (Figure 5-3, TRE).
4. Straightening of a reach by cutoffs (Figure 5-5, TRE).
5. Closure of a chute channel (Figure 5-6, TRE).
6. Tectonic activity (Figure 5-7, TRE).
7. Clear water diversion and release combined with downstream storage (Figure 5-9, TRE).

5.5.4 Quantitative Response of River Systems

Physical models and mathematical models are used to assess quantitative response of river systems. Examples shown in The River Environment include:

1. Evolution of man-made side channels (TRE 6.3.1).
2. Manchester Islands model study (TRE 7.3.3).
3. Mathematical modeling of response to dikes (TRE 6.4.4).
4. Mathematical modeling of Pools 24, 25 and 26 in the Upper Mississippi and Lower Illinois Rivers (TRE 5.4).
5. Mathematical modeling of disposal of dredged material in the main channel (TRE 7.7.3).

Chapter 6

DATA NEEDS AND DATA SOURCES

6.1 Data Needs

The data needed for the geomorphic, hydraulic, and environmental analysis of river systems can be categorized as follows: (1) topographic and hydrographic data, (2) geologic data, (3) hydrologic data, (4) hydraulic data, (5) environmental data, and (6) climatologic data (TRE 8.2).

The types of data needed for river analysis and the relative importance of each type of data are listed in Table 6-1. Data with a degree of importance "Primary" are basic data required for any geomorphic, hydraulic, and environmental study of a river such as the Upper Mississippi. Whenever possible, these data should be directly collected from the field. Other data with a degree of importance "Secondary" are also very helpful in an analysis of river but are a secondary requirement.

Limited data synthesis using records of surrounding stations and/or empirical, experimental and theoretical relations is possible. The quality of synthesis is also described as good, fair, or poor in Table 6-1. It is apparent that hydrologic and hydraulic data can be more readily synthesized than other types of data.

Table 6-1 Checklist of Date Needs

Description of data or needed information	Degree of Data Importance	Quality of Synthesized Data
Maps and charts:		
Hydrographic	Primary	Poor
Topographic	Primary	Poor
Geologic	Secondary	Poor
Navigation charts	Secondary	-
Dredging surveys	Secondary	-
Potamology surveys	Secondary	-
County and city plats	Secondary	-
Aerial and other photos:		
Large scale photos of river and surrounding terrain	Primary	-
Small scale stereo pairs of river and surrounding terrain	Secondary	-
Color infrared photos for flow patterns, scour zones, and vegetation	Secondary	-
Ground photos	Secondary	-
Information on existing structures: locks, dams, dikes, diversions, or outfalls:		
Plans and details	Primary	-
Construction details	Secondary	-
Alterations and repairs	Secondary	-
Foundations	Secondary	-
Piers and abutments	Secondary	-
Scour	Primary	Fair
Aggradation	Primary	Fair
Field investigations	Primary	-
Hydraulics, Hydrology and Soils:		
Discharge records	Primary	Fair
Stage records	Primary	Fair
Flood frequency curves for stations near reach	Secondary	Fair
Flow duration curves (hydrographs)	Secondary	Fair
Newspaper, radio, television, accounts of large floods	Secondary	Fair

Table 6-1 Checklist of Data Needs (continued)

Description of data or needed information	Degree of Data Importance	Quality of Synthesized Data
Channel geometry		
Main channel	Primary	Fair
Side channel	Primary	Poor
Islands	Primary	Poor
Navigation channel	Secondary	Poor
Floodplain	Secondary	Poor
Slopes	Primary	Poor
Backwater calculation	Secondary	Poor
Bars	Secondary	Poor
Sinuosity	Secondary	Poor
Type (braided, meandering, straight)	Secondary	Poor
Controls (falls, rapids, restriction, rock outcropping dams, diversions)	Primary	Poor
Sediment discharge		
Size distribution	Primary	Fair
Bed and bank material sizes	Primary	Fair
Roughness coefficient n	Primary	Fair
Bed load	Primary	Fair
Suspended load	Primary	Fair
Wash load	Secondary	Fair
Ice:		
Recorded thickness	Secondary	Fair
Dates of freeze up and break up	Secondary	Good
Flow patterns and jams	Secondary	Good
Damage	Secondary	-
Regulating structures:		
Dams, diversions	Primary	-
Intake, outfalls	Secondary	-
Scour survey around hydraulic structures (piers, abutments, dikes)	Secondary	-
Planned and anticipated water resources projects	Primary	-
Lakes, tributaries, reservoirs or side channel impoundments	Primary	-
Field Surveys:		
Onsite inspections and photographs	Primary	-

Table 6-1 Checklist of Data Needs (continued)

Description of data or needed information	Degree of Data Importance	Quality of[1] Synthesized Data
Sample sediments	Secondary	-
Measure water and sediment discharge	Secondary	-
Observe channel changes or realignment since last maps or photos	Secondary	-
Identify high water lines or debris deposits due to recent floods	Secondary	-
Check magnitude of velocities and direction of flow	Secondary	-
Outcroppings	Secondary	-
Subsurface exploration	Secondary	-
Environmental data:		
Forests	Primary	Poor
Vegetation	Primary	Poor
Wildlife	Primary	Fair
Fish habitat	Primary	Fair
Turbidity	Primary	Fair
Chemical quality	Primary	Fair
Water temperature	Primary	Good
Climatologic data:		
National Weather Service records for precipitation	Secondary	Poor
Wind	Secondary	Poor
Temperatures	Secondary	Fair
Land use:		
Zoning maps	Secondary	-
Recent aerial photographs	Secondary	-
Planning committee records	Secondary	-
Urban areas	Secondary	-
Industrial areas	Secondary	-
Recreational areas	Secondary	-
Primitive areas	Secondary	-

6.2 Data Sources

The best data sources are national data centers where the principle function is to disseminate data. But it is usually necessary to collect data from a variety of other sources such as a field investigation, interviews with local residents, and search through library materials of federal, state, and local agencies. The following list of sources is provided to serve as a guide to the data collection task:

Topographic Maps:

(1) Quadrangle maps -- U. S. Department of the Interior, Geological Survey, Topographic Division; and U. S. Department of the Army, Army Map Service.
(2) River plans and profiles -- U. S. Department of the Interior, Geological Survey, Conservation Division.
(3) National parks and monuments -- U. S. Department of the Interior, National Park Service.
(4) Federal reclamation project maps -- U. S. Department of the Interior, Bureau of Reclamation.
(5) Local areas -- commercial aerial mapping firms.
(6) American Society of Photogrammetry.

Planimetric Maps:

(1) Plats of public land surveys -- U. S. Department of the Interior, Bureau of Land Management.
(2) National forest maps -- U. S. Department of Agriculture, Forest Service.
(3) County maps -- State Highway Agency.
(4) City plats -- city or county recorder.
(5) Federal reclamation project maps -- U. S. Department of the Interior, Bureau of Reclamation.
(6) American Society of Photogrammetry.
(7) ASCE Journal -- Surveying and Mapping Division.

Aerial Photographs:

(1) The following agencies have aerial photographs of portions of the United States: U. S. Department of the Interior, Geological Survey, Topographic Division; U. S. Department of Agriculture, Commodity Stabilization Service, Soil Conservation Service and Forest Service; U. S. Air Force; various State agencies; commercial aerial survey; National Oceanic and Atmospheric Administration; and mapping firms.
(2) American Society of Photogrammetry.
(3) Photogrammetric Engineering
(4) Earth Resources Observation System (EROS) Photographs from Gemini, Apollo, Earth Resources Technology Satellite (ERTS) and Skylab.

Transportation Maps:

(1) State Highway Agency.

Triangulation and Benchmarks:

(1) State Engineer.
(2) State Highway Agency.

Geologic Maps:

(1) U. S. Department of the Interior, Geologic Survey, Geologic Division; and State geological surveys or departments.
(Note -- some quadrangle maps show geologic data also).

Soils Data:

(1) County soil survey reports -- U. S. Department of Agriculture, Soil Conservation Service.
(2) Land use capability surveys -- U. S. Department of Agriculture, Soil Conservation Service.
(3) Land classification reports -- U. S. Department of the Interior, Bureau of Reclamation.
(4) Hydraulic laboratory reports -- U. S. Department of the Interior, Bureau of Reclamation.

Climatologic Data:

(1) National Weather Service Data Center.
(2) Hydrologic bulletin -- U. S. Department of Commerce, National Oceanic and Atmospheric Administration.
(3) Technical papers -- U. S. Department of Commerce, National Oceanic and Atmospheric Administration.
(4) Hydrometeorological reports -- U. S. Department of Commerce, National Oceanic and Atmospheric Administration, and U. S. Department of the Army, Corps of Engineers.
(5) Cooperative study reports -- U. S. Department of Commerce, National Oceanic and Atmospheric Administration and U. S. Department of the Interior, Bureau of Reclamation.

Stream Flow Data:

(1) Water supply papers -- U. S. Department of the Interior, Geological Survey, Water Resources Division.
(2) Reports of State engineers.
(3) Annual reports -- International Boundary and Water Commission, United States and Mexico.
(4) Annual reports -- various interstate compact commissions.
(5) Hydraulic laboratory reports -- U. S. Department of the Interior, Bureau of Reclamation.
(6) Corp of Engineers, U. S. Army, District offices.

Sedimentation Data:

(1) Water supply papers -- U. S. Department of the Interior, Geological Survey, Quality of Water Branch.
(2) Reports -- U. S. Department of the Interior, Bureau of Reclamation; and U. S. Department of Agriculture, Soil Conservation Service.
(3) Geological Survey Circulars -- U. S. Department of the Interior, Geological Survey.
(4) Corps of Engineers, U. S. Army, District offices, reservoir operation and dredging records.

Water Quality:

(1) Water supply papers -- U. S. Department of the Interior, Geological Survey, Quality of Water Branch.
(2) Reports -- U. S. Department of Health, Education, and Welfare, Public Health Service.
(3) Reports -- State public health departments.
(4) Water Resources Publications -- U. S. Department of the Interior, Bureau of Reclamation.
(5) Environmental Protection Agency, regional offices.
(6) State water quality agency.

Irrigation and Drainage Data:

(1) Agricultural census reports -- U. S. Department of Commerce, Bureau of the Census.
(2) Agricultural statistics -- U. S. Department of Agriculture, Agricultural Marketing Service.
(3) Federal reclamation projects -- U. S. Department of the Interior, Bureau of Reclamation.
(4) Reports and Progress Reports -- U. S. Department of the Interior, Bureau of Reclamation.

Power Data:

(1) Directory of Electric Utilities -- McGraw Hill Publishing Co.
(2) Directory of Electric and Gas Utilities in the United States -- Federal Power Commission.
(3) Reports -- various power companies, public utilities, State power commissions, etc.

Basin and Project Reports and Special Reports:

(1) U. S. Department of the Army, Corps of Engineers.
(2) U. S. Department of the Interior, Bureau of Land Management, Bureau of Mines, Bureau of Reclamation, Fish and Wildlife Service, and National Park Service.
(3) U. S. Department of Agriculture, Soil Conservation Service.
(4) U. S. Department of Health, Education, and Welfare, Public Health Service.
(5) State departments of water resources, departments of public works, power authorities and planning commissions.
(6) Upper Mississippi River Basin Coordinating Committee.

Environmental Data:

(1) Sanitation and public health -- U. S. Department of Health, Education and Welfare, Public Health Service; State departments of public health.
(2) Fish and wildlife -- U. S. Department of the Interior, Fish and Wildlife Service; state game and fish departments.
(3) Municipal and industrial water supplies -- city water departments; State universities; Bureau of Business Research; State water conservation boards or State public works departments, State health agencies, Environmental Protection Agency, Public Health Service.
(4) Watershed management -- U. S. Department of Agriculture, Soil Conservation Service, Forest Service; U. S. Department of the Interior, Bureau of Land Management, Bureau of Indian Affairs.
(5) Upper Mississippi River Conservation Committee.

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