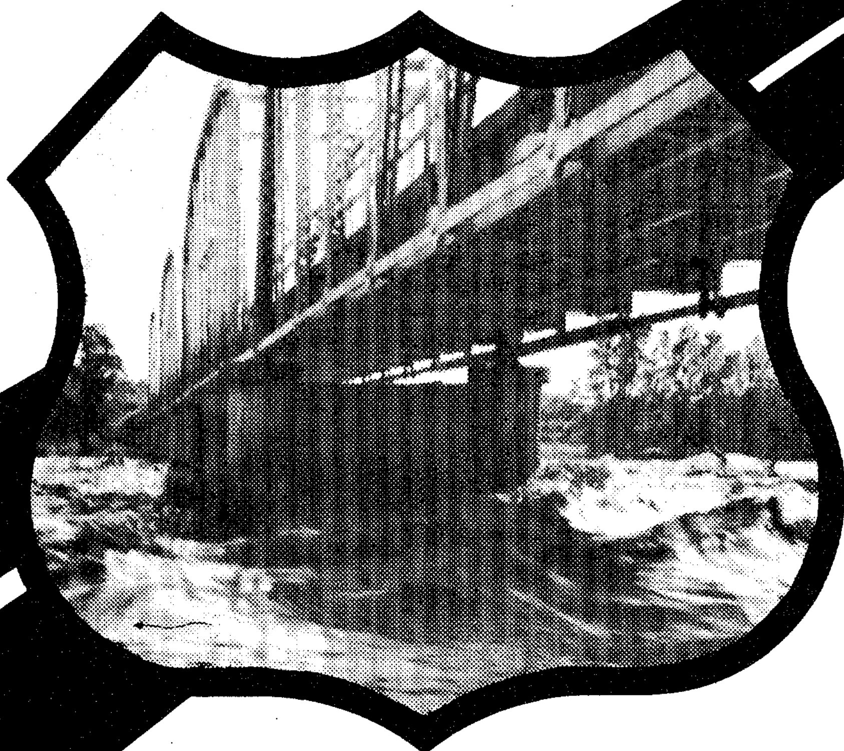
Report No. FHWA/RD-80/159

STREAM CHANNEL DEGRADATION AND AGGRADATION: ANALYSIS OF IMPACTS TO HIGHWAY CROSSINGS

March 1981

Final Report



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DEPARTMENT OF TRANSPORTATION · UNITED STATES OF AMERICA

Prepared for

FEDERAL HIGHWAY ADMINISTRATION

Offices of Research & Development

Environmental Division

Washington, D.C. 20590

FOREWORD

This final report describes the second phase of a research study on stream channel degradation and aggradation. Gradation changes are long-term channel bed elevation changes which extend for long distances along the streambed. The final report describes the relationships between gradation problems and highway crossing design, methods for determining the magnitude of gradation problems, and countermeasures for control of degradation and aggradation. The first phase is documented in an interim report (FHWA/RD-80/038) titled "Stream Channel Degradation and Aggradation: Causes and Consequences to Highways." The interim report describes the extent of gradation problems nationwide, examines the causes of gradation changes, and describes the data base used for both reports.

Research in highway drainage and stream crossing design is included in the Federally Coordinated Program of Highway Research and Development in Project 5H "Protection of the Highway System from Hazards Attributed to Flooding." Roy E. Trent is the project manager and Stephen A. Gilje is the Contract Manager.

Sufficient copies of this report are being distributed to provide a minimum of one copy to each FHWA regional office, division office, and State highway agency. Direct distribution is being made to the division offices.

for D. Solomon
Charles F. Scheffey
Director, Office of Research

NOTICE

Technical Report Documentation Page

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16. Abstract

Aggradation and degradation are long-term changes in stream channel elevation. The effects of gradation changes are not the same as local scour or erosion because they extend greater distances along the stream-bed. Degradation is a more common problem than aggradation and in general, has a more severe impact on highway crossings. Although gradation changes do occur naturally, human activities are responsible for the most severe cases. Channel alteration, streambed mining, and the construction of dams and control structures are the major causes of gradation problems. Every alluvial river in the U.S., has a potential for gradation change. The prevalence of human activities as chief cause of gradation changes means many river suffer to some degree. To aid in the anticipation of gradation changes the highway engineer should be aware of the principles of geomorphology and river mechanics. Analysis procedures for predicting aggradation and degradation can be simple or complex. The simplest techniques are qualitative in nature, involving the application of geomorphic concepts. The most complex techniques involve the analysis of entire drainage systems using detailed dynamic computer modeling of water and sediment transport processes. Between these two extremes are numerous quantitative techniques involving geomorphic and basic engineering relationships as well as some simpler modeling techniques.

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U. S. DEPARTMENT OF TRANSPORTATION

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SUBJECT	FHWA BULLETIN
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CHAPTER I

INTRODUCTION

Aggradation and degradation are generally considered to be changes in streambed elevation caused by natural factors and events or human activities. A study of countermeasures for hydraulic problems at bridges by Brice, Blodgett, et al. (1978) considered 224 sites.* Volume II of the countermeasures report (Report No. FHWA RD-78-163) lists the 224 case histories in numerical sequence by site number. Each case history includes data on bridge, geomorphic, and flow factors; a chronological account of relevant events at the site; and an evaluation of hydraulic problems and countermeasures. These case histories emphasize the effectiveness of various countermeasures used at sites having problems of local or general scour. Of these 224 case histories, 39 (17.4 percent) were actually experiencing hydraulic problems related to gradation changes. This significant incidence of gradation-related problems at highway bridges indicated that further research was needed.

The first phase of an extensive investigation of aggradation and degradation was reported by Keefer, McQuivey, and Simons (1980) (Report No. FHWA/RD-80/038). The data base that was presented contained 110 case histories of bridge crossings at which gradation problems exist. The data base was analyzed to determine the geographic extent, nature, and cause of gradation problems. It contained information on remedial measures and their success or failure. Preliminary consideration was given to the recognition of gradation problems at existing bridge crossings. A general appraisal of current technology for calculating gradation changes was included.

This report describes the second phase of the investigation of aggradation and degradation. It first summarizes the significant results and findings reported by Keefer, McQuivey, and Simons (1980) and then presents a general discussion of the river systems, the relationships between grade change problems and highway crossing design considerations, the general information required for analysis of degradation and aggradation such as geomorphic and engineering concepts, the methods for determining gradation changes, the applications of methods available for estimating gradation change rates, and the analysis of countermeasures to control gradation changes. Several case histories documenting the recommended methodologies are also presented.

*References are presented after Chapter X.

DEFINITIONS

Understanding aggradation-degradation and its relationship to highway river crossings requires a shift in point of view from the immediate vicinity of the crossing to a broad perspective of the river. Traditionally, bridge design is focused at the crossing site. Historical records are examined and flood flows determined. A bridge opening capable of passing the flood flows is designed. The stream channels are occasionally straightened or modified in some way to accommodate the required structure. In streams flowing over bedrock this approach is adequate; in alluvial streams it may prove inadequate.

Streambeds composed of erodible material are either in delicate states of equilibrium with the flowing water or are unstable. The channel moves slowly and persistently from side to side and, more important to this study, vertically in response to changing discharge and sediment load. Fixing the channel at a point in space by placing approach abutments and piers in the flood plain and channel often results in undersirable side effects such as deep scour and/or channel widening as the stream tries to balance the discharge and sediment load. The sometimes disasterous consequences of these side effects have now been recognized and research has been undertaken to allow for their consideration in the design of new bridges and in the protection of existing structures.

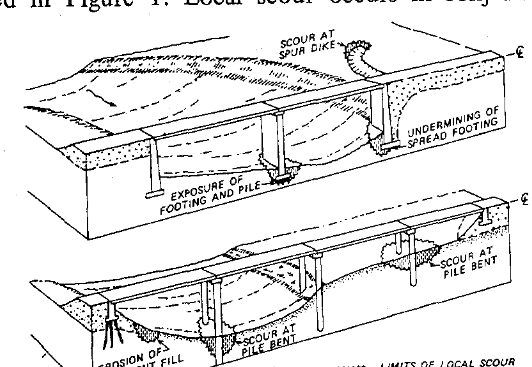
A complete description of the relationships of highways to the alluvial river environment is presented by Richardson, Simons, et al. (1974). The report clearly indicates that successful hydraulic design on an alluvial river requires a broad spatial perspective. For example, the width of a meandering river may only be several hundred meters. However, over the life of a typical bridge it may be necessary to consider the width of the stream meanders. The meander width may be three to twenty times the river width [Leopold et al. (1964)]. Meanders migrate slowly downstream and, upon encountering a highway crossing, can cause a variety of problems. The slow downstream movement of meanders emphasizes that design in alluvial rivers requires a broad perspective in time as well as space. Similarly, vertical movement of a channel is slow and extends for long distances. In the late 1800s a huge natural log jam called the Great Red River Raft was removed by the U.S. Army Corps of Engineers from the Red River below Shreveport, Louisiana. The raft had retarded flow in the river for decades. Its removal accelerated the flow and resulted in more than 5.8 m of degradation taking place at Shreveport over 50 years. The degradation extended upstream more than 161 km along the river. Vertical stream morphology changes take place slowly but well within 30- to 50-years life of a bridge. It is necessary to look at where the river or channel bed has been and where it is now and anticipate its position in the future. Aggradation and degradation are phenomena that can also take place on an historically stable channel as the result of human activities. It, therefore, becomes important to conduct research on proposed man-made modifications to the hydrologic system.

Changes in bed level that may affect highway bridge crossings can be described by considering three types of interrelated phenomena: local scour, general scour, and aggradation and degradation.

Local Scour

Local scour occurs in the bed of the channel around the base of piers, embankments, and similar obstructions and is due to the action of vortex systems induced by obstructions to the flow as illustrated in Figure 1. Local scour occurs in conjunction with or in the absence of degradation, aggradation, and general scour. Local scour does not normally occur over the entire channel width. The basic mechanism causing local scour is the vortex of fluid resulting from the pileup of water on the upstream edge and subsequent acceleration of flow around the nose of the pier or embankment. The action of the vortex is to erode bed materials away from the base region. If the transport rate of sediment away from the local region is greater than the transport rate into the region, a scour hole develops. As the depth is increased, the strength of the vortex is reduced, the transport rate is reduced, equilibrium is reestablished, and scouring ceases.

Local scour is normally associated with a single flow or flood event and can develop in a few hours or days.

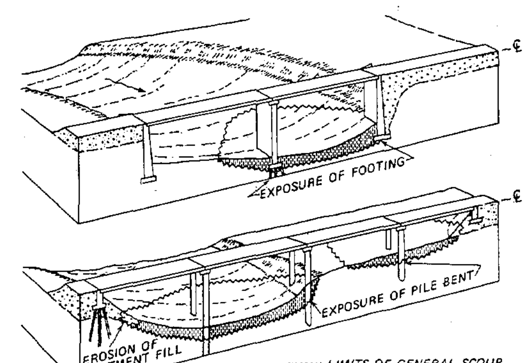


Source: Brice, Blodgett, et al. (1978)

Figure 1. LOCAL SCOUR AND RELATED HYDRAULIC PROBLEMS AT BRIDGES

General Scour

General scour occurs at contractions caused by bridge abutments, spur dikes, embankments, and debris accumulation at bridge openings as illustrated in Figure 2. It is caused by a reduction in flow area resulting in an increased average velocity and bed shear stress. Hence, there is an increase in stream power at the contraction; more bed material is transported through the contracted section than is transported into the section. As the bed level is lowered, the velocity decreases, shear stress decreases, and equilibrium is restored when the transport rate of sediment through the contracted section is equal to the incoming rate. General scour normally will occur over the entire channel width and is



Source: Brice, Blodgett, et al. (1978)

Figure 2. GENERAL SCOUR AND RELATED HYDRAULIC PROBLEMS AT BRIDGES

associated with a single flow or flood event lasting only a few hours or days.

Aggradation and Degradation

The terms aggradation and degradation are not defined in precisely the same way by engineers, geologists, and geomorphologists. The differences in the definitions come from defining the limits of the perspective used to view the river in time and space. The purpose of this section is to define the terms as they are used in this report and contrast the definitions with terms often used to describe vertical stream changes.

Leopold et al. (1964) provide a definition of aggradation and degradation from the viewpoint of the geomorphologist. They also introduce into the definition the terms scour and fill. These are the two terms most likely to be confused with aggradation and degradation.

> "With the rise in stage accompanying flood passage through a river reach, there is an increase in velocity and shear stress on the bed. As a result the channel bed tends to scour during high flow. Because sediment is being contributed from upstream, as the shear decreases with the fall of stage the sediment tends to be deposited on the bed or the bed fills. Channel scour and fill are words used to define sedimentation during relatively short periods of time, whereas the terms degradation and aggradation apply to similar processes that occur over a longer period of time. Scour and fill involve times measured in minutes, hours, days, perhaps even seasons, where aggradation and degradation apply to persistent mean changes over period of time measured in years."

Contrast this with Simons and Senturk (1977) in an engineering text where the following is provided:

> "A river is stable when the geometry of a cross section is constant in time. If the bottom level increases in elevation the streambed is aggrading. If the bottom level decreases in elevation the streambed is subject to degradation."

Simons and Senturk omit consideration of the time frame.

Aggradation and degradation are generally considered to be long-term (over time periods ranging from days and months to years) changes in streambed elevation caused by natural factors and events or human activities. For the purposes of this report, the definitions of aggradation and degradation are based upon a detailed study of many case histories and encompass both the Leopold et al. (1964) and Simons and Senturk (1977) defintions as illustrated in Figures 3 and 4. This detailed study of case histories of gradation changes indicates that most changes are due to human activities and that significant gradation changes normally take place with the first significant flow event. These local gradation changes can take place in a few days, weeks, or months depending on the duration of the flow event. The significant impact on the river system is human activities, such as, channel alterations, land-use changes, streambed mining and excavation, construction activities, damming, and reservoir regulation, etc. The river system is the most unstable during the first year, and at that time the gradient of the stream will make a significant adjustment or gradation change in the

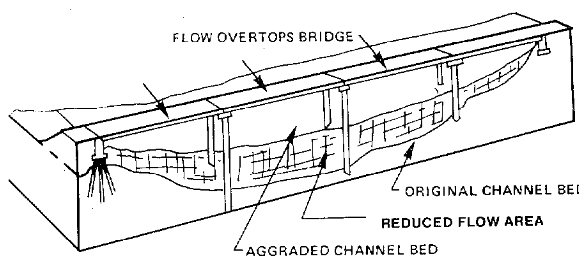


Figure 3. AGGRADATION AND RELATED HYDRAULIC PROBLEMS AT BRIDGES

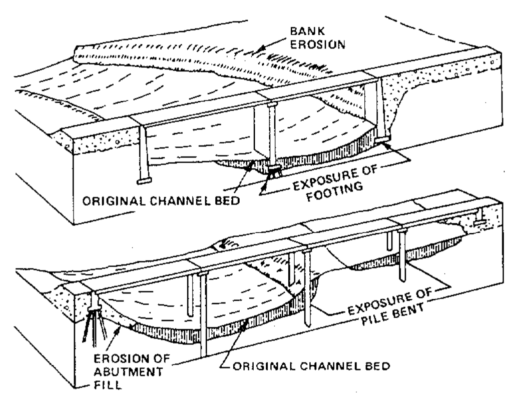


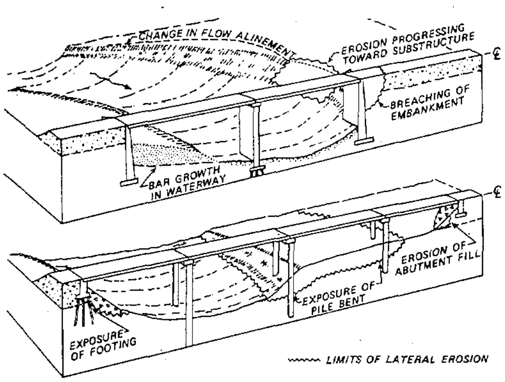
Figure 4. DEGRADATION AND RELATED HYDRAULIC PROBLEMS AT BRIDGES

location. These gradation changes may then continue at a slower rate over periods of time measured in years. It may then take years for the gradation problem or change to impact the river system many miles upstream or downstream or a tributary many miles away.

Spatial perspective also enters into the definition. For the purpose of this report, the terms scour and fill will be associated with changes in bed elevation that take place over distances no greater than one to three channel widths. Degradation and aggradation will be associated with changes occurring over many channels widths. The perspective in viewing problems will often extend for kilometers both upstream and downstream of a bridge crossing.

The large spatial perspective will minimize discussions of ripples, dunes, and other alluvial forms of roughness elements. The roughness elements must be considered when assessing computational methods but will not play a major role in the discussion.

Lateral stream erosion at a bridge can be a consequence of aggradation and degradation. The typical effects of lateral stream erosion are illustrated in Figure 5.



Source: Brice, Blodgett, et al. (1978)

Figure 5. LATERAL STREAM EROSION AND RELATED HYDRAULIC PROBLEMS AT BRIDGES

OBJECTIVES AND SCOPE

Objectives

This study has three major objectives. The first objective is to create a data base that is national in scope containing case histories of highway bridge crossings affected by gradation changes. The second objective is to analyze the data base to determine (a) the regional extent of any gradation problems, (b) causes of gradation problems, (c) national impact of gradation problems on highway crossings, and (d) mitigative measures and their degree of success or failure. The third objective is to assess the technology available to highway engineers for evaluating gradation problems. The assessment includes ways to recognize gradation problems, to analyze them, to take possible mitigative measures at existing structures, and to account for gradation changes in newly designed structures.

These objectives have been divided into two phases. The first phase was designed to meet the first two objectives and conduct preliminary research for meeting the third objective. Report No. FHWA/RD-80/038 presents the results of the first phase activity. The second phase was designed to meet the third objective; this final report covers that objective and reviews the results of the first two objectives.

Scope

The general scope of the research presented in this report was implied in the definitions of aggradation and degradation. Specifically, the study will concentrate on changes in river gradation that occur over both short and long periods of time and that take place over distances many times the stream width. Local and general scour caused by passage of single flood events will not be considered, nor will the passage of small-scale alluvial bed forms such as ripples, dunes, or bars (except as required for analytical purposes). These phenomena have been covered by others [ASCE Task Committee (1975), Richardson, et al. (1974)]. Rivers with histories of gradation change will only be considered if they have significantly affected highway river crossings or have the potential to do so.

Several research areas were included in the scope of Phase I. Most important of these areas was the creation of a case history data base. This data base was developed from records kept by highway engineers throughout the United States. The second area was an analysis of the data base. The analysis was designed to determine the regional extent of gradation problems and their impacts on highway crossings. An important aspect of the analysis was to determine the cause of problems. Specifically, the relative magnitude of man-related versus natural problems was determined. Damming and gravel mining are examples of human impacts; fault shifts and base level changes on alluvial fans are examples of natural causes.

The final area included in Phase I was a general appraisal of the technology related to gradation problems. A complete, annotated bibliography of past and current research was developed. Methods for (a) recognizing gradation problems, (b) calculating rates and limits of gradation changes, and (c) determining design and effectiveness of remedial measures were reviewed and evaluated. The evaluation completed as part of Phase I was designed to identify potentially useful methodology. Documentation of the techniques and their specific applications to highway problems are included in this report.

Phase II of the research effort is covered in this report. Included in the scope of Phase II are research activities designed to provide highway engineers with means to include gradation changes in design considerations. Also included are means for recognizing and selecting remedial measures for gradation problems at existing bridge sites.

The design portion of Phase II includes consideration of the effect of gradation changes on water surface profiles, local and general scour, debris loads, and the potential for lateral movement. Analytic tools are presented for computing rates and limits of gradation changes. Example applications are presented in detail using the data base from Phase I.

The basis for the remedial measures portion of Phase II is the data base from Phase I. Various methods used to control gradation are evaluated for use in different circumstances, with cost-effectiveness being an important factor.

ROLE OF ANNOTATED BIBLIOGRAPHY

The annotated bibliography of the literature, information, and studies reviewed provides a description of many stream reaches that have undergone gradation problems. These case histories obtained from the literature and studies of greatest interest have been listed and discussed in detail in the Phase I report. From the literature and the state highway departments, it was not difficult to document more than 100 case histories. In fact, numerous potential case histories were eliminated because of time and money constraints. As a result, not all possible case histories in the literature have been documented or annotated.

The annotated bibliography provides information that is valuable to state highway personnel to evaluate problems caused by gradation changes. It includes procedures for recognizing potential changes before they become major problems and procedures for predicting gradation changes, including engineering restraints imposed by progressive gradation changes and guidelines for the design of mitigative measures.

CASE HISTORY DATA BASE

General Description

The usefulness and validity of this study depends to a large degree on the detailed evaluation of specific aggradation/degradation sites reported as case histories in Phase I and selected case histories from other reports. Examples of such other reports are Brice, Blodgett, et al. (1978), Keeley (1967 and 1971), URS/Ken R. White Company (1975), and State Department of Highways and Public Transportation, Paris, Texas (1976).

Research Procedures

The request for bridge sites with gradation problems resulted in more than 200 possible sites being provided by state highway agencies. Published reports provided another 75 possible sites that required verification and updated data. From these sites, 110 were selected for further documentation. Specifically the sites were selected to

- determine the probable causes of gradation changes,
- establish the relative significance of human activities on river gradation,
- provide a regional description of degrading and aggrading streams,
- determine highway problems related to gradation changes,

- establish and evaluate guidelines and methods to recognize gradation problems,
- determine and evaluate mitigative measures that have been used by state highway agencies, and
- provide a data base to examine and evaluate methods for prediction of gradation changes required in Phase II.

Very few sites provided all the items listed above. Some sites were selected to illustrate specific causes of gradation changes (natural or human-induced). Some were selected because of excellent historical data, and others illustrate various types of mitigative measures.

All physiographic regions of the United States are represented in the case histories. Consideration of 200 possible sites plus information obtained during the field site visits were necessary to provide a better regional description of degrading and aggrading streams.

Organization of Data Base

Documentation of case histories involved a consideration of many factors. To standardize data collection and analysis, a standard format for the case histories was used. This standard format ensured that all relevant factors were considered by each of the several investigators involved in data collection, field site visits, and analysis. This case history format was divided into four sections, following the format used by Brice, Blodgett, et al. (1978).

Description

Each case history title includes the name of the river being crossed, the designated highway number, and the name of the nearest city or town. A brief description of the geomorphic and river characteristics includes the channel pattern, whether if it is a perennial or an ephemeral stream, the streambed slope, the bed material, the bank shape, the bank material, the channel width, and a statement about the overbank channel and/or floodplain. A brief description of the bridge characteristics includes the year constructed, its length, the angle of skew of the river, the bridge's structural foundation, the abutment type, and the type of deck construction. The hydrologic characteristics are briefly described by designating the drainage area above the highway crossing and the bankfull discharge when such information is available.

Gradation Problem

Each case history contains a chronological sequence of events related to the gradation changes. Also listed are the probable causes of the gradation problem. The hydrologic and geomorphic implications are discussed by documenting the bed elevation changes. Documentation includes historical data collected at or near the bridge site, photographs, and, when available, streambed profile information upstream and downstream of the highway crossing.

Countermeasures

Where possible, a chronological summary of mitigative measures is described and their effectiveness evaluated. Photographs are included to illustrate the problems caused by gradation changes at highway crossings. In many cases, countermeasures were applied at highway crossings to mitigate other problems. These countermeasures are described to evaluate their impact on the gradation problem.

Discussion

This section provides an assessment of the gradation problem in both time and space. An evaluation of the mitigation measures is presented, or if no mitigative measures were implemented, recommendations are made. Because many factors are involved at each highway crossing, it is sometimes necessary to discuss other stream problems and other mitigative measures applied at the site along with their impacts on the gradation problem.

Sources of Information

Information on the chronological sequence of gradation changes, the bridge characteristics, and mitigative measures was generally obtained from the agency responsible for the bridge. Construction plans, maps, and aerial photographs were obtained when available at each site.

In several cases, useful maintenance information was obtained from district highway offices, but in most districts, maintenance records were not adequate to provide the detailed information necessary. Hydrologic and geomorphic information was obtained from publications of files of the U.S. Geological Survey (USGS). To fully assess human activities it was often necessary to obtain supplemental information from other state and federal agencies responsible for the design, construction, and/or operation of facilities that have had an impact and influence on gradation or bed elevation changes (dams, gravel mines, etc.).

List of Case Histories in This Report

Table 1 is a complete list of the case histories documented in Phase I. Each case history title includes the name of the river being crossed, the designated highway number, and the name of the nearest city or town.

These 110 case histories are a significant portion of the existing data base on gradation problems and their impact on highway crossings.

Neither sufficient time nor funds were available to document and visit the sites of all the potential case histories provided by the state highway agencies. However, data from the undocumented sites are an important part of the data base and were valuable in assessing the regional extent of gradation problems. This unpublished information is summarized by state in the interim report by Keefer, McQuivey, and Simons (1980).

Data Obtained From Other Reports

The remaining portion of the data was obtained from published reports. Brice, Blodgett, et al. (1978) documented 39 case histories of gradation problems. Table 2 is a list of the sites documented in *Countermeasures for Hydraulic Problems at Bridges, Volume II, Case Histories for Sites 1-283.* These documented case histories provided additional information about the impact and extent of streambed elevation changes on highway crossings.

Table 1. DOCUMENTED CASE HISTORIES

Case History	Location
1	Rattlesnake Wash at I-40 Kingman, Arizona
2	Oraibi Wash at SR-264 near Old Oraibi, Arizona
3	Little Colorado River at U.S.-666 near St. Johns, Arizona
4	Little Colorado River at SR-77 near Holbrook, Arizona
5	Fries Wash at I-40 near Kingman, Arizona
6	Quartzite Canyon at U.S.-60 Arizona
7	Santa Cruz River at I-19 near Sahuarita, Arizona
8	Rillito Creek at I-10 near Tucson, Arizona
9	Rillito Creek at U.S.-89 near Tucson, Arizona
10	Avondale Wash at SR-85 near Phoenix, Arizona
11	Holy Moses Wash at U.S.-66 near Kingman, Arizona
12	Walker Creek at U.S.-160 near Mexican Water, Arizona
13	St. Francis River Floodway (Ditches 60 & 61) at U.S.-63 near Marked Tree, Arizona
14	Red River at SH-41 between Forman and Fulton, Arkansas
15	Burnt Cane Lake at S.H.-38-50 near Widener, Arkansas
16	Crow Creek at I-30 near Forest City, Arkansas
17	Sulphur River at U.S.-71 near Fort Lynn, Arkansas
18	Boeuf River at S.H.-144 near Lake Village, Arkansas
19	Stillwater Creek at S.R.-299 near Redding, California

Table 1. DOCUMENTED CASE HISTORIES (Continued)

Case History	Location
20	Smith River at U.S.-101 near Crescent City, California
21	San Diego River at S.R.-67 near Lakeside, California
22	Cuyama River at S.R.-166 near Santa Maria, California
23	Kelsey Creek at S.R.-89 near Kelseyville, California
24	Mad River at S.R.-299 near Blue Lake, California
25	Hosler Creek at S.R.-96 near Hoopa, California
26	Broaddus Creek at S.R.-20 near Willits, California
27	Wildcat Creek near Ft. Morgan in Morgan County, Colorado
28	15th Street Bridge over the South Platte River, Denver, Colorado
29	Cruz Gulch at U.S.-24 near Colorado Springs, Colorado
30	Little Missouri River at U.S.-22 north of Killdeer, North Dakota
31	Small stream at S.R.-24 south of Fort Yates, North Dakota
32	Tributary to East Fork of the Nishnabotna River at S.H.-37 near Defiance, Iowa
33	Culver Creek Tributary to Boyer River at S.H.-37 near Dunlap, Iowa
34	Mosquito Creek at S.H.-191 near Portsmouth, Iowa
35	Silver Creek at I-80 near Shelby, Iowa
36	Big Whiskey Creek at U.S.-20 near Lawton, Iowa
37	Iowa River at S.H.-14 near Marshalltown, Iowa
38	Allen Creek at S.H.-127 near Manolia, Iowa
39	Graybill Creek at S.H.-92 near Carson, Iowa
40	One Hundred and Two River near Gravity, Iowa
41	Floyd River near Jame, Iowa
42	Kansas River near Bonner Springs, Kansas
43	Stone House Creek, U.S.-24 and U.S.-59 at Williamstown, Kansas
44	Middle Fork of Beargrass Creek at I-64, Jefferson County, Kentucky
45	Poor Fork of the Cumberland River at U.S.-119, Harlan County, Kentucky
46	Chadwick Creek at I-64, Boyd County, Kentucky
47	Taylor Creek I-471, Campbell County, Kentucky
48	Pond Creek at U.S.-119, Pike County, Kentucky
49	Amite River at S.H.-37 near Grangeville, Louisiana
50	Cool Creek at I-55 near Kentwood, Louisiana
51	Whitten Creek at S.R.-37 at Baywood, Louisiana
52	Lawrence Creek at S.R.-16 near Franklinton, Louisiana
53	Comite River at S.H.-64, East Baton Rouge Parish, Louisiana
54	West Pearl River at I-59, St. Tammany Parish, Louisiana
55	East Pearl River at I-10, St. Tammany Parish, Louisiana
56	Mississippi River at I-494, South St. Paul, Minnesota
57	Yalobusha River at S.H.-9 near Calhoun City, Mississippi
58	Perry Creek at I-55 near Grenada, Mississippi
59	Batupan Bogue at S.H.-8 at Grenada, Mississippi
60	Black Creek at S.R.-7 near Avalon, Mississippi
61	Pigeon Roost Creek at S.R.-305 near Lewisburg, Mississippi
62	Homochitto River at S.R.-33 at Rosetta, Mississippi
63	Tillatoba Creek at S.H.-35 at Charleston, Mississippi
64	Kimsey Creek at I-29 near Mound City, Missouri
65	Middle Fork Grand River S.H.-46 near Grant City, Missouri
66	Davis Creek at I-70 near Sweet Springs, Missouri
67	West Gallatin River at I-90 near Bozeman, Montana
68	Niobrara River at S.R.-12 near Niobrara, Nebraska

Table 1. DOCUMENTED CASE HISTORIES (Continued)

Case History	Location
69	Muddy Creek at S.R.-50 in Otoe County, Nebraska
70	South Fork Little Nemaha River at S.R.-50 near Cook, Nebraska
71	Logan Creek at S.R.-9 near Pender, Nebraska
72	Elk Creek at S.R.-15 near Jackson, Nebraska
73	Little Nemaha River County Road near Unadilla, Nebraska
74	Small Creek at U.S.-73 south of Decatur, Nebraska
75	Papillion Creek at S.R.-64, U.S.-6, S.R.-92, I-50, S.R.-370 near Omaha, Nebraska
76	East Branch Pemiqewasset River near Lincoln, New Hampshire
77	Arroyo Seco at U.S.-84 near Espanola, New Mexico
78	Chupaderos Arroyo at S.R.-30 near Espanola, New Mexico
79	Washes at S.R.-78 near Velarde, New Mexico
80	White Water Creek at U.S.-180 near Glenwood, New Mexico
81	Caddo Creek at S.H.-53 west of Milo, Oklahoma
82	North Fork of Walnut Creek at U.S.-62 near Blanchard, Oklahoma
83	Wallowa Lake Bridge over Grande Ronde River near Island City, Oregon
84	Pacific Highway West Bridge over the Willamette River near Harrisburg, Oregon
85	Mt. Hood Highway Bridge over White River, Oregon
86	South Fork Forked Deer River at U.S.-51 near Halls, Tennessee
87	Cane Creek at U.S.-51, S.R.-19, CR-8044, near Ripley, Lauderdale County, Tennessee
88	North Sulphur River at F.M-68 near Paris, Texas
89	Rowdy Creek at F.M.-38 near Paris, Texas
90	Merrill Creek at S.R.-34 near Ladonia, Texas
91	Merrill Creek at F.M-1550 near Ladonia, Texas
92	Baker Creek at F.M-1550 near Ladonia, Texas
93	Mallory Creek at F.M.-137 near Howland, Texas
94	Cherry Creek at F.M-1184 near Howland, Texas
95	Weber River at I-80 N between Echo and Riverdale, Utah
96	Salina Creek at I-70 near Salina, Utah
97	Virgin River at I-15 near Bloomington, Utah
98	East Fork of the Virgin River at U.S.-89 near Mt. Carmel, Utah
99	Boxelder Creek at I-25 near Glenrock, Wyoming
100	Carpenter Creek at S.R.-192 near Sussex, Wyoming
101	Cole Creek at S.R.-192 near Sussex, Wyoming
102	Unnamed Draw on Kaycee - Mayoworth Road near Kaycee, Wyoming
103	Elk Creek at S.R.-789 near Basin, Wyoming
104	Alkali Creek at Ralston-Badger Basin Road near Powell, Wyoming
105	Old Badwater Bridge over Badwater Creek near Shoshone, Wyoming
106	Tenmile Draw Creek at I-80 near Point of Rocks, Wyoming
107	Cheyenne River at S.R.-63 south of Eagle Butte, South Dakota
108	Big Elk Creek Bridge #47-080-535 at I-90 South Dakota
109	Polo Creek Bridge #41-162-082 at I-90 South Dakota
110	Bear Butte River Bridge #47-015-427 at I-90 South Dakota

Table 2. DOCUMENTED CASE HISTORIES OF GRADATION PROBLEMS FROM BRICE, BLODGETT AND OTHERS (1978)

Site No.	Location
1	Cache Creek at I-505 near Madison, California
7	Stony Creek at I-5 near Orland, California
24	Yuba River at S.R.-20 near Smartville, California
27	Fishing Creek at L.R.-19026 at Light Street, Pennsylvania
39	Trinity River at AT and SF Railroad Bridge near Lavon, Texas
42	Fishing Creek at S.R.-487 at Orangeville, Pennsylvania
52	Pigeon Roost Creek at S.R.-305 near Lewisburg, Mississippi
85	Sand Creek at Quebec Street Bridge and I-270 at Denver, Colorado
88	Henrys Fork at S.R.-88 near Rexburg, Idaho
90	Boise River at Fairview Avenue at Boise, Idaho
123	Grande Ronde River at S.R.-82 at Island City, Oregon
125	Cow Creek at I-5 near Azalea, Oregon
148	Merrill Creek at F.M.-1550 near Ladonia, Texas
159	Boeuf River at U.S.-82 near Lake Village, Arkansas
168	Pfeiffer Creek at S.R.-25 near Batesville, Arkansas
170	Red River at S.R.-41 near Forman, Arkansas
173	Sulphur River at U.S.-71 near Fort Lynn, Arkansas
174	Homochitto River at S.R.-33 at Rosetta, Mississippi
176	West Fork Crooked Creek at S.R.-206 near Gaither, Arkansas
178	Soldier River at S.R.-37 at Soldier, Iowa
179	Soldier River at Monona County Bridge near S.R.-183 1.5 miles north of Soldier, Iowa
180	East Nishnabotna River at U.S.-34 near Red Oak, Iowa
181	East Nishnabotna River at S.R.-48 near Red Oak, Iowa
193	Lawrence Creek at S.R.-16 near Franklinton, Louisiana
198	Whitten Creek at S.R.-37 at Baywood, Louisiana
205	Mississippi River at I-494 at South St. Paul, Minnesota
212	Tuscolameta Creek (North Canal) at S.R.-35 at Walnut Grove, Mississippi
218	Tallahatchie River (Fort Pemberton cutoff) at U.S.-82 at Greenwood, Mississippi
227	Middle Fork Grand River at S.R.-46 near Grant City, Missouri
239	Elkhorn River at U.S.-30 at Arlington, Nebraska
242	Logan Creek at S.R.-9 near Pender, Nebraska
246	South Fork Little Nemaha River at S.R.-50 near Cook, Nebraska
247	South Platte River at U.S.-83 at North Platte, Nebraska
261	South Fork Forked Deer River at U.S.-51 near Halls, Tennessee
266	Boulder Creek at Bridge 394-2.4, Mt. Baker-Snoqualmie National Forest, Washington
282	Homochitto River at U.S.-61 near Doloroso, Mississippi

Four additional publications provided detailed documentation on gradation problems. In two reports by Keeley (1967 and 1971), 28 case histories of general highway problems in Oklahoma are documented. Several of these case histories are highway crossing problems resulting from streambed elevation changes.

A third report, *Sulphur River Degradation Survey, District 1, Fannin, Delta and Lamar Counties, Texas,* was prepared by the State Department of Highways and Public Transportation, Paris, Texas (1976). The report documents several case histories of degradation from 1930 to 1976.

The fourth report, *Bridge Foundation Investigation and Scour Study – South Platte River and Cherry Creek at Denver, Colorado,* was prepared by URS/Ken R. White Company, Denver, Colorado (1975).

Several other reports provided documentation on aggradation and degradation sites and are listed in the annotated bibliography.

Method of Study

The three major research efforts in Phase II were to

- determine the effects of gradation on the complex of other highway bridge design considerations,
- develop methods for predicting gradation changes at highway stream crossings, and
- evaluate mitigative measures for controlling and designs for alleviating stream gradation problems.

The general research procedures and approaches used will be described at this time.

Case History Data Base

Sutron worked closely with the Contract Manager, Federal Highway Administration, to develop the case history data base. The research method was based somewhat on the experience of Brice, Blodgett, et al. (1978). The search for suitable data was initiated through a letter from the Contract Manager to Regional Federal Highway Administration officials. The regional offices, in turn, requested state highway offices to forward data on bridge sites with gradation problems. These responses were forwarded to the Contract Manager and jointly reviewed with Sutron. States with large numbers of gradation problems were selected for visits. Sutron met with local highway engineers and viewed the sites while collecting all available information on the history, effect, and remedial measures related to the gradation changes. Special forms were used to ensure a uniform level of information from each site. The information from the site visits was cataloged to form the data base.

Assessment of Technology

Assessment of technology required gathering and reviewing a large number of published abstracts and papers. Several methods were used to locate information.

A major source of information was the Office of Water Research and Technology's Water Resources Scientific Information Center. There, a computer search was conducted of all keywords related to gradation problems. These keywords included scour, fill, aggradation, degradation, erosion, and a number of others. More than 500 pages of annotated bibliography related to sediment transport were obtained.

A second major source of information was personal libraries. Mr. Stephen Gilje, Contract Manager, provided many useful documents. Drs. Daryl Simons and Stanley Schumm of Colorado State University contributed many references. Dr. Raul S. McQuivey, the Sutron principal investigator, also provided papers and data.

Several data sources were also used. State highway engineers were quite helpful in providing references manuals and papers. The principal investigator obtained a number of useful references from other highway researchers through contact at technical meetings. A National Technical Information Service (NTIS) information search similar to that at the Office of Water Research and Technology's Water Resources Scientific Information Center also yielded useful references. The U.S. Geological Survey's National Headquarters Library was a valuable source of reference documents.

Countermeasures

For the purpose of this report, countermeasures are defined as any measures that serve to control or prevent hydraulic or stream-related structural problems at highway crossings. For this study, countermeasures of interest are those that protect crossings specifically from grade changes within a river system. These grade changes become evident in the form of aggradation, degradation, and lateral instability.

The classification and analysis of the effectiveness of countermeasures are based on a data base containing more than 200 case histories documenting hydraulic problems at crossings. The data base is made up of the 110 case histories documented as a part of this study, 86 of the case histories reported by Brice, Blodgett, et al. (1978) in Report No. FHWA/RD-78/163 and other reports listed in the annotated bibliography.

Of the case histories in the overall data base, 106 documented the use of some countermeasure to correct a hydraulic problem. These cases were first classified by the grade change type causing the problem (aggradation, degradation, or lateral erosion) and then further classified by the type of countermeasure used to control the problem.

In addition to evaluating countermeasures built at existing bridges, those built as parts of new crossings were considered. Of particular interest is the effect of various new stream crossing design practices that are intended to provide high safety factors against grade changes. These practices include minizing the number of piers or other obstructions located in the stream channel, constructing very deep foundations, providing extensive bed armoring, etc.

Applications

Several documented case histories were selected to illustrate in detail the objectives of the research project. The case histories selected were at sites distributed geographically throughout the United States, and they represented different examples of gradation problems. The methods available for estimating gradation rates, maximum depths, and highway engineering consequences to stream crossing are discussed in sufficient detail that state highway engineers can follow the procedures as guidelines. These methods range from very simple techniques that require little field data and manpower to fully dynamic water and sediment transport models that require extensive field data and a significant amount of manpower with mathematical modeling skills.

Organization of Report

The remainder of this report is divided into the following eight sections:

- highway problems related to gradation changes and the regionalization of problems,
- relationships between gradation problems and highway crossing design considerations,
- the river system,
- geomorphic and engineering concepts and relationships,
- methods of determining gradation changes,
- application of methods,
- countermeasures: classification and analysis of effectiveness, and
- summary and conclusions,
- recommendations, a glossary, and a list of references.

Highway Problems Related to Gradation Changes and the Regionalization of Problems

This chapter summarized the contents of the case history data base. Four major subsections discuss the general analysis approach, man's effects, natural effects, and combined effects. The regional limits of gradation problems documented in the continental United States are discussed and illustrated.

Relationships Between Gradation Problems and Highway Crossing Design Considerations

The purpose of this chapter is to present an analysis of other hydraulic hazards at highway crossings associated with grade changes. First a description of the types of grade changes is presented; next a discussion of the causes of grade changes is summarized; and finally, impacts of grade changes on specific design consideration at bridges and culvert crossings are reviewed.

The River System

This chapter presented a description of the natural development of river systems and the response of those systems to both man and natural induced changes.

Geomorphic and Engineering Concepts and Relationships

This chapter discusses information required for analysis including the geomorphic and engineering concepts necessary to understand the physical processes that govern a river system.

Methods of Determining Gradation Changes

Three basic solution procedures for predicting and designing gradation changes are presented in this chapter: (a) qualitative, involving geomorphic concepts, (b) quantitative, involving geomorphic concepts and basic engineering relationships, and (c) quantitative, involving sophisticated mathematical modeling concepts.

Application of Methods

In this chapter, several methods and levels of gradation change analysis are discussed. They are then applied to several case histories to illustrate the analysis approach that can be applied to existing and new highway crossings with possible gradation problems.

Countermeasures: Classification and Analysis of Effectiveness

This chapter presents the classification and effectiveness of countermeasures to control or prevent aggradation, degradation, and lateral instability. It is based on a data base containing 334 case histories documenting hydraulic problems at highway crossings.

Summary and Conclusions, Recommendations, Glossary, and References

The final chapters present the summary and conclusions, a list of the major recommendations, a detailed glossary of technical terms used throughout the report, and a complete list of references.

CHAPTER II

HIGHWAY PROBLEMS RELATED TO GRADATION CHANGE AND REGIONALIZATION

GENERAL DESCRIPTION AND APPROACH

This chapter presents the analyses of gradation problems from the case history data base. First, the types of gradation problems are described; then, the case history data base is analyzed to show the causes of gradation problems and a tabulation that summarizes the problem categories is given; and finally, the regionalization of gradation problems based upon the case history data base is presented.

TYPES OF GRADATION PROBLEMS

The typical effects of aggradation and degradation are illustrated in Figures 3 and 4, respectively. One problem associated with grade changes is lateral erosion (illustrated in Figure 5) in which the channel widens to a width greater than the bridge opening or attempts to outflank the bridge opening.

Aggradation

The highway crossing problem most associated with aggradation, as illustrated in Figures 6, 7, and 8, is reduction of flow area, which results in possible flow over the bridge deck or culvert embankment. Highway traffic is immediately disrupted or the potential for disruption exists. In addition to the potential for disruption of traffic, it is also possible that the bridge or embankment will be swept away as a result of an increase in horizontal force and turning movement. The reduction in flow area also increases the backwater produced by the crossing at lower, more frequent discharges. Aggradation at bridge crossings also results in expensive maintenace costs; it becomes necessary to excavate the deposited material in the flow area upstream and downstream of the bridge to provide the necessary flow area to pass the design flow.

Aggradation changes crossing conditions such

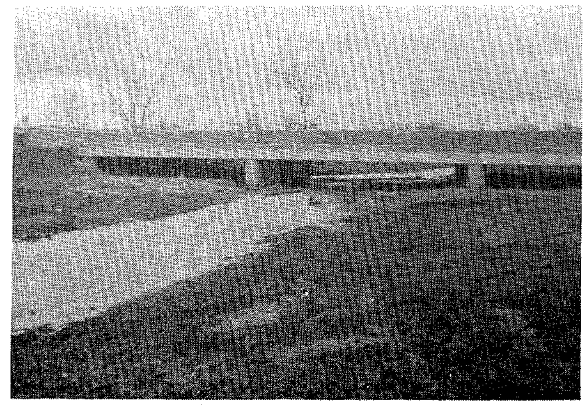


Figure 6. AGGRADATION ON THE LITTLE COLORADO RIVER AT US-666 NEAR ST. JOHNS, ARIZ.

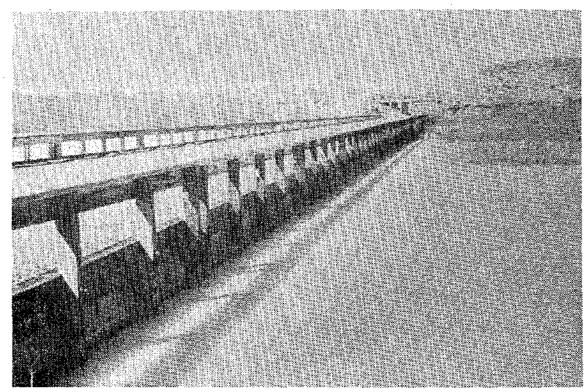


Figure 7. AGGRADATION ON BADWATER CREEK AT HIGHWAY 20 NEAR SHOSHONE, WY.

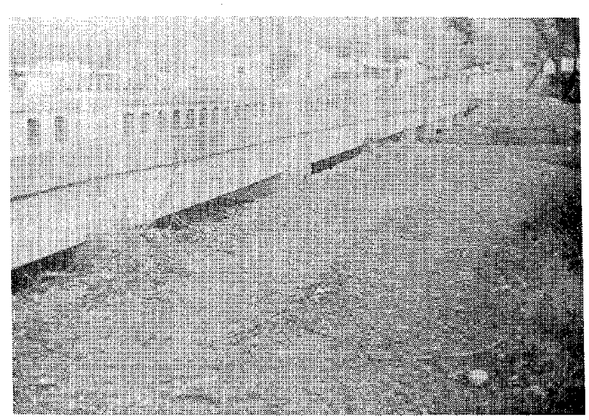


Figure 8. AGGRADATION ON WHITEWATER CREEK AT US-62 NEAR GLENWOOD, N. M.

that other stream hazards (which under original stream conditions caused no problems) are more dangerous to the structure. An example is the increased hazard from floating debris and ice. With the reduced flow area at a crossing, there is an increased possibility of frequent debris buildup causing an increase in horizontal force and turning moment. In such a case, aggradation at the site would be the major cause of failure.

Degradation

The highway crossing problems associated with degradation are the exposure of footings (illustrated in Figure 9), the exposure of pile bents, the erosion of the abutments (illustrated in Figures 10 and 11), and the undermining of cutoff walls (illustrated in Figure 12) and other flow-control structures. Degradation also undermines bank protection (shown in Figure 13), resulting in instability of channel banks, and increases debris problems. Ultimately degradation can result in the loss of the crossing structure (illustrated in Figure 14) and, in fact, is responsible for the loss of many bridges and culverts throughout the United States.

Degradation changes crossing conditions such that other stream hazards (which under original stream conditions caused no problems) are more dangerous to the structure. For example, a local scour of 1 m would be no problem under design conditions; it could, however, cause bridge failure when degradation has lowered the general streambed a few meters. In such a case, although local scour is often stated as the cause of failure, degradation is the major contributor to the failure.

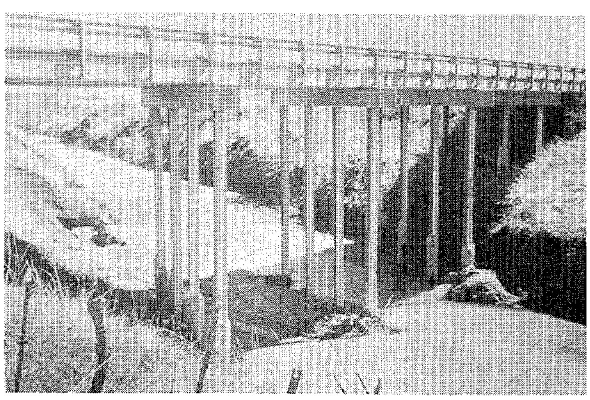


Figure 9. DEGRADATION ALONG NORTH SULPHUR RIVER, TEX.

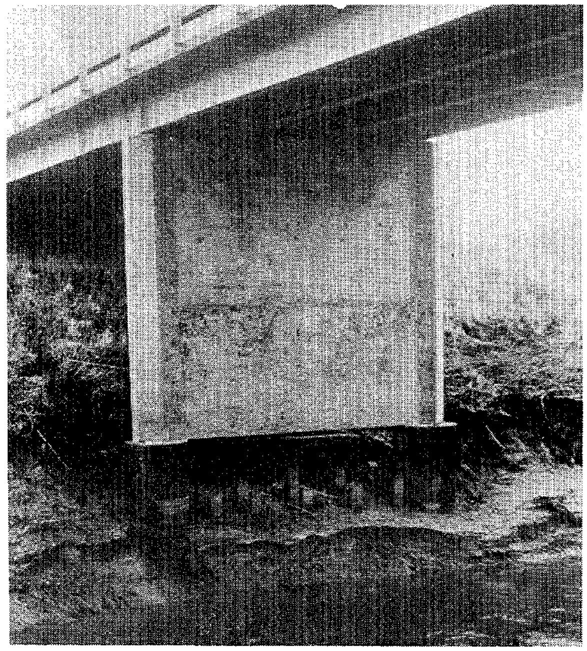


Figure 10. DEGRADATION ON THE LITTLE NEMAHA RIVER NEAR UNADILLA, NEB.

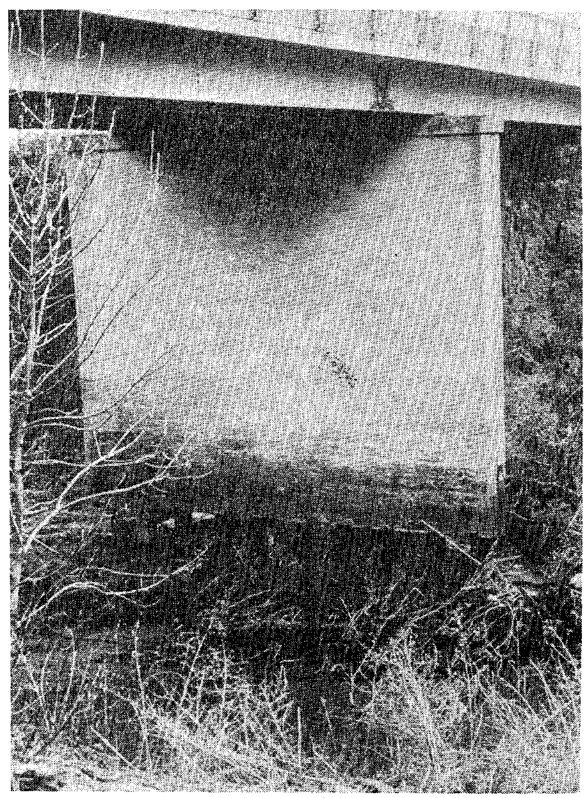


Figure 11. DEGRADATION ON SMALL CREEK AT US-73 SOUTH OF DECATUR, NEB.

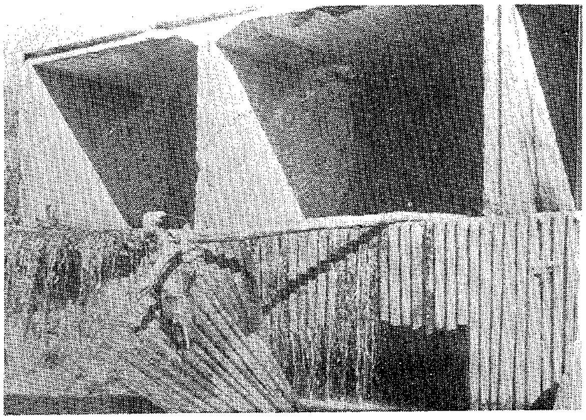


Figure 12. UNDERMINING OF CUTOFF WALL AT SH-199, OKLA.

Figure 13. UNDERMINING OF BANK PROTECTION AT SR-78 NEAR VELARDE, N. M.

Figure 15 shows a degradation problem and a debris problem and local scour. All of these factors are an important consideration to this highway crossing.

Lateral Erosion

Lateral erosion, often a consequence of aggradation or degradation, was documented in 14 case histories. Aggrading streams can become braided, and the movement of individual braids can cause the movement of a bank line at the crossing site. Often aggradation causes alternate bar deposition, which enhances lateral migration through a tendency toward a higher degree of sinuosity. Lateral erosion also accompanies degradation in the form of bank sloughing and increased meander activity as illustrated in Figures 16 and 17. This natural meandering activity serves to reduce the channel bed slope back to an equilibrium condition.

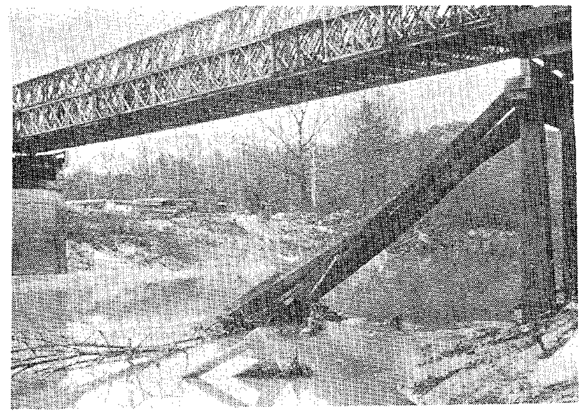


Figure 14. BRIDGE FAILURE AT SR-8 OVER BATUPAN BOGUE NEAR GRENADA, MISS.

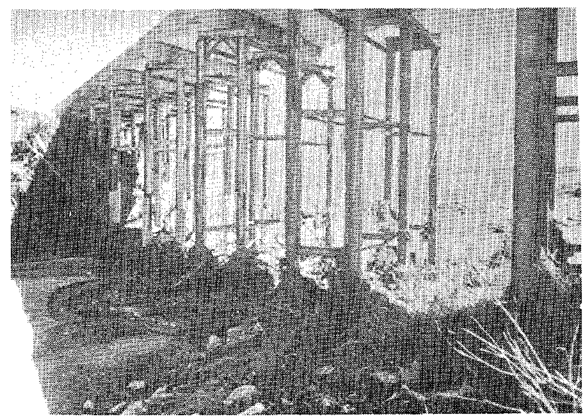


Figure 15. DEGRADATION ON THE SULPHUR RIVER, TEX.

Figure 16. BANK SLOUGHING ~~ON~~ PERRY CREEK AT I-55 NEAR GRENADA, MISS.

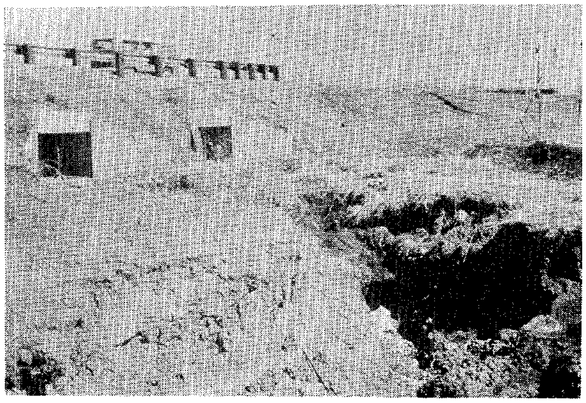


Figure 17. BANK SLOUGHING ALONG CRUZ GULCH NEAR COLORADO SPRINGS, COL.

Summary

Aggradation and degradation problems may range in degree from rather trival, requiring only routine maintenance for correction, to complete failure of the structure. It is difficult to set definite limits within these extremes. However, most of the problems documented in the case histories are between the extremes, in that they present a definite hazard to the bridge and are subject to corrective action less drastic than replacement of the structure. For bridges that have been swept away in catastrophic floods, it is usually difficult to decide which part failed first, what exactly caused the failure, what practical countermeasures might have been taken to prevent the loss, and whether it would have been prudent to design for such an event.

ANALYSIS OF CASE HISTORY DATA BASE

An analysis of 110 case histories was included in Report No. FHWA/RD-80/038 and is merely summarized here.

Classification by Aggradation, Degradation, and Lateral Erosion

Of the 110 grade problems documented (Table 3), 81 may be classified as degradation cases (about 75 percent) and 29 as aggradation (about 25 percent); 15 of these problems were also classified as lateral erosion. None of the cases documented involved lateral erosion alone. Channel aggradation is apparently not as common a problem as degradation. Also, aggradation in a bridge waterway does not reduce the stability of foundations or become a problem until the waterway area or bridge or clearance is reduced below the minimum value needed to convey the design flood.

Degradation is among the more common causes of hydraulic problems at bridges. Brice, Blodgett, et al. (1978) reported only one aggradation site and 39 degradation sites. The results of this study indicate that there are about three serious degradation sites for every serious aggradation site.

Table 3. TYPES OF GRADATION PROBLEMS

Gradation Problem	Case History Number
Aggradation	3, 4, 8, 10, 12, 22, 25, 30, 37, 40, 44, 45, 47, 48, 60, 66, 67, 68, 77, 78, 79, 80, 85, 101, 105, 106, 107, 108, 109
Degradation	1, 2, 5, 6, 7, 9, 11, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 26, 27, 28, 29, 31, 32, 33, 34, 35, 36, 38, 39, 41, 42, 43, 46, 49, 50, 51, 52, 53, 54, 55, 56, 57, 58, 59, 61, 62, 63, 64, 65, 69, 70, 71, 72, 73, 74, 75, 76, 81, 82, 83, 84, 86, 87, 88, 89, 90, 91, 92, 93, 94, 95, 96, 97, 98, 99, 100, 102, 103, 104, 110
Lateral Erosion	8, 14, 56, 59, 62, 63, 65, 69, 70, 73, 74, 75, 78, 82, 84

Classification of Cause

Gradation changes that have an impact on highway crossings can be classified into those resulting from (a) natural causes or factors and (b) human activities. An analysis of the case histories indicates that very few gradation changes were caused by natural factors. Some should perhaps be classified as being caused by a combination of natural and man-induced factors, but they are so few that a separate category is not warranted. Because man's activities dominate the causes for gradation problems, they will be discussed first.

Human Activities

Some of man's activities have had far-reaching consequences on streams and have caused or contributed to aggradation and degradation problems at bridges. An analysis of the case histories reveals that human activities have been the major cause of aggradation and degradation problems at bridges. Gradation problems at bridges caused by human activities can be classified as

- channel alterations,
- land use changes,
- streambed mining/excavation,
- construction activities, and
- damming and reservoir regulation.

Each of the five categories can be described more specifically with subcategories (Table 4).

Channel Alterations

Straightening, dredging, clearing and snagging, artificial constrictions, and other alterations to natural channels are the major causes of streambed elevation changes, with channel straightening being the dominant activity as illustrated in Figure 18.

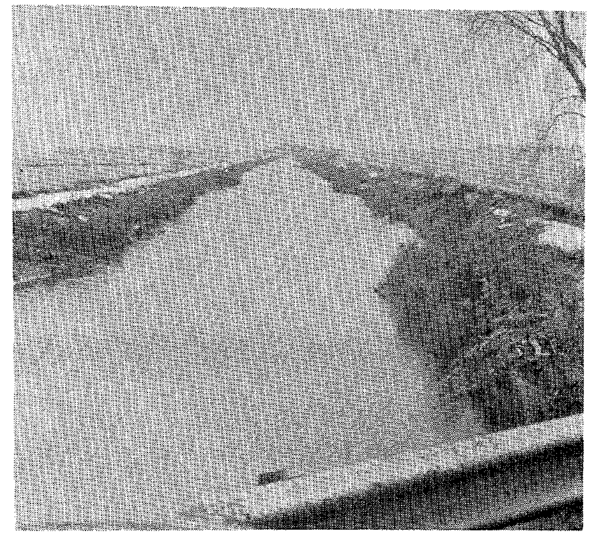


Figure 18. STRAIGHTENED CHANNEL ON LOGAN CREEK AT SR-9 NEAR PENDER, NEB.

Straightening of natural channels, principally to improve drainage for agricultural purposes, has been widely practiced in the United States since about 1900. The documented case histories indicate that this practice has had an impact on many streams and highway crossings in Nebraska, Iowa, Texas, Tennessee, and Mississippi and to a lesser degree in Kansas, Oklahoma, Missouri, and Arkansas. Table 4 indicates that 41 of the case histories with gradation problems at highway crossings were caused by channel straightening. Data exist to document another 50 to 100 case histories in which gradation problems were the direct result of channel straightening. Channelization of streambeds at highway crossings is currently done on a small scale, and an attempt is made to keep stream slopes similar. In addition, highway engineers are cautious about stabilizing the reaches that are changed. Although there are some examples of streambed channelization done by highway agencies that have resulted in degradation, highway engineers have been generally successful in preventing gradation problems [Brice, Blodgett, et al. (1978)].

Many of the straightened channels have degraded, and degradation is usually accompanied by channel widening, unstable banks, and serious debris problems. The degradation is attributed to an increase in channel slope that results from shortening of channel length. The increase in channel slope increases the velocity and the shear stress on the bed. As a result, the channel bed degrades until it becomes armored or until the channel widens and begins to meander to reduce the channel slope back to an equilibrium condition. The degrading bed and unstable banks also deliver more debris to the bridge site. The mechanics of degradation are discussed further in the following chapters.

Table 4. SIGNIFICANCE OF MAN'S ACTIVITIES ON RIVER GRADATION

Activity	Cause	Case History Number
Channel Alteration	Drainage/Dredging	51, 53, 56
	Channelization/Straightening/ Cutoffs	2, 15, 16, 18, 27, 32, 33, 34, 35, 36, 37, 38, 39, 41, 46, 50, 53, 57, 58, 61, 62, 64, 65, 69, 70, 71, 72, 73, 75, 81, 82, 84, 86, 87, 88, 95, 96, 98, 99, 102, 106
	Clearing/Snagging	87
	Constrictions	3, 4, 19, 25, 26, 27, 44, 61, 79, 84, 102
	Structure Alignment/Design	8, 10, 22, 29, 54, 102, 109, 110
	Tributary Gradation As a Result of Mainstream Gradation	59, 64, 72, 74, 89, 90, 91, 92, 93, 94
Land Use Changes	Urbanization	29, 44, 47, 75
	Agriculture	40, 62, 82
	Strip Mining	45, 48
	Logging/Clearing	25, 86
Streambed Mining/ Excavation	Sand and Gravel	7, 8, 11, 19, 20, 21, 23, 24, 26, 42, 43, 49, 51, 53, 83, 84
	Borrow Pit	1, 5, 9
Construction Activities		47, 48, 106
Damming and Reservoir Regulation	Clear Water Releases	13, 14, 17, 41, 42, 43, 57, 72, 74, 84
	High Sustained Regulated Flows	13, 14, 17
	Backwater	12, 30, 101, 105, 107
	Low Sustained Regulated Flows	68, 84
	Dam Breach or Removal	74
	High Controlled Irrigation Canal Releases	104

Circled numbers represent aggradation sites.

Case History 62, the Homochitto River at SR-33 at Rosetta, Mississippi, is a classic example of degradation and its impact on highway crossings. A fairly complete historical background exists on the problem and the various mitigative measures employed to arrest it over many years.

Another example is Case History 86, the South Fork of Forked Deer River at US-51 near Halls, Tennessee. This case history is presented in its entirety in the Phase I report because of the completeness of the data documenting the causes, impact, and mitigative measures.

Degradation in a main channel often leads to degradation of tributaries, even where the tributaries have not been altered. Straightening and consequent degradation on the North Sulphur River in Texas led to degradation at a bridge on Merrill Creek, a tributary (Case History 91). Another example of tributary degradation is seen in Case History 72, Elk Creek at SR-15 near Jackson, Nebraska. This case history illustrates the impact of degradation of the Missouri River below Gavins Point Dam and is presented in its entirety in Report No. FHWA/RD-80/038.

Evidence indicates that degradation as a consequences of channel alteration is most rapid during a period shortly followed the alteration and continues at a decreasing rate. For example, Yearke (1971) measured the degradation following channel straightening on the Peabody River in New Hampshire. He found that the major degradation occurred in the first year and successively smaller amounts occurred in subsequent years. On the Homochitto River at Doloroso, Mississippi [Brice, Blodgett, et al. (1979)], the river course to the Mississippi River (downstream from the US-61 bridge) was shortened from about 28 km to about 15 km in 1938. Degradation began at the bridge in 1944 and reached 5 m by 1945. Continued degradation only amounted to an additional 1.3 m during the period 1945-57. However, degradation that begins in the lower reaches of a stream may continue upstream over a longer period. For example, the main wave of degradation on the Homochitto River did not reach Rosetta, Mississippi (Case History 62), 26 km upstream, until the period 1949-66.

With documented evidence that channel alterations are the major causes of gradation problems (74 case histories), it is important that highway engineers recognize the potential for impact at highway crossings.

Land Use Changes

Urbanization, agriculture, strip mining, and logging/clearing are land use activities of man that cause gradation problems. Problems in 11 of the documented case histories can be directly related to land use changes.

Cruz Gulch at US-24 near Colorado Springs, Colorado (Case History 29), is an example of the impact from urbanization on a highway crossing (Figure 19).

Natural vegetation is extremely important in maintaining channel stability on small streams. The lateral stability of most streams in the United States, particularly in agriculture or lumbering regions, has very probably been affected by the clearing of natural vegetation. Because clearing has occurred over the past hundred years, the magnitude of the effect at a particular crossing site is sometimes difficult to assess.

Strip mining in an upland area may cause aggradation and later degradation when the mining ceases. Although only two case histories were documented, many more examples are known.

Only about 10 percent of the case histories documented show gradation problems from land use changes; it is, however, a significant problem. Highway engineers should be aware of land use changes in design of new bridges and the maintenance of existing bridges.

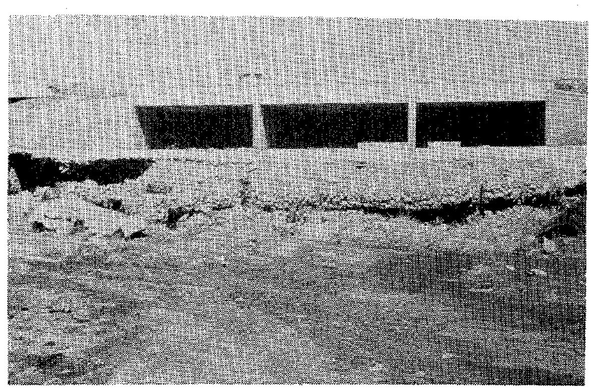


Figure 19. CRUZ GULCH AT US-24 NEAR COLORADO SPRINGS, COL.

Streambed Mining/Excavation

If sand or gravel is excavated from an alluvial channel in quantities that represent a substantial percentage of the bedload in transport, the channel will probably degrade. In addition, excavation of gravel from pits or trenches in or along the stream may result in a change in flow alignment at the bridge. Nineteen case histories illustrating these consequences are listed in Table 4. In some states, operators may legally continue to remove sand or gravel despite clear evidence of the resulting damage at a bridge.

Case History 52, Lawrence Creek at SR-16 near Franklin, Louisiana, illustrated in Figure 20, is presented in its entirety in the Phase I interim report as an example of the impact of streambed mining on highway crossing. In about 20 percent of the case histories documented, streambed mining was the major cause of a lowering of the streambed elevation.

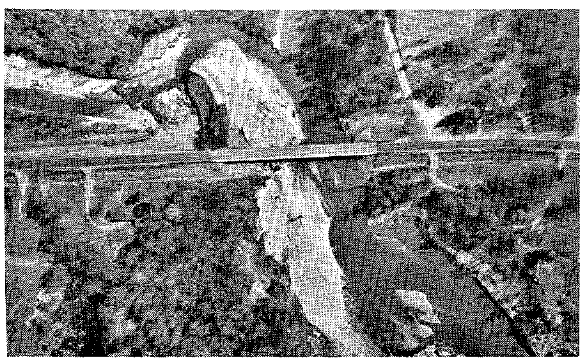


Figure 20. SR-16 BRIDGE OVER LAWRENCE CREEK NEAR FRANKLINTON, LA.

Damming and Reservoir Regulation

The effects of dams and reservoirs on a stream are complex and have not been thoroughly investigated. Twenty-two case histories illustrate the consequences of damming and reservoir regulation (Table 4). These consequences include clear water releases; high, sustained, regulated flows; backwater; low, sustained, regulated flows; dam breech or removal; and high, controlled, irrigation canal releases.

Downstream from a reservoir, channel degradation is to be expected because of removal of sediment load. This effect has been documented for many streams. The total amount of degradation can be predicted fairly accurately for sand bed streams if the channel bed material size distribution is known. This requires a rigorous field investigation at depths up to 15 m. In gravel-bed streams, aggradation may occur downstream from the dam because the flow releases are insufficient to transport gravel brought in by tributary streams. As pointed out by Kellerhals et al. (1976), channel avulsions, which can present a serious threat to many engineering structures, are associated with most aggrading situations. Rapid lowering of the river stage may result in severe bank slumping from pore-water pressures in the banks. However, the more general effect of reservoirs is probably to lessen hydraulic problems at bridges by reducing flood peaks and lateral erosion rates. An increase in stream stability has been attributed to reservoirs by Keeley (1971) for the North Canadian River in Oklahoma and by Brice (1977) for the Sacramento River in California.

An example of a problem downstream of a dam is given in Case History 72, Missouri River belows Gavins Point Reservoir, in the Phase I report. It summarizes the degradation at several locations as illustrated by the continual changing of stream stage for a given discharge.

An example of gradation problems caused by backwater upstream of a reservoir and reduced flow regulated by an upstream reservoir is Niobrara River at SR-12 at Niobrara, Nebraska. There are many examples of highway problems caused by backwater and reduction in flow by regulation, but this problem was so critical that the crossing was moved several miles upstream and an entire community was moved because of flooding and groundwater problems.

Construction Activities

Construction activities that impact the natural stability of river systems include building construction (residential or commercial), and road and highway construction. Serious erosion can occur during construction when protective vegetation is removed and steeply sloping cuts and fills are left unprotected. Such erosion can create local problems of serious downstream aggradation as the natural sediment-carrying capacity of a river system becomes overloaded. This is particularly critical to culverts and small appurtenances, especially in new developments. In the construction of major highways and interstate roads, large areas extending for miles may be exposed to erosion conditions for long periods of time. Although greater attention is currently being given to the problems of erosion from construction sites, it is important for the highway engineer to recognize the potential for such a problem when considering the stability of a river system.

Summary

Man's activities are an important factor in causing gradation problems at highway crossings. More than 80 percent of gradation problems are directly related to those activities. Highway engineers should evaluate the short- and long-term impacts of human activities on each highway crossing for both design and maintenance considerations.

Natural Causes

It was difficult to isolate case histories with gradation problems caused solely by natural factors because of the extensive activities of man. Less than 20 percent of the case histories documented could be related to natural causes. Table 5 lists the natural causes of gradation problems and their associated case histories. No attempt was made to isolate which factors started the gradation problem or which factors were dominant.

Table 5. SIGNIFICANCE OF NATURAL EFFECTS ON GRADATION CHANGES

Causes	Case History Number
Alluvial Fans	60, 85, 108, 109
Armoring	67
Braiding	67, 108
Debris	57, 58, 60, 61, 108
Meandering/Migration	8, 14, 22, 28, 62
Recurrent Flooding/High Stream Velocity	62
Delta Growth	68
Channel Bed and Bank Material Erodibility	97

Circled numbers represent aggradation sites.

Alluvial Fans

Case History 108, the Elk River at I-90 near Piedmont, South Dakota, is an excellent example of gradation resulting solely from natural casues. The case history is included to illustrate the impact of locating a highway crossing on an alluvial fan. This aggradation problem is illustrated in Figure 21.

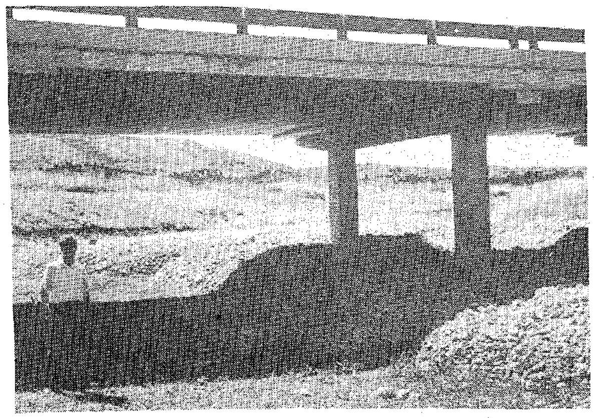


Figure 21. AGGRADATION ON BIG ELK CREEK AT I-90 IN S. DAK.

Other Factors

Other identified problems arising from natural causes and complications included natural armoring, braiding, debris, meandering/migration (natural cutoffs), recurrent flooding/high stream velocity, delta growth, and channel bed and bank material erodibility.

General Summary

The case history data base provides evidence that aggradation changes are a significant cause of problems at bridges. Almost three times as many degradation-related problems (81 cases) have been documented as aggradation-related ones (29 cases).

Human activities are the dominant causes of gradation problems. Channel alteration, principally straightening, is the primary cause. Degradation below dams is also a frequent cause of problems, as is streambed mining. Natural factors seldom cause gradation changes, with aggradation of channels under bridges built on alluvial fans being the most common problem of this type.

REGIONALIZATION OF GRADATION PROBLEMS

General

Although more than 100 case histories were considered in this study, they did not uniformly represent all regions of the country. However, certain areas of the country contributed an above average share of gradation problems, and some general conclusions can be drawn concerning their location.

Two short discussions are presented: first, gradation problems are identified with regions known to have high sediment runoff, and then gradation problems are identified with specific major river basins.

Regions of High Sediment Yield

Figure 22 illustrates the location of the documented case histories. It also identifies concentrations of sediment in major U.S. streams.

With few exceptions, the sites with gradation problems lie in areas of the country with high sediment yields. The western and central portions of the country, from a line along the western borders of Arizona and Montana to the Mississippi River, seem to be particularly prone to gradation problems. A fair number of sites are located in northwest California.

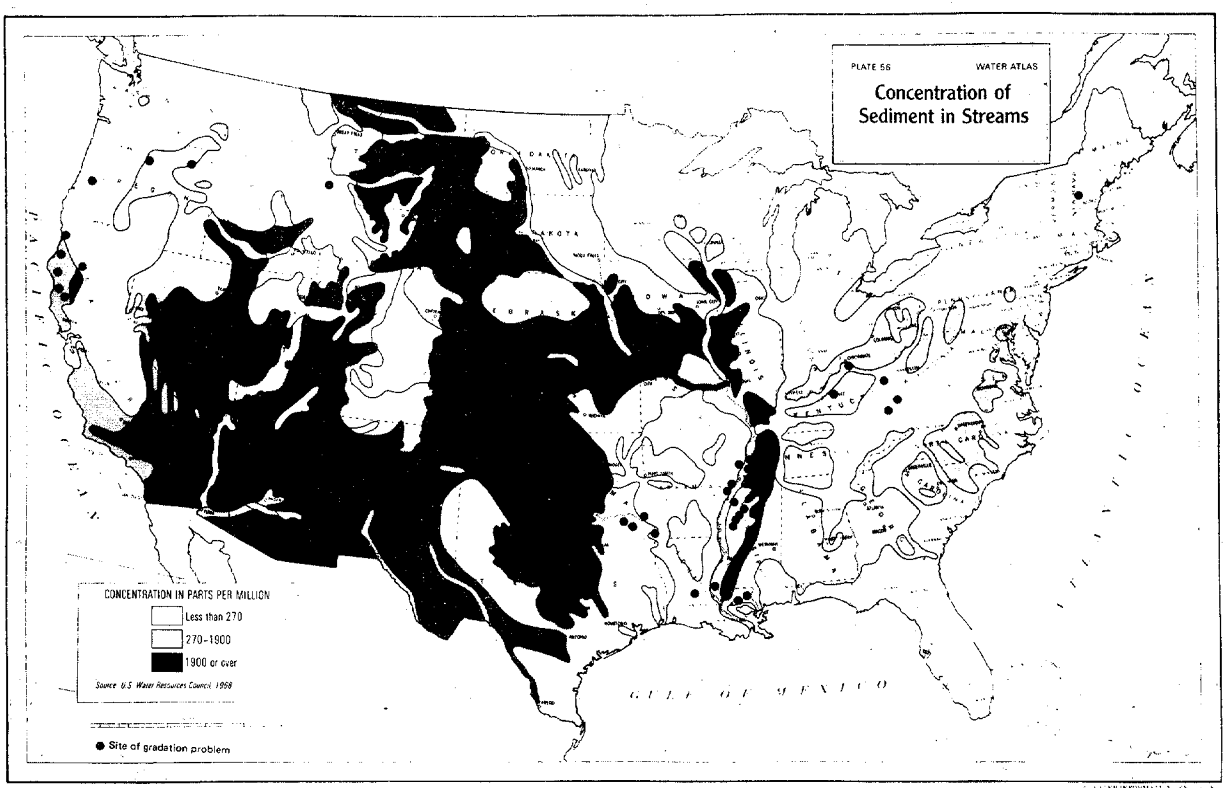


Figure 22. LOCATION OF CASE HISTORY SITES AND SEDIMENT CONCENTRATIONS

The correlation between sites with gradation problems and streams with high sediment loads is intuitively sensible. High sediment loads imply easily erodable soil and streams with sand beds. Such streams are more likely to change course or shape than streams flowing in bedrock channels or channels with very large sediment. Almost no case histories are located in the Appalachian Mountains or in the mountainous regions of the West.

The absence of streams with high sediment yield does not rule out gradation problems in such streams. The sediment yield does, however, provide a rough guide.

Major Drainage Basins

Figure 23 identifies the major river drainage basins in the United States and locates the case histories used in this study. Most of the case histories lie in three drainage basins: the Upper Mississippi, the Lower Mississippi, and the Colorado. The Red-White and Lower Missouri basins also contain significant numbers of case history sites.

Missouri Basin

The nature of the problems in the various basins varies widely. The great number of sites along the Missouri River are near the lower portion of its drainage. The extensive reservoir regulation on the Missouri has cut off its sediment load (Figure 22). In the absence of this load, the river and its tributaries are degrading rapidly.

Lower Mississippi Basin

The gradation problem sites that lie in the Lower Mississippi Basin have been primarily caused by channelization. Many stream changes made to improvde navigation have increased stream gradients and resulted in degradation.

Red-White Basin

The Red River Basin in a unique case of widespread degradation. Removal of the Great Red River

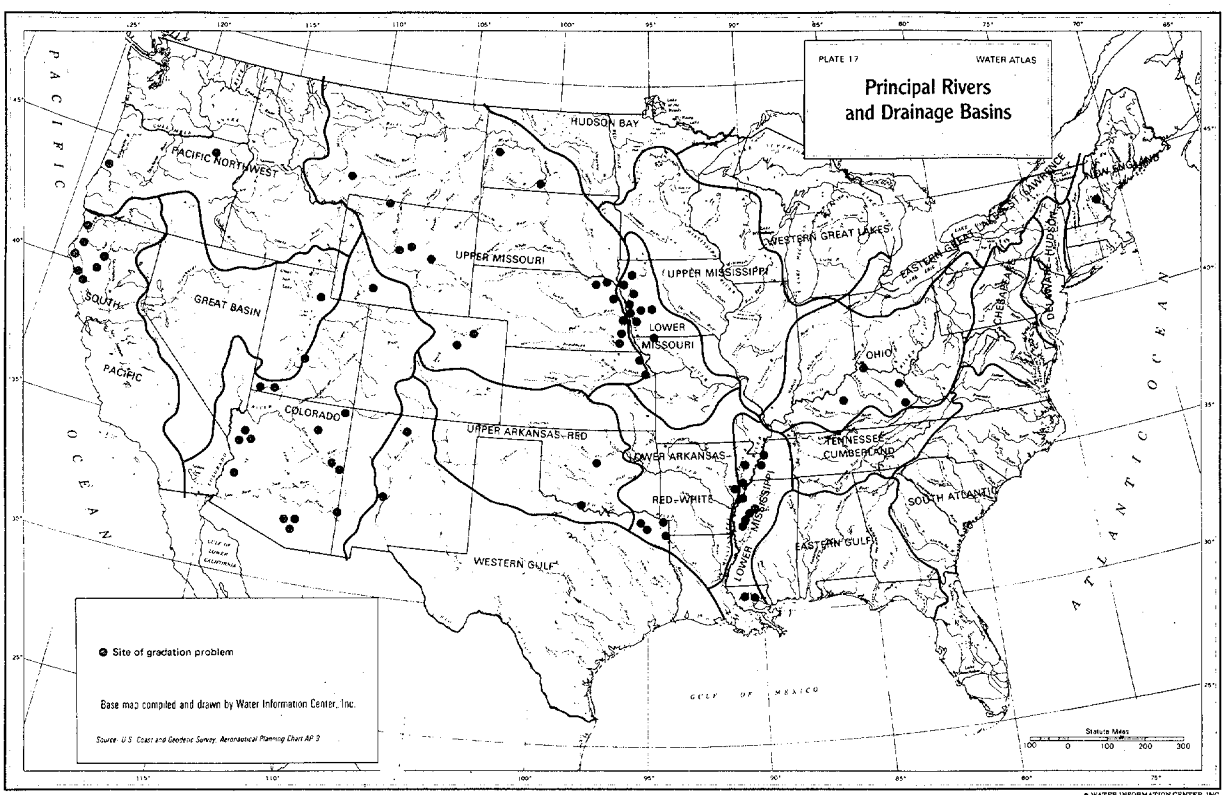


Figure 23. LOCATION OF CASE HISTORY SITES NEAR PRINCIPAL RIVERS AND DRAINAGE BASINS

Raft below Shreveport, Louisiana, resulted in massive degradation over a 20- to 30-year time span. The lowering of the Red River also accelerated degradation on its tributaries. Dams along the Texas-Oklahoma border (notably Denison Dam, Denison, Texas) have also contributed to degradation of the Red River.

Colorado Basin

No readily identifiable cause is associated with the case history sites in the Colorado Basin. The locations of gradation problems were widely scattered and usually associated with local channel straightening, damming, or other impacts.

Summary

Regionalization of gradation changes does not offer a comprehensive answer for highway engineers. It is, rather, one of several methods that provide clues to such problems or potential problems. Bridge engineers in all areas with high sediment yield should consider the possibility of gradation problems. Highway engineers should be particularly vigilant at bridges near the lower reaches of the Missouri, Red, and Mississippi rivers. Bridges in the other areas of the country should be evaluted on a case-by-case basis. If the sediment yield of a stream is found to differ appreciably from that of the surrounding terrain, grade change problems should be suspected.

CHAPTER III

RELATIONSHIPS BETWEEN GRADATION PROBLEMS AND HIGHWAY CROSSING DESIGN

GENERAL

The preceding chapter discussed the types of gradation problems experienced at highway crossings and the causes of those problems. This chapter discusses the effects those gradation problems have on specific design considerations at bridge and culvert crossings.

EFFECT OF GRADE CHANGES ON DESIGN

Highway-related designs in the river environment include design of the bridge or culvert used to convey water under or through the structure, flow control structures, channel realignments, and approach embankments. For this report, the components of river crossing design have been classified as bridge systems and culvert systems (flow control, channel realignment, and approach embankments will be considered as components of each of these). A review of current design practices documented specifically how crossing design procedures are affected by grade changes.

Federal Highway Administration design manuals and training documents were reviewed to establish the design procedures that are most affected by gradation changes at crossings and how or to what extent they are affected. Documents reviewed include HEC-5, HEC-9, HEC-10, HEC-11, HEC-13, HEC-14, HEC-15, HDS-1, HDS-3, and Richardson et al. (1974) as well as several documents relating to culvert and stable channel design published by the U.S. Army Corps of Engineers [Fletcher and Grace (1974) and Fenwick (1969)] and the U.S. Department of Agriculture (1977).

This study indicates that the parameters influenced most by grade changes are those taken as input to the design procedures and not the actual design concepts or equations and charts given in each of the reviewed documents. Among those input parameters are design discharge, channel roughness, slope, velocities, shear stresses, geometry, base levels, and flow depth. Inadequate evaluation of these input parameters is reflected in designs that are not flexible enough to accommodate the changing regime of rivers over the design life of a given structure.

As well as affecting the design parameters listed above, grade changes can also precipitate various stream-related hazards. These hazards are particularly related to the structural integrity of the crossing design. Grade changes can influence the frequency of occurrence of ice jams and debris accumulation problems, which can subject the structures to extremely high lateral forces for prolonged periods of time. Another common problem at stream crossings is the undermining of piers and abutments by degradation, lateral channel migration, and channel widening. Any of these stream hazards alone can produce costly structural damage or even complete structural failure at a crossing as evidenced in the case histories referenced in Chapter II.

To present a clear understanding of how channel crossings and associated appurtenances are affected by the various gradation changes, it is necessary to look at current design practices and point out where to expect changes or problems. To facilitate this approach, the case histories were reviewed in detail and categorized by class. For each class or crossing element, the design components affected were documented. The results are tabulated in Table 6. Case history information presented by Brice, Blodgett, et al. (1978) was also reviewed.

Bridges

The bridge system as defined in this study consists of the bridge superstructure, piers, approach embankment, bank protection, and flow-and-debris control structures.

The case history data base reveals the problems encountered at bridge crossings relating to grade changes. These problems include bridge capacity, pier and abutment alignment, footing depth at piers and abutments, and construction depth for flow-control structures. Table 6 provides a comparison of the various design parameters adversely affected at the bridge crossing sites. Only those parameters specifically referred to as being affected are listed.

Table 6. DESIGN PARAMETERS ADVERSELY AFFECTED AT BRIDGE CROSSINGS

Case History Number	Gradation Type		Design Component Affected										Others and Notes
	Aggradation	Degradation	Capacity	Flow-Through Velocity	Backwater	Pier & Abutment Alignment	Opening Width	Opening Depth	Pier Footing Depth	Abutment Footing Depth	Angle of Attack	Improved Entry Design	
1		X							X				
2		X							X	X			
3	X		X		X			X					
4	X		X		X								
5		X								X			
6		X							X				
7		X					X			X	X		
8	X	X					X			X	X		Lateral Erosion
9		X				X	X		X	X	X		
10	X		X		X		X	X					
11		X							X				
12	X		X		X			X					
13		X					X						
14		X					X		X				Lateral erosion
15		X					X		X	X			
16		X					X						
17		X					X		X	X			
18		X					X			X			Check dam undermining
19		X					X		X	X			
20		X					X		X				
21		X					X		X	X			
22	X	X	X		X						X		Major problem is aggradation
23		X					X		X				
25	X		X		X			X	X				

Table 6. DESIGN PARAMETERS ADVERSELY AFFECTED AT BRIDGE CROSSINGS (Continued)

Case History Number	Gradation Type		Design Component Affected										Others and Notes
	Aggradation	Degradation	Capacity	Flow-Through Velocity	Backwater	Pier & Abutment Alignment	Opening Width	Opening Depth	Pier Footing Depth	Abutment Footing Depth	Angle of Attack	Improved Entry Design	
26		X		X			X			X			Bridge causes constriction
27		X					X		X	X			
28		X							X				
30	X		X					X					
32		X					X		X				Culvert and drop structure used as countermeasures
33		X					X		X				
34		X					X		X	X			
35		X					X		X	X			Design anticipated those changes
36		X					X		X	X			
37	X		X		X			X					
38		X							X	X			
39		X							X	X			
40	X		X		X								
41		X											Piers on rock, no damage
42		X					X						Debris hazards
43		X					X		X	X			Debris hazards
45	X		X		X			X					
49		X		X			X		X	X			
50		X					X	X	X	X			
51		X					X		X	X			
52		X					X		X		X		Debris problems
53		X					X		X	X	X		Debris problems

Table 6. DESIGN PARAMETERS ADVERSELY AFFECTED AT BRIDGE CROSSINGS (Continued)

Case History Number	Gradation Type: Aggradation	Gradation Type: Degradation	Design Component Affected: Capacity	Flow-Through Velocity	Backwater	Pier & Abutment Alignment	Opening Width	Opening Depth	Pier Footing Depth	Abutment Footing Depth	Angle of Attack	Improved Entry Design	Others and Notes
54		X					X		X		X		
55		X					X		X	X	X		
56		X							X				Lateral erosion
57		X		X			X		X	X			
59		X					X		X	X			Lateral erosion
60	X		X		X			X					
61		X					X		X	X			Debris problems
62		X					X		X	X	X		Lateral erosion
63		X							X	X	X		Lateral erosion
64		X							X	X			
65		X							X				Lateral erosion
66	X		X	X	X			X					
67	X												No problem at site
68	X		X		X		X						
69		X					X		X	X			Lateral erosion
70		X					X		X				
71		X					X						
72		X											Has not caused problem
73		X					X		X				Lateral erosion
74		X					X		X	X			Lateral erosion
75		X					X		X	X			Lateral erosion
76		X					X		X	X			
78	X		X		X		X	X			X		Lateral erosion
79	X		X	X	X		X	X			X	X	
80	X		X		X		X	X					
81		X					X		X	X			

Table 6. DESIGN PARAMETERS ADVERSELY AFFECTED AT BRIDGE CROSSINGS (Continued)

Case History Number	Gradation Type		Design Component Affected										Others and Notes
	Aggradation	Degradation	Capacity	Flow-Through Velocity	Backwater	Pier & Abutment Alignment	Opening Width	Opening Depth	Pier Footing Depth	Abutment Footing Depth	Angle of Attack	Improved Entry Design	
82		X					X		X	X	X		Lateral erosion
83		X					X		X	X			Stable except at high Q
84		X		X			X				X		Lateral erosion
85	X		X	X	X			X					On alluvial fan
86		X					X		X	X	X		
88		X					X		X	X			
89		X					X		X	X			
90		X					X		X	X			
91		X					X		X	X			
92		X					X		X	X			
93		X					X		X	X			
94		X					X		X	X			
95		X											Degradation anticipated in design
96		X											
97		X							X				
98		X											Control structure used at constriction
98′	X		X		X			X					
99		X							X	X			
100		X					X		X	X			
101	X		X	X	X			X					
103		X					X		X	X			
104		X					X		X	X			
105	X		X	X	X			X					
107	X		X	X	X		X						
109	X		X	X	X			X					
110		X					X		X	X			Design Q exceeded

Even though a change in grade produces changes in bed slope, flow-through velocity, and flow depth, such effects are only listed when specifically referred to in the case histories. The parameters most often affected at degrading sites are opening width, pier footing or pile depth, and abutment footings or pile depth. At aggrading sites, flow-through capacity, bridge backwater, and opening depth were most often affected.

A review of current bridge design practice reveals more specifically where a design will be affected.

Design Practice

In the past, bridges were designed and constructed without the aid of a hydraulics engineer. It was considered adequate that the flow opening be designed on past experience rough calculations, individual judgment, and intuition. Any backwater produced by the bridge was considered a necessary nuisance.

In the more recent past, with the spread of urbanization and with property values increasing at an unprecedented rate, it has become imperative that the backwater produced by a new bridge be predicted and kept within reasonable limits. Another aspect of bridge construction that has become more and more important over the past two decades is economy of design. Optimum bridge width and length, number and location of piers, and depth of foundations (footings and/or piles) are becoming more and more critical as construction costs continue to escalate. All of these considerations have resulted in increasing importance being placed on the proper hydraulic design of bridge crossings.

Under current practice the hydraulics engineer interfaces with the bridge engineer in three principal areas: in designing an opening large enough to pass some design discharge without producing adverse backwater conditions; in evaluating local scour conditions to aid in establishing foundation depths; and in evaluating the need for designing bank protection and/or flow control structures and designing them when required.

Backwater

Bridge backwater is computed using an equation similar to Equation (1) with adjustments for flow in contact with the bridge deck or submergence and local scour effects [HDS-1 (1978)].

$$h_1{*}=K{*}a_2 \frac{V^2_{n_2}}{} + a_1\left[\left(\frac{A_{n_2}}{A_4}\right)^2 - \left(\frac{A_{n_2}}{A_1}\right)^2\right]\frac{V_n{}^2}{2g} \qquad (1)$$

where

$h_1{*}$	=	total backwater,
$K{*}$	=	total backwater coefficient,
a_1 and a_2	=	kinetic energy coefficients,
A_{n_2}	=	gross water area in constriction measured below normal state,
V_{n_2}	=	average velocity in constriction or Q/A_{n_2},
A_4	=	water area at section where normal stage is reestablished, and
A_1	=	total water area at the full flow section upstream of the crossing.

The variables in this expression that will be influenced by long-term grade changes are the velocities and flow areas. The total backwater coefficient ($K{*}$) and the kinetic energy coefficients will also be influenced by changing grade patterns. The kinetic energy coefficients, a_1 and a_2, are functions of the velocities at the full flow section upstream and the constricted section through the bridge, respectively. Therefore, these coefficients will change in response to velocity changes. The total backwater coefficient is a function of the various components of the crossing itself such as abutment type, pier type and configuration, angle of skew of crossings, eccentricity, and flow angle of attack as well as flow velocity and depth.

Of the two principal grade changes experienced at crossings, degradation generally produces less of an impact on backwater than aggradation. Degradation influences the variables in Equation 1 in several ways. In general, it reduces the bed and energy slope in the vicinity of the crossing, correspondingly reducing the flow velocity and resulting in slightly larger flow depths. In channels whose bed materials are composed of wide ranges of particle sizes, the reworking of this material can increase bed roughness, further reducing flow velocity and increasing flow depth. However, these changes in flow depth are minor when compared with the depth of degradation required to produce them.

Degradation is frequently accompanied by an increase in channel width that along with the increased vertical clearance produced increases the available flow area, thus reducing backwater influences created by the bridge. Degradation has also been seen to cause lateral instability, which will influence the total backwater coefficient by impacting the crossing skew, angle of attack, and eccentricity.

A significant effect on backwater conditions produced by degradation is the clogging of the bridge opening with debris. Degradation produces large amounts of debris as a result of bank failure. Two mechanisms of slope failure commonly associated with degradation are rotational failure and bank caving (Figure 24). The erosion that takes place during the degradation process unloads the base of the slope, which increases the moment of the driving weight forces and causes failure. The bank failure then deposits trees, brush, and other floating debris into the channel. When that debris becomes lodged on a bridge, it effectively reduces the cross-sectional flow area and thus the capacity of the bridge. The blockage increases energy losses through the structure creating increased flooding from the associated backwater. The debris becomes a major obstruction to flow, and bridge failure could result from the increased horizontal forces produced by excess water pressure and the floating debris.

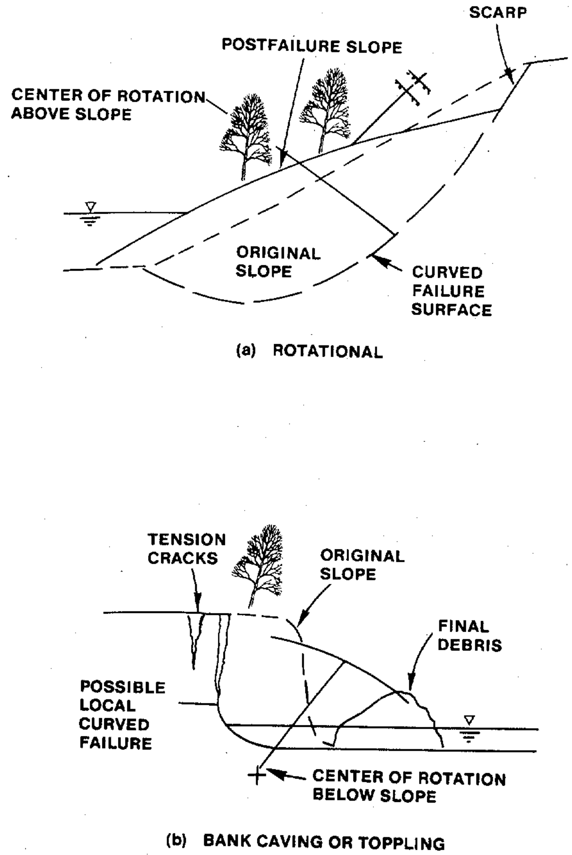


Figure 24. MECHANISM OF SLOPE FAILURE

While limited degradation produces predominantly beneficial consequences (with the exception of debris problems) with respect to backwater, aggradation produces adverse conditions. The obvious consequence of aggradation is the reduction of flow capacity through the bridge, increasing the frequency of flow contact with the bridge deck and resulting in increased backwater and more frequent inundation of surrounding areas. In these cases the bridge becomes a major obstruction to flow. However, aggradation of the magnitude necessary to totally block a stream is rare. Failure often results from the increased horizontal forces produced by both excess water pressure and floating debris. Aggradation is usually accompanied by a general steepening of the channel slope. This steepening will produce results contrary to those of degradation with respect to the variables in Equation 1, with the exception of the total backwater coefficient. Aggradation has also been known to produce lateral instability, which could influence the total backwater coefficient in the same way that degradation does.

In general, the major impact of a bridge crossing on backwater is produced by the obstruction created by the structure. At a degrading site, the flow area through the structure is increased, producing less-frequent adverse backwater effects. At an aggrading site, the flow through the area is reduced, increasing the frequency of or potential for adverse backwater effects. Along with the adverse backwater effects comes an increase in the anticipated 50- and 100-year flood limits, possibly inundating areas that previously experienced little or no flood damage. These problems are documented in Case History 37 in Report No. FHWA/RD-80/038.

Case History 37 describes the grade changes experienced along the Iowa River at SH-14 near Marshalltown, Iowa. The straightening of the Iowa River upstream of Marshalltown in the first half of this century increased the sediment-carrying capacity of that portion of the river. However, nothing was done to vary the sediment-carrying capacity downstream of Marshalltown. Thus, the added sediments are being deposited at the end of the river straightening, and the river is aggrading and the floodplain deteriorating at Marshalltown. As shown in Figure 25, the rating curve at the bridge has changed dramatically as the aggradation in the river has progressed. Because of the aggradation, Marshalltown is subject to frequent flooding.

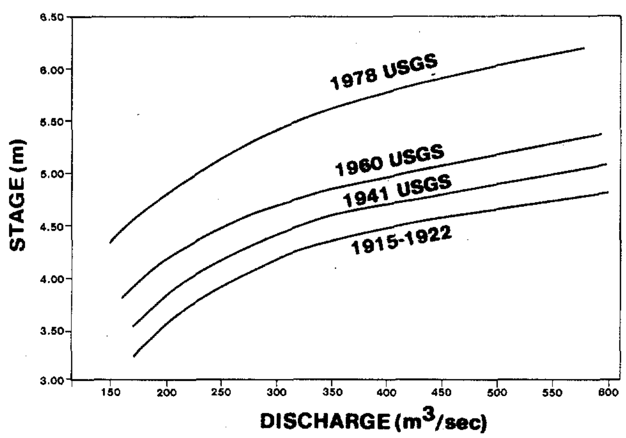


Figure 25. STAGE DISCHARGE OF IOWA RIVER AT MARSHALLTOWN, IA.

Pier and Abutment Foundation Depth

When designing pier and abutment foundations, the engineer is principally interested in the ground elevation at the foundation location and the physical properties of the bed material that will be supporting the foundation. Gradation changes at a site will not significantly affect the properties of the supporting material, but they will influence the bed elevation and thus affect the bearing capacity of a foundation located at some elevation below the surface. Both local scour and long-term general stream degradation will influence the depth at which a foundation should be placed. Local scour has been studied extensively in laboratory flumes, and techniques are available for its prediction [Richardson et al. (1974)]. Local scour problems are generally understood, and prediction techniques are a part of present bridge design practice. However, the effects of general stream aggradation and degradation have not typically been included in the design process for establishing foundation depths.

Grade changes influence the design of foundations in two ways. First, they alter the normal base elevation of the channel bed. Since local socur computations predict a depth of scour below this normal bed elevation, ignoring the long-term grade change in an analysis to determine an adequate depth for pier and abutment foundations could result in the use of an inadequate base elevation for the foundations. Long-term degradation also influences the normal hydraulic characteristics of the channel, such as slope, velocity, flow depth, degree of constriction at the bridge, bed roughness, etc. Since these parameters are the inputs to local scour depths, changes in their magnitudes will influence expected local scour depths. Aggradation at a crossing site influences the hydraulic characteristics so as to increase computed local scour depths. Degradation, on the other hand, influences the hydraulic characteristics at a crossing so as to reduce the computed local scour; however, although the computations will show less scour, its importance as a hazard is increased.

Degradation in river systems produce the most apparent consequences to foundation systems. As can be seen in Table 6, virtually all of the case histories experiencing degradation had problems associated wtih inadequate foundation depth at piers and/or abutments. Besides the vertical drop associated with degradation, the lateral instability that often comes as a consequence has been seen to undermine piers and abutments that were originally designed to be well outside the main flow channel. A good example of this undermining can be found in Case History 14.

Case History 14 describes the grade changes experienced at the SH-14 bridge over the Red River between Forman and Fulton, Arkansas. Several large reservoirs, such as the Denison Dam, have been built on the Red River and have caused channel degradation and accelerated lateral movement. This degradation and lateral movement have been mainly caused by the increased sediment-carrying capacity resulting from clear water dishcarges and increased sustained flows from the Denison Dam. Figure 26 documents the streambed migration and degradation that occurred between 1962 and 1974. The 23 m of lateral movement combined with 2.1 m of general streambed degradation at this site have exposed the foundations at Piers 4 and 5 causing the significant structural weakening of the bridge.

Aggradation, in general, will not significantly influence the depths to which bridge foundations should be constructed since any sediment buildup will effectively increase the depth to which a foundation is placed. As discussed above, the local hydraulic conditions at a crossing site will be influenced in such a way as to increase the computed local scour depth, but the relative magnitude of this increase will be overshadowed by the depth of general aggradation.

The discussion given here for bridge pier and abutment foundation depth will also hold for foundations of any flow or debris control structures constructed in connection with the crossing design.

Bank Protection

If erosion is to be prevented, the need for bank protection must be anticipated and the proper type and amount of protection provided. Bank protection is used along approach embankments to protect against lateral instability at a crossing site. This type of protection is also used in connection with the design of such flow-control structures as spurs, retards, dikes, jetties, and spur dikes. The bank protection used can be constructed from various materials such as dumped riprap, rock-and-wire mattress and gabion, concrete pavement, sacked pavement, concrete-grouted riprap, concrete-filled fabric mat, etc.

In general, designing bank protection involves (a) sizing the bank material to resist the erosive forces of the design discharge and (b) estimating the vertical and lateral extent of the protection required. The flow parameters used in satisfying these requirements are depth, velocity, and shear stress. Each of these parameters will be influenced by aggradation or degradation at the site. Aggradation will reduce flow depth somewhat while increasing velocity and shear stress, and degradation will increase flow depth and reduce velocities and shear stresses.

Degradation at a crossing site affects the design of embankment protection in several ways. First, the depth to which the embankment key is set or the length of toe will be influenced. Design recommendations currently state that "the bank protection should extend a minimum vertical distance of 1.5 m below the streambed." Allowance should also be made for any expected additional degradation by adding it to the 1.5 m for toe trenches or increasing the length of tow blanket accordingly. With degradation, some degree of channel widening usually occurs and tends to increase the embankment slope. Since embankment stone is sized after considering both velocity and bank slope, an increase in slope can reduce the resistance of the embankment to erosion. Degradation is sometimes accompanied by an increase in lateral instability, which is a reflection of the natural tendency of the stream to increase its sinuosity. Design charts presented in HEC-11 were derived assuming the flow velocity to be parallel with the bank protection. With increased sinuosity, this assumption would be invalid and an adjustment would need to be made to the size of the design stone.

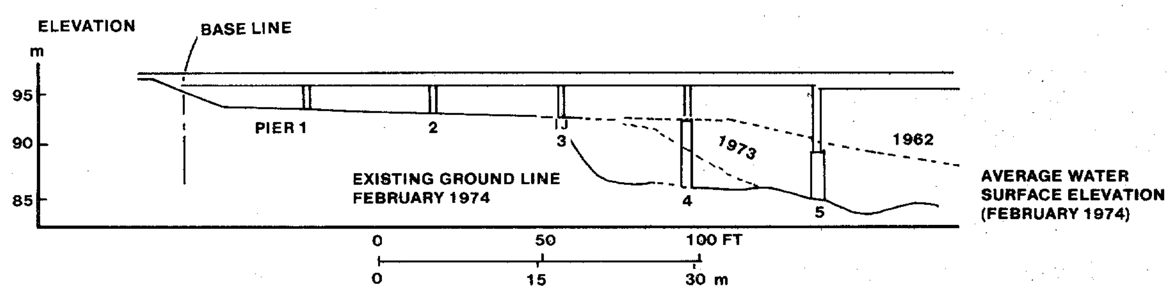


Figure 26. STREAMBED MIGRATION AND DEGRADATION AT HIGHWAY 41 BRIDGE, RED RIVER NEAR FULTON, ARIZ.

The most serious effects at an aggrading site is the frequent loss of freeboard and the overtopping of bank protection. Under these situations, failure could come from the development of flow piping at the interface between bank protection and the natural ground surface as water attempts to flow back into the channel on the recession side of the flood hydrograph. By anticipating aggradation problems at such a site, the vertical extent of bank protection can be increased (even going so far as to place small levees along the channel) and/or keys can be constructed to integrate bank protection with the natural ground surface.

Another effect at an aggrading site is the increased velocity and shear stresses associated with a slope buildup. Adequate consideration of these parameters and how they will be influenced at a site will aid the highway engineer in proper selection of a bank protection scheme.

The discussion given here for adjustments to embankment protection is also valid for embankment protection at flow-control structures constructed in connection with the crossing design.

Culverts

When considering the effects of grade changes on the design of culverts, the entire culvert design system must be considered. This larger design system includes the culvert, channel protection (both upstream and downstream), energy dissipators, and debris control structures and transition elements. In an adequate system design, the water is passed from a natural regime condition upstream of the channel crossing without upsetting the delicate balance between the hydraulics of the river flow and the physical channel properties such as bed material characteristics and channel geometry.

The problems of aggradation and degradation at a culvert crossing site arise from two distinctly separate mechanisms. First, there is the problem of inadequate design or construction of the culvert system. Inadequate design can cause either scour or deposition on either side of the crossing. This is more a local phenomenon that should not be confused with an overall change in stream morphology produced by some outside factor or factors separate from the construction of the culvert system. A brief discussion of the implications of inadequate design and/or construction of the culvert system will be given at the end of this section.

The major problem of interest here is the effect of a change in stream morphology that can alter the design characteristics of the crossing and render it ineffective in transmitting its design discharge. As with bridge crossings, the hydraulic and geometric parameters affected by changes in river regime are slope, width, depth, velocity, and angle of attack. Factors specifically related to culvert design are headwater elevation, tailwater elevation, and channel invert elevation at the crossing. Changes in these parameters will produce changes in the entrance and exit conditions at the crossing that will influence the operational capacity of the crossing design. As documented in the case histories, these changes can reduce the capacity of the system and undermine its structural integrity; they can also reduce the effectiveness of energy dissipators and/or transition elements. Any one of these effects could cause the eventual failure of the crossing. An exemplary problem is shown in Figure 27.

The case history data base reveals that changes at culvert crossings relating to grade changes include headwater elevation, tailwater elevation, capacity, entrance and exit velocities, flow-through velocity, effectiveness of modified inlet and outlet design, and depth of foundations. Table 7 provides a comparison

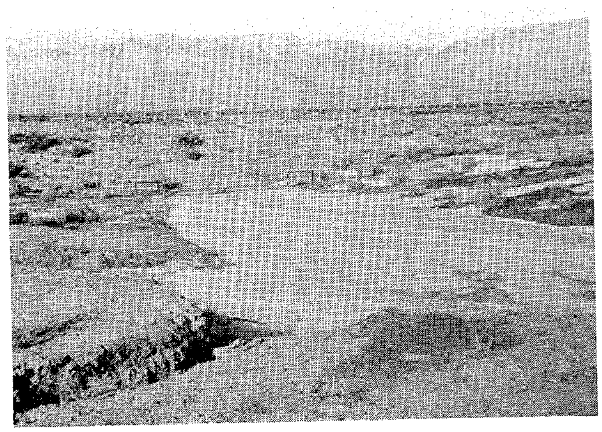


Figure 27. AGGRADATION AT CULVERT ENTRANCE, I-80 NEAR POINT OF ROCKS, WY.

Table 7. DESIGN PARAMETERS ADVERSELY AFFECTED AT CULVERTS

Case History Number	Gradation Type		Design Component Affected								Others and Notes
	Aggradation	Degradation	Headwater	Tailwater	Capacity	Flow-Through Velocity	Outlet Velocity	Inlet Design	Outlet Design	Depth to Footings	
29		X		X					X	X	
31		X		X					X	X	
44	X		X		X						Inadequate inlet design
44′	X		X		X						
47	X		X		X			X			Partially caused by inadequate design
48	X		X		X			X			
53		X	X	X			X		X	X	
77	X		X	X	X			X	X		
87		X		X			X		X	X	
102	X	X	X	X	X		X	X	X	X	Aggradation U/S inadequate design Degradation D/S
106	X		X		X			X			Design inadequate

of the various design components adversely affected at the crossing site as documented in the referenced case histories. Again, only those parameters specifically referred to as being influenced are listed. The factors most often influenced at degrading sites are tailwater elevation, outlet design, and construction depths for foundations. At aggrading sites, headwater elevation, culvert capacity, and inlet design were the most often influenced components of the culvert system. In general, the influence that grade changes have on the design of culvert systems is predominantly felt in two areas, culvert capacity and structural stability.

A review of current culvert design practice reveals more specifically where a design will be affected.

Design Practices

The objective of current culvert design practices is the hydraulic design of culverts, using improved inlets, energy dissipators, and transition elements where appropriate, that will pass a design discharge from a natural regime condition upstream of the crossing without upsetting the balance between the natural river hydraulics and the physical properties of the channel. The design involves an iterative process of selecting the proper combination of culvert barrel, improved inlet design, and outlet design. Outlet design would consist of energy dissipators, channel transitions, and bank protection. A simplified culvert design flow chart is presented in Figure 28.

The components of the system design include analysis of site characteristics, hydrologic analysis, culvert barrel design, improved inlet design, outlet design, and final analysis of the design system under varying flow conditions. The three design blocks in the flow chart (barrel, inlet, and outlet) are not separate by any means. As indicated in the figure, each component interacts with the other producing an integrated design.

Culvert design is aided by charts and nomographs presented in the Federal Highway Administration (FHWA) design manuals HEC-5, HEC-10, HEC-13, and HEC-14. Essentially, these design charts give headwater conditions produced by various culvert types and sizes, and inlet and outlet configurations. The design charts are used to construct performance curves (graphs of discharge vs. headwater elevation) for the various culvert configurations. One such curve is shown in Figure 29. The curve shows the perform-

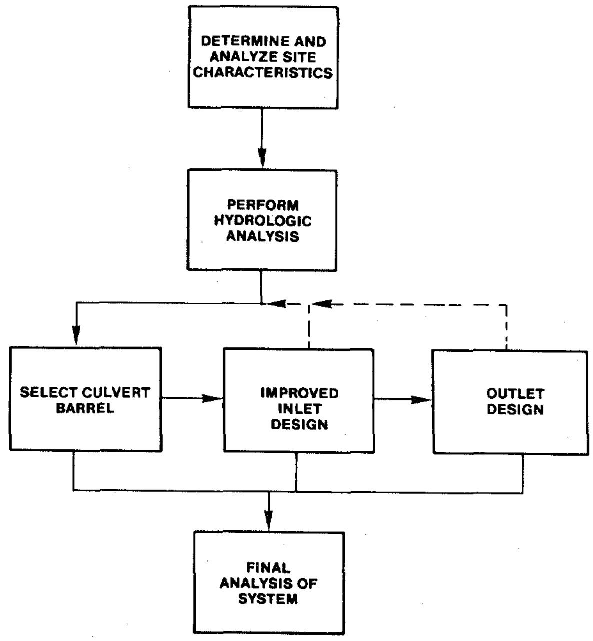


Figure 28. CULVERT DESIGN FLOW CHART

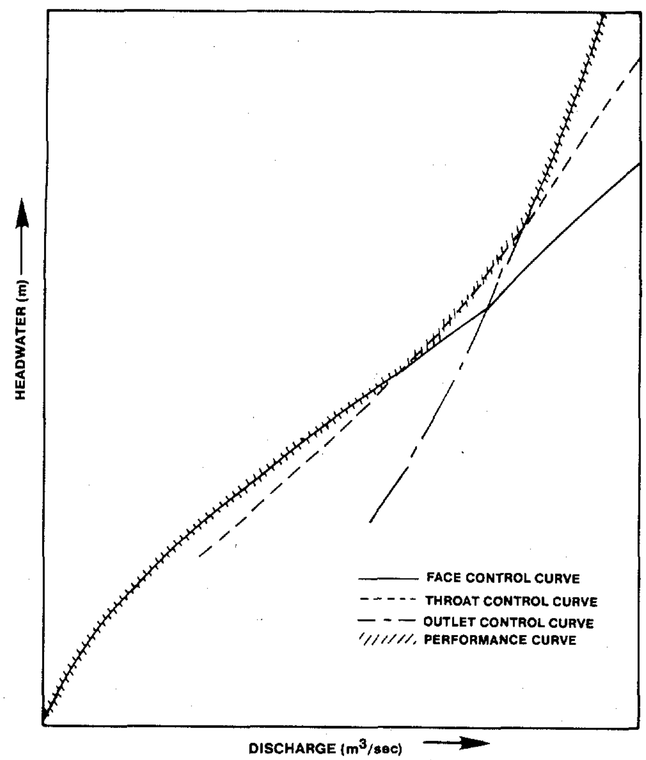


Figure 29. SCHEMATIC PERFORMANCE CURVE

ance of the culvert under the conditions imposed by face, throat, and outlet controls. The operational performance curve is shown as a combination of the various control curves indicating that under various conditions, the culvert system will operate under the influence of different controls. The design charts presented in the FHWA manuals were developed predominantly through laboratory research with some prototype evaluation and testing. Laboratory experiments are conducted in fixed bed flumes under ideal flow conditions. The experiments essentially analyze the energy losses produced by various culvert configurations by determining what flow capacity is produced by various headwater conditions. Some of the assumptions in these experiments are upstream ponding (no approach velocity), matching channel and culvert, uniform culvert and channel alignment, clear water discharges, and rigid (nonalluvial) boundaries. In an alluvial system, all of these assumptions can and generally are violated.

Essentially all the parameters used in the compilation of these design charts will be influenced, directly or indirectly, by grade changes in a river reach. The design parameters primarily impacted are headwater elevation, tailwater elevation, invert elevations at entrance and exit, culvert velocities (entrance, flow-through, and outlet), and cross-sectional flow area. The changes in these variables produced by gradation changes eventually impact culvert capacity and structural stability (Figure 30).

General aggradation trends can approach a crossing site from either the downstream direction or

Figure 30. DEGRADATION DOWNSTREAM OF CULVERT

the upstream direction. The direction is primarily determined by the condition that has produced the gradation changes. Aggradation produced by the closure of a dam moves upstream from the dam, and aggradation produced by increased sediment load from an upstream source moves downstream. In either case, the impacts on culvert design are essentially the same; flow capacity is reduced.

The most significant effect produced by aggradation is the variation in channel invert elevation just upstream and downstream of the culvert. Since the culvert elevation is fixed, it cannot respond to changes in bed level. As a result, the deposited sediments attempt to plug the culvert, leaving its capacity greatly reduced. Along with reducing the effective flow area of the culvert, this action will increase the headwater and tailwater elevations, resulting in drastically changed performance characteristics for the culvert. From the performance curve in Figure 29, the effects of this capacity loss can be visualized; with a loss in flow capacity, the performance curve would shift left and up. This shift might not be linear since it will affect each component of the curve individually. Loss of culvert capacity increases the headwater elevation and changes the design relationship between headwater and tailwater. This in turn can impact the outlet design by producing outlet conditions incompatible with those of the designed structure. The most pressing danger produced by such a condition is the damming effect produced by the embankment as the culvert clogs. Culvert clogging increases the frequency and magnitude of flooding in the vicinity of the crossings and produces an associated increase in property damage. Clogging also leaves the embankment more susceptible to overtopping, which would interrupt traffic flow and possibly breach the embankment. The increased difference in headwater and tailwater elevation produced would increase potential outlet velocities and have the potential to induce flow piping around the culvert or at weak spots along the embankment. Any one of these consequences could result in failure of the crossing system.

The condition cited above is documented in Case History 48 along Pond Creek at US-119 in Pike County, Kentucky. The bed of Pond Creek is aggrading and has aggraded approximately 0.5 to 1.0 m over the past 5 years. Aggradation is caused by the numerous coal strip mining operations in the area. Many culverts along the stream (including the one at US-119) have been partially filled with sediment, and the channel itself is losing flow capacity. Flooding in the area is frequent and continues to worsen.

As with aggradation, degradation profiles can approach a culvert crossing from either an upstream or downstream direction. Degradation can be produced by some trapping of the normal sediment load upstream of the crossing, such as at a dam. Degradation is characteristic of a reduction in base level downstream of the crossing. Conversely to the aggradation situation, degradation profiles affect the structural stability of the crossing rather than the capacity of the culvert.

Degradation downstream of the crossing reduces the channel bed level just downstream of the structure, which reduces the design tailwater level. This reduction increases the relative energy level of outlet flows and aggravates local scour problems at the outlet. Once again, considering the design performance curve presented in Figure 29, such a degradation condition will create a shift in the outlet control curve that in turn could potentially impact the other components of the curve. The additive effect of the long-term degradation combined with increased local scour could potentially undermine the structure and cause its failure. An example of the effect of this type problem is documented in Case History 58 (Figure 31).

Case History 58 describes conditions in Perry Creek at I-55 near Grenada, Mississippi. Perry Creek has been degrading for a number of years. In 1975, the apron of this culvert was beginning to break, and the degradation at the outlet was estimated at 1.2 to 1.5 m. During the flood of April 1977, the downstream apron continued to break up, and the left downstream wingwall broke away from the culvert barrel. The scour hole at the downstream end of the culvert was about 33 m in diameter and the low-water surface was about 2.5 m below the flow line of the culvert. Degradation in 1978 was about 8 m and is the result of several channel modifications on Perry Creek and Batupan Bogue.

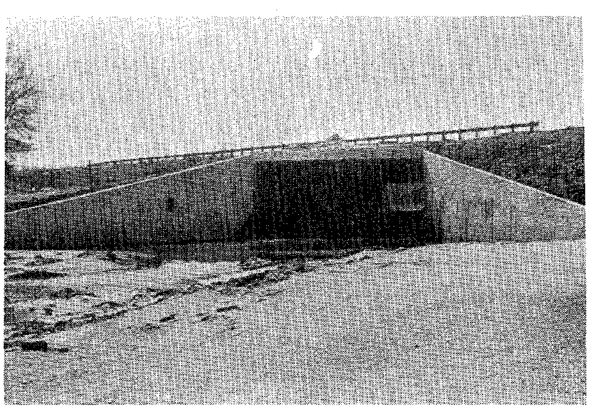


Figure 31. PERRY CREEK AT I-35 NEAR GRENADA, MISS.

General degradation trends upstream of a culvert usually produces aggradation problems, if any, at a culvert site. There cannot be degradation immediately upstream of a crossing because the culvert acts as a bed elevation control point; for water to pass through the embankment, it must go through the culvert at this elevation. The local aggradation problem induced is due to the inability of the culvert to pass the large sediment and debris loads that are a direct result of the upstream degradation. The increased debris load will also affect the charts used to design the culvert since one of the assumptions in the development of these charts is that the flow will be clear, i.e., it will have no sediment or debris loads. The impact of these type problems on the design of the culvert will be similar to that already discussed in relation to aggradation trends.

Inadequate Design and/or Construction

Inadequate design and/or construction of culvert systems can induce changes in channel regime. This change in regime is a result of the fact that the construction of culvert system produces a control point in the river environment. The culvert can be considered a control in two respects; first, the fixed culvert invert and alignment produces vertical and lateral control, and second, the design capacity of the culvert (reflected in the performance curve) produces some changed discharge condition just downstream of the structure.

Improper placement of the culvert invert creates several problems. If the invert is too low, the culvert tends to fill with sediment, reducing its available capacity as the channel attempts to increase its bed level locally. The reduced capacity increases the magnitude and frequency of ponding upstream of the crossing, compounding the aggradation problem as sediment drops out of the decelerating flow. Placing the culvert inlet too high will produce similar results only to a lesser degree. The increased invert elevation will produce a small pond at low flow and a corresponding area or volume of no effective flow at higher discharges. This area will be susceptible to aggradation. The reduced sediment load in the water discharge as a result of the upstream aggradation can produce a degradation problem downstream of the culvert outlet analogous to that produced by clear-water releases from reservoirs. Placing the invert too high will also increase the flow energy at the culvert outlet, producing additional degradation downstream of the structure.

A culvert designed with inadequate capacity can also influence the regime of a stream. Underdesign produces excess ponding upstream, inducing aggradation. The consequences of aggradation are as discussed above. Downstream of the culvert underdesign can induce a degradation profile as the relatively clear-water, high-energy discharges exiting the culvert attempt to increase their sediment loads.

Examples of the consequences of inadequate culvert design and/or construction are discussed in Case History 102 and 106.

SUMMARY

The effects of grade changes on highway crossing design considerations have been shown to be significant. This study shows that the parameters influenced most by grade changes are those used as inputs to the hydraulic design procedures and not the actual design equations and charts. These input parameters include design discharge, channel roughness, energy and bed slope, velocities, shear stresses, geometry, base level, and flow depth and alignment. Design practices were reviewed to determine more specifically where designs would be affected.

Problems encountered at bridge crossings include bridge capacity, backwater, pier and abutment alignment, footing depth at piers and abutments, and construction depth for flow-control and debris-control structures. With respect to bridge capacity and backwater, aggradation produces the most severe problems. However, debris problems associated with degradation can also have a significant impact. Foundation depths for piers, abutments, and flow-control structures can be influenced in two ways by grade changes: the normal streambed base level will be altered and the "normal" hydraulic conditions at a site used as input to local scour computations will be changed. Foundation depth problems were encountered in almost all the case histories reviewed for this study in which degradation problems were encountered. The important components of bank protection design adversely affected by grade changes are key depths and the vertical extent of bank protection.

Problems encountered at culvert crossings can be the result of general grade changes produced by long-term changes in stream morphology or inadequate design and/or construction of culvert systems. The design components most often influenced are culvert capacity and structural stability. The greatest danger produced by aggradation is partial plugging of the culvert opening resulting in a damming effect and increasing the magnitude and frequency of flooding upstream of the structure. Degrading stream reaches affect culvert systems by reducing their structural stability. General streambed degradation has been seen to undermine the foundations of culvert systems resulting in their complete failure.

CHAPTER IV

THE RIVER SYSTEM

GENERAL

The river system, composed of a watershed and channels and those entities that act on and react to them, is a prime example of a highly nonlinear, complex system. It consists of subsystems with overlapping components that can be categorized under nonexclusive headings as hydrologic factors, hydraulic factors, geologic factors (including materials and soils), climatic factors, biologic factors, and the influences of humans. Because of the overlapping and complex nature of the processes, the individual effects of river system components are often difficult to fully understand. However, a knowledge of the physical processes involved and typical system responses permits a better understanding of the river system.

River systems develop as a result of the interaction of the natural forces at work in the environment, the geologic structure of a region, and impacts imposed by development. In general, two types of forces are common to all the components of the fluvial system: those resulting from gravity and those resulting from inertia, friction, and cohesion. Gravity is involved in rainfall, infiltration, runoff, groundwater drainage, sediment movement, mass wasting, weathering, plant growth, and human activities. Conversely, inertia, friction, and/or cohesion sometimes counteract the actions of gravity by developing flow resistance, landslide stability, and erosion resistance. Because gravity and friction-cohesion forces affect many processes, the effects can be either positive or negative. For example, although gravity promotes downslope water movement resulting in soil erosion, it also prevents some soil particles from being eroded because of their weight.

The geologic structure within a region is principally a function of (a) the dynamic forces at work throughout time, (b) catastrophic events such as earthquakes, eruptions, fire, etc., and (c) the interaction of the initial geologic structure with the action of water, wind, and ice. This sculpturing produces the landforms with which we are familiar today. Some of these forms owe their origins purely to erosion processes, other forms may be depositional; and still others owe their existence to combinations of both processes.

Ideally, the basic principles underlying the development of landforms can be considered in simple terms. A given land area is composed of a particular set of rocks that have particular mineralogical compositions and specific physical properties. The geochemistry, structure, and environment of the rocks will determine their in situ weathering. Because these rocks were formed at different temperatures and pressures within the earth, when they are exposed at the surface, they are no longer in equilibrium with their environment and thus begin to decompose. Where a gradient is created by gravity, the moving water, earth, air, and ice help in the attack upon the rock and remove the products of weathering. In the process, landforms of various aspects are created. In a given environment, the physical and chemical constitution of the rocks determines the way in which they will break down and, in turn, the size and quantity of debris made available to erosion processes. The variations in environmental and sturctural characteristics from region to region are responsible for the individuality of river systems. They point to the need for an awareness of all factors involved in the development of a river system as an aid in understanding river response.

TIME SCALES

When analyzing the development of response of a river system, it is most important to understand the time frame under consideration. When considering the historic development of a river system, the geologist or geomorphologist would view the situation in terms of thousands of years. It would not be practical for an engineer to view time in this manner; the engineer considers a more immediate response or development period in terms of years; usually limited to 50 to 100 years.

Schumm (1971) and Shen (1971) provide a framework to gauge the time of a gradation change process. It is possible to think of a river over several thousand years when it should be slowly decreasing its gradient in response to the denudational lowering of a sediment source area that is not being uplifted.

Thus, during an extended time, a river is continually adjusting to the slow erosional modification of the entire drainage system; i.e., it is "degrading."

This longer period of time is commonly, if not incorrectly, referred to as geologic time. However, both the geologist and engineer recognize grades in stable rivers and canals that adjust during a short time to changes of discharges but that over a decade or a century are essentially unchanged. These rivers are open systems in dynamic equilibrium during what may be referred to as "graded time." Finally, over a very brief span of time, a channel may be an open system in a steady state as water and sediment move through it and its characteristics do not change. The consideration of longer time spans is wholly the concern of the geologist, whereas the events of graded time are of concern to both the geomorphologist and civil engineer.

The third time span that can be considered is steady state time, which is described in terms of hours and days, a period over which there is no significant gradation change. Steady state time is the concern of those interested in flood prediction and irrigation. For a full understanding of the variables influencing channel character, consideration of all three spans of time is necessary.

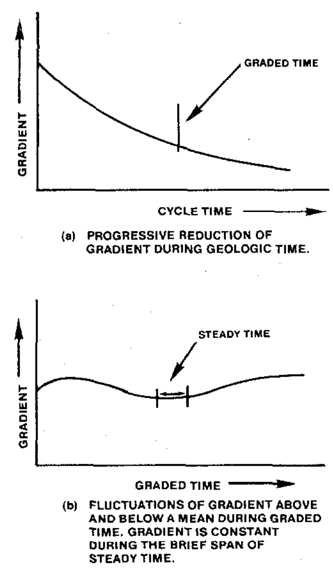
Table 8 illustrates this problem by listing, in a hierarchy of increasing degrees of dependence, variables influencing river morphology for the three different spans of time. The absolute length of these time spans is not important. Rather, the significant concept is that a drainage system may be considered in relation to time spans of different duration.

The three time periods being discussed are compared graphically in Figure 32. Figure 32(a) shows the gradual reduction in system gradient over cycle (geologist) time. The line marked "graded time" on the curve in Figure 32(a) indicates the relationship between geologic and graded time; when considering geologic time, graded time appears as an instantaneous occurrence. In Figure 32(b) graded time is expanded and the progressive changes of geologic time appear during graded time. During a "steady state time" span (defined as one week or less) [Figure 32(b)], a true steady state may exist in contrast to the dynamic equilibria of graded time. These brief periods of time are referred to as steady state time because in hydraulics, steady flow occurs when none

Table 8. RIVER VARIABLES DURING TIME SPAN

VARIABLES	STATUS OF VARIABLES DURING DESIGNATED TIME SPANS: GEOLOGIC (>1000 yr)	GRADED (days to 1000 yr)	STEADY (sec to day)
Time	I	N R	N R
Initial Relief	I	N R	N R
Geology	I	I	I
Paleoclimate	I	I	I
Paleohydrology	D	I	I
Relief	D	I	I
Valley Dimensions	D	I	I
Climate	X	I	I
Vegetation	X	I	I
Hydrology	X	I	I
Channel Morphology	X	D	I
Observed Qw Qs	X	X	D
Hydraulic of Flow	X	X	D

Key I = Independent; D = Dependent; N R = Not relevant; X = Indeterminate



Source: Schumm and Lichty (1965)

Figure 32. TIME SPANS FROM TABLE 8 (CHANNEL GRADIENT IS USED AS THE DEPENDENT VARIABLE IN THESE EXAMPLES)

of the variables involved at a section changes with time.

During graded and steady state time, channel morphology reflects a complex series of independent variables, but the discharge of water and sediment integrates most of the other independent variables. The nature and quantity of the sediment and water that moves through a channel during those times is primarily responsible for the morphology of stable alluvial channels. It is in this period of graded and steady state time that the research presented in this study falls.

RIVER SYSTEM RESPONSE

A knowledge of the trends or tendencies apparent in the development of natural systems is important to the understanding of river system response. (A natural system is defined as a system insulated from human activities.) Several of these tendencies specifically related to meander development are discussed by Inglis (1949).

Inglis notes that the factors controlling flow in open channels include discharge, sediment load, slope, and water temperature. Although these components are dynamic in nature, there is a year-to-year dominant condition of discharge, sediment load, slope, and temperature at which equilibrium is closely approached.

Inglis also stated that small variations from the dominant condition are adjusted by bed scour when the sediment load is in deficit. The finer grades are then washed out of the bed mix so that coarser grades are exposed on the bed; when the charge is excessive, deposition takes place, and the bed-mix becomes finer.

Inglis states:

> "In a channel in which the discharge varies, the channel obviously cannot alter its slope, width, and depth whenever the discharge changes; but it will tend to do so; and each channel is the integrated effect of all the various discharges which have flowed. The banks of the channels being more resistant to scour than the bed, the biggest changes take place in the bed and it is a well-established fact that after low floods, the beds of rivers rise, to be scoured out again in the next high flood. This scour is not due solely to side resistance (flume flow); but even more to the fact that silt charge which can be carried, increases directly as the effective velocity; and hence, as the discharge increases, the silt per cubic foot of water also increases.
>
> "In channels with widely fluctuating discharges and silt charges, there is a tendency for silt to deposit at one bank and for the river to move to the other bank. This is the origin of meandering.
>
> "While the discharge of a river is increasing, there is a tendency for the meanders to increase in length and hence to erode the concave bends at their downstream ends. This cutting is highly complex in character; but there is a tendency for a river bend to move downstream, though this is often masked by cut-offs, resistant strata, etc.
>
> "Bed movement is essentially a progression, resulting from continuous 'trading' throughout the river. This 'trading' is not continuous, the river bed tending to build up in some parts and scour in others; but, over a considerable period of time, movement is general, the finer particles moving more quickly and in greater quantity than the coarser particles; so that there is a progressive diminution in size of particles down a river. Despite this change of grade, the weighted mean charge is determined throughout the river by the material entering the river; and bank erosion and meandering are also controlled by this charge and the discharge available to carry it.
>
> "It might be thought that a stage would be reached when a year to year stability would occur. To a certain extent this is so – when a reach of river attains its maximum length, the average slope then being a minimum. Conditions may then remain relatively steady for some years; but no final stability is reached – partly because the discharge and charge vary during each year, and from year to year; but chiefly because changes occur in the river upstream and downstream – which upset the temporary balance."

The point that must be stressed is that a river, through time, is dynamic, that man-induced and nature-induced changes frequently set in motion responses that can be propagated for long distances, and that, in spite of the river's complexity, all rivers are governed by the same basic forces. It is absolutely necessary for the design engineer to have at hand competent knowledge of the hydrologic, hydraulic, geologic, climatic, and biologic factors that influence the system's response. It is also important to be aware of the influences of humans in the system and the possible impacts that they could produce.

To emphasize the inherent dynamic qualities of river channels, evidence is cited below to demonstrate that most alluvial rivers are not static in their natural state. In fact, scientists concerned with landforms (geomorphologists), vegatation (botanists), and the past activities of man (archaeologists), do not consider the landscape as unchanging. Rivers, glacial terrain, aeolian deposits, and seacoasts are highly susceptible to change with time. Over a relatively short period of time, perhaps in some cases as long as a person's lifetime, components of the landscape may be relatively stable. Nevertheless stability cannot be automatically assumed. Rivers have changed their locations and forms in as short a time period as a few years.

Response to Changes Induced by Humans

Rivers change position and morphology (dimension, shape, pattern) as a result of many natural or human-induced factors. Some of these factors were discussed as they related to specific case histories in Chapter II. The most common human activities that induce river response are channel alterations, streambed mining, damming and reservoir regulation, land-use changes, and construction activities.

Channel Alterations

Channel alterations include dredging, channelization/straightening/cutoffs, clearing/snagging, constrictions, and structure alignment (as documented in Chapter II). Although each of these changes produces somewhat unique results, the overall effect produces degradation in the river system.

Dredging, channelization, straightening, and the construction of cutoffs effectively shorten the flow-path of the river and subsequently increase its slope. This change produces an imbalance in the water-sediment complex in the system. The natural system response is to attempt to adjust itself to its original slope either by increasing its length or reducing its bed elevation or by some combination of these throughout the affected reach. The response path taken by the river system depends on the relative stability of the bed material and banks. The most frequently reported response is for the flow to degrade the bed to some point and then start eroding the bank, producing some new meander pattern. The initial degradation increases the apparent bank angle, reducing the stability of the bank. Subsequently, the bank caves and bank vegetation is lost. That vegetation, in many cases, is the primary stabilizing factor along the channel and when it is lost, the bank stability is reduced further.

Clearing and snagging operations are generally undertaken as a part of watershed improvement schemes to improve flow capacity and reduce potential flood hazards as documented in Case History 87. However, these activities produce other results as well. Clearing and snagging operations reduce the resistance to flow through a river reach and increase the channel velocities and thus the potential kinetic energy of water discharges. The increased flow energy will be dissipated through increased sediment transport rates within the river system. As occurs with channel straightening, the response initially involves degradation and then lateral erosion. The system is responding in an attempt to reduce its energy slope to a level that will once again return the system to a condition of semiequilibrium.

Constrictions and flow alignment problems are generally associated with river control projects and river crossing design. Flow alignment problems are also associated with those channel straightening projects in which the transition from old channel to the new one is too abrupt. In the case of constrictions, the increased potential energy created by the damming effect just upstream of the constricted reach increases the amount of energy that must be dissipated through the constricted reach as the potential energy is transformed to kinetic flow energy. The increased flow energy increases the sediment transport rates through the reach and produces a local degradation problem. Within the constricted or laterally controlled stretch of river, flow alignment can generally be assumed to be fixed, eliminating the potential for lateral instability.

Flow alignment problems in transition zones downstream of channel alterations generally produce lateral stability problems. Improper flow alignment directs the flow towards the natural bank destroying the natural vegetation and resulting in progressive undercutting and bank failure. In the extreme, the channel would cut through the bank and create a new channel just downstream of the transition.

Streambed Mining

Another common human activity that upsets the natural balance in a river environment is streambed mining. Sand and gravel mining activities affect the sediment movement and supply in a channel system. Such operations can be beneficial or detrimental depending on watershed and river characteristics and the management practices followed.

Under proper management, removal of sand and gravel can increase the stability of a river system that is overloaded with sediment (supply greater than transport capacity). The overloaded condition can exist as a result of the natural characteristics of the watershed or from abnormal events. These events could include land conversion changes in the watershed, construction, seismic activities, etc. The overload of sands and gravels can form large gravel bars and also provide material to form an armored layer of coarse particles on the streambed. This activity encourages lateral migration from the shifting of the thalweg in response to the development and movement of the bars and the relatively erodible bank material. With this condition, controlled removal of gravel bars by extraction and limited mining may actually enhance channel bank stability. Hence, proper river management is required to maintain equilibrium between excess production of sand and gravel and extraction of sand and gravel.

Excessive sand and gravel removal (removal greater than supply in any given reach) can endanger the stability of the river system and bridges by inducing degradation and headcutting. The extent of damage to the system that can result from the headcut is a function of the volume and depth of the gravel pit, location of the pit, bed-material size, flood hydrographs, and sediment inflow rates and volume. The presence of a gravel pit can add energy to the system by increasing the water surface slope (or energy slope) just upstream of the pit. The steeper slope has greater erosive power and can initiate bank erosion and headcutting. These processes supply additional sediment to the river in quantities greater than it is capable of carrying locally and can induce aggradation downstream of the pit.

Damming and Reservoir Regulation

The development of reservoirs for storage and flood control produces a system response both upstream and downstream of the structure. The reservoirs serve as traps for the sediment normally flowing through the system, producing a clear water release downstream of the dam.

When the stream flowing into a reservoir encounters the ponded water, its sediment load is deposited and a delta is formed. This deposition in the reservoir flattens the gradient of the channel upstream. The flattening of the upstream channel induces aggradation causing the bed of the river to rise threatening highway installations and other facilities. The potential impacts of this deposition and delta formation are evidenced upstream of Elephant Butte Reservoir on the Rio Grande River. The aggradation profile at that site extends many miles upstream of the reservoir. At Albuquerque, New Mexico, the riverbed has aggraded until it is currently several meters above the level of the city. Ultimately the river may be subjected to a flow whose magnitude is sufficient to overflow existing banks and cause the water to seek an entirely new channel.

The construction of a dam influences downstream channel stability in two ways. Besides trapping the sediment load, it changes the downstream natural flow characteristics. Both the sediment load and flow conditions were responsible for establishing the natural regime of the channel prior to construction of the dam. Clearwater released from a reservoir immediately picks up a new sediment load downstream if the discharge is sufficient to erode the bed and transport the sediment. When this condition prevails, the net result is the erosion of the channel and lowering of the bed. This process will continue until the stream is again in balance with the new flow characteristics.

Land-Use Changes

Humans have made many land-use changes that have disturbed the natural conditions under which river systems develop. These changes include agricultural activities, urbanization, and commerical development of land resources. Of these, the widespread use of land and associated practices for agriculture is the most significant.

The opening of new lands for agriculture necessarily disturbs natural conditions. Indigenous forests are removed, and native grasslands are burned, overgrazed, or turned over with plows. Except under extremely rare conditions, the removal of the protective vegetation and the loosening of the soil during cultivation results in a speeding up of the erosion process. The streams and rivers draining these new agricultural areas are suddently overloaded with sediment, producing an imbalance in the equilibrium between sediment and water discharges. If the overloading persists, the drainage system will be built up and its slope steepened to increase the system's ability to transport the increased load.

The magnitude of the change in erosion rates can be significant. Plot studies in the United States indicate that the removal of forest cover and conversion of land to intertilled crops can accelerate erosion between 100 and 1000 fold [Musgrave (1957) and Gottschalk (1958)]. The conversion of grassland to cultivated crops may also increase erosion between 10 and 50 fold. Although no detailed inventory of the total soil loss in the United States has been compiled, the erosion from croplands alone is estimated at about 4 billion tons a year.

Another important land-use change that affects river systems is urbanization. Urbanized areas, when fully developed are actually low sediment-producing areas because a large percentage of the land is protected against erosion by roofs, streets, parking lots or well-cared for lawns and parks, and curbs, gutters, and storm sewers. The most significant long-term urbanization effect on river morphology and system development is the resultant increase in peak discharges. The increase in discharge comes from the reduction in overland flow resistance produced by the greatly increased impervious land-area. The reduced resistance increases overland flow velocities, reducing peaking times and increasing peak discharges. The imperviousness also reduces infiltration losses and allows more of the rainfall from a given event to run off as overland flow.

The response of the river system to these changes is to degrade and reshape those channels immediately downstream of the urbanized area. The degree of change depends primarily on the relative sizes of the urbanized area and the tributary streams. The more specific cause of the degradation will be the increased flow velocities (kinetic energy) and the reduced sediment load in the runoff. The larger peak discharges reshape the channel systems making them generally wider and more sinuous.

Although the response of the small streams draining the urban area is to degrade, the response in the main stem of the drainage system varies depending on the relative magntiude of the total runoff from the entire system and the runoff produced from the urban area. If the urban area is the primary contributor of runoff for the system, the main steam responds similarly to its tributaries; however, if the urban area only produces a low percentage of the runoff from the entire river system, the result would be opposite. The increased sediment load produced as a result of tributary degradation could produce an overload condition on the main stem. This condition would create an aggradation problem on the main stem as the river attempted to steepen its slope to increase its sediment carrying capacity. It is important to note the system response changes as one moves to lower-order streams.

Construction Activities

The response to urbanization cited above is the permanent response after an urban area has been developed. During actual construction of the urban area, however, human activities produce extremely high erosion rates, which have been known to drastically overload stream systems. These large sediment loads result from the removal of trees and other vegetation and excavation and grading activities that accompany all construction activities. The construction of large blocks of housing developments and shopping centers exposes large tracts of land to serious erosion for up to 2 to 3 years before complete stabilization. Road and highway construction can also cause local problems or downstream sediment problems. Large areas, including steeply sloping cuts and fills, may be exposed to erosive conditions for long periods of time, increasing sediment loads that reach streams.

The response of the river system to these activites would be to aggrade. The response would be similar to those discussed previously for other overloading problems.

Response to Natural Changes

The various river system responses to nature-induced changes are similar to those of man-induced changes. Essentially, the only difference is the process initiating response. Natural changes are induced primarily by natural channel alterations, earthquakes and tectonic activities, and climatic conditions.

Natural Channel Alterations

The most common natural channel alterations are cutoffs and chute development, which are associated with natural channel straightening. The general consequence of cutoffs is to shorten the flow path and steepen the gradient of the channel. The local steepening can significantly increase the velocities and sediment transport. This action can also induce significant instability such as bank erosion and degradation in the local area of the cutoff as well as upstream of it. The degradation, in fact, proceeds upstream until it reaches some natural or man-made base level control. This degradation lowers the base level of tributary streams, causing them to degrade too. The material scoured in the reach affected by the cutoff is carried to the next downstream reach of river and deposited. The result is an increase of base level and flood stage in the downstream reaches of the river and subsequent increases of base level for tributary streams causing aggradation.

Earthquake and Tectonic Activity

Among the natural phenomena that can affect the river environment are earthquakes and tectonic activity. Large portions of the United States are subjected to at least infrequent earthquakes. Associated with earthquake activity are severe land slides, mud flows, uplifts and lateral shifts in terrain, and liquefaction of otherwise semistable materials, all of which can have a profound effect upon river systems.

Drainage patterns that have been disrupted by uplift or some complex warping of the Earth's surface have been noted by geologists. In fact, complete reversals of drainage lines have been documented. In addition, convexities in the longitudinal profiles of both rivers and river terraces (such profiles are concave under normal development) have been detected and attributed to upwarping. Further, the progressive shifting of a river towards one side of its valley has resulted from lateral tilting. Lateral shifts on the order of 320 km have been reported for the Brahmaputra River in Bangladesh and India due to tectonic movements. These shifts occurred approximately 200 years ago [Cole (1969)].

Tectonic and natural geologic activities produce various responses from river systems. These responses include channel changes, local reach and system scour or deposition, headcutting, bank instability, landslides, rockslides, mudflows, and slide lakes. More specifically, upwarding, tilting, and uplifts will produce severe degradation problems upstream and aggradation problems downstream. The degradation profile could move as a head cut. Landslides, rockslides, and mudflows will produce extensive aggradation problems and the development of numerous slide lakes. Lateral shifting will produce very severe local lateral erosion problems from the misalignment of flow that has the potential to reform the entire channel downstream increasing its sinousity and drastically altering its pattern.

Neotechnics should not be ignored as a possible cause of local river instability. The highway engineer must be aware of local geological conditions within the river system being studied for two reasons. First, recognizing and understanding the consequences of past geologic activities makes it possible to anticipate the system responses to these activities. Second, although it may not be possible to design for this type of future natural disaster, knowledge of the probability of its occurrence is important.

Climatic Changes

Rivers change position and morphology (dimensions, shape, pattern) as a result of changes of hydrology. Hydrology can change as a result of climatic change over long periods of time, as a result of natural stochastic climatic fluctuations (droughts, floods), or by man's modification of the hydrologic regime. Long-term climatic change, while interesting from an academic point of view, does not have an impact on modern river stability. Of interest here are impacts produced as a result of natural stochastic climatic fluctuations such as droughts or floods.

To understand how climatic fluctuations influence channel stability it is necessary to look at channel morphology. Channel dimensions, shape, and pattern have been studied by many investigators* and the following proportionalities have been found to exist in nature:

$$b \sim Q^n$$
$$d \sim Q^m$$
$$s \sim (1/Q)^p$$
$$\lambda \sim Q$$

where

b	=	water surface width at characteristic discharge,
d	=	water depth at characteristic discharge,
Q	=	characteristic discharge,
s	=	slope of energy line,
λ	=	meander wavelength, and
n,m,p	=	variable exponents.

The independent variable in each of these relationships is the discharge; since discharge is directly proportional to rainfall, any change in climatic factors will influence the runoff. From the above relationships, it can be seen that any change in the characteristic discharge of a system will influence the dimension, shape, and patten of the channel. More specifically, during periods of repeated flooding (several successive "wet" years), the width, depth, and meander wavelength of a channel will attempt to increase while the channel bed degrades to reduce its slope. During successive "dry" years (drought periods), the width, depth, and meander wavelength will respond by decreasing in magnitude while the bed aggrades to increase its slope.

Summary

River systems can exist in a very precarious equilibrium state. When a river system is near a threshold of change, any alteration to the river environment can bring about numerous subsequent system responses. The factors causing instability can be man-induced or nature-induced: however, the primary source of channel instability is human activities [Keefer, McQuivey, and Simons (1980)]. Table 9 gives a listing of man-induced and natural causes of gradation problems and the typical response as documented by the case histories. Note that some activities cause either an aggradation or degradation response. It is important for the highway engineer working in the river environment to understand and anticipate the various sytem responses to the natural and unnatural activities influencing a particular crossing site.

*Lane (1955), Leopold and Maddock (1953), Leopold and Wolman (1957), and Schumm (1971) to name a few.

Table 9. RIVER SYSTEM RESPONSE TO CHANGE

PROBLEM CAUSE	RESPONSE	
	AGGRADATION	DEGRADATION
Human Activities		
Channelization/Straightening		X
Clearing/Snagging	X	X
Streambed Mining		X
Daming and Reservoir Regulation		
Upstream	X	
Downstream		X
Land Use Change		
Urbanization and Community Development		X
Agriculture	X	
Construction Activities	X	
Natural Processes		
Cutoffs		X
Alluvial Fan Development	X	
Tectonic Activity	X	X
Climatic Changes	X	X

CHAPTER V

GEOMORPHIC AND ENGINEERING CONCEPTS AND RELATIONSHIPS

The analysis of aggradation and degradation problems requires a broad knowledge of geomorphic and engineering concepts and relationships of alluvial boundary open channel flow and sediment transport. Several enigneering texts, reports, and documents contain much of the required information [ASCE (1975), Chow (1959), Graf (1971), Henderson (1966), Richardson et al. (1974), Simons, et al. (1975), Simons et al. (1980), Simons and Senturk (1977)]. Primary sources of information are *Sedimentation Engineering* [ASCE (1975)], *Highways in the River Environment* [Richardson et al.(1974)] and *Sediment Transport Technology* [Simons and Senturk, (1977).] A review of these documents is recommended for the engineer unfamiliar with the complexities of alluvial boundary flow.

This chapter reviews the required background information for analyzing aggradation and degradation, referencing the details when appropriate. First, a discussion of river and watershed classification is presented; then, channel control identification, open channel flow concepts, applicable geomorphic relationships, sediment transport relationships, incipient motion considerations, and upland area sediment yield considerations are discussed.

RIVER AND WATERSHED CLASSIFICATION

River and watershed classification discussed here is based on stream properties that are observable in aerial photographis and/or in the field. The major function of the classification schemes is twofold. First, it provides an opportunity for the engineer to become acquainted with the characteristics of the river system and watershed he is working with. Second, it provides clues that can aid in the determination of channel stability. Channel stability, as well as several other aspects of stream behavior, is reflected in the physical appearance of the stream and its channel. Since the best guide to the future behavior of a stream during the life span of an engineering structure is its behavior during the immediate past, the information gained from river and watershed classification can be valuable to a stability analysis.

General

Several approaches are possible for delineating and analyzing the physical processes that govern river systems. As a first step, watersheds and rivers can be classified into different groups based on certain physical characteristics. These characteristics are usually a direct result of the physical processes that govern the system and, therefore, provide an initial view of the controlling phenomena. Once subdivided, watershed and rivers can be described in terms of the processes that control their response. After these processes are understood as well as possible, their relative importance can be evaluated.

Watershed Classification

Many factors combine to characterize a watershed. At present, there does not seem to be any widely accepted or employed watershed classification system. However, as the population continues to grow and interest continues to increase in environmental issues, more emphasis is being placed on water, mineral resources, timber, energy, recreation, and residential uses of upland watersheds. Therefore, a better understanding of the processes and responses of various watershed activities is required. Such an understanding can be aided through the implementation of a watershed classification scheme.

Research has identified various methods for quantifying watersheds according to morphological measures. Horton (1945) was one of the first to recognize and utilize the relationship between stream order and number and length of streams in that sequence. In general, drainage area and stream length increase as the stream order increases. However, as stream order increases, the number of streams in that order decreases, which qualitatively indicates that a large stream has a large drainage basin and is longer than a small stream. However, the "Horton analysis" of a drainage basin does not provide a quantitative basis for these qualitative observations. Langbein et al. (1964) and Strahler (1952, 1957) developed methods for determining additonal watershed indices. Many of the most widely used morphological characteristics and their related measurement techniques have been summarized by Chow (1964). Some of the widely used characteristics are watershed area, stream length, average main stream slope, drainage density (miles of stream per square mile of watershed area), measures of watershed shape such as circularity ratio (ratio of the basin area to the area of a circle with the same perimeter as the basin), and watershed slope (such as average overland slope or relief ratio, which is the total watershed relief divided by the distance from the outlet to the furthest point on the basin divide).

Little more in the way of watershed classification has been utilized beyond morphological measures. However, a watershed cannot be completely described by those measures alone. Indices of soils, geology, vegetation, hydrology, and hydraulics are also needed to describe the watershed more accurately. Pfankuch (1975) and Rosgen (1975) have presented methods for delineating important watershed channel characeristics that can lead to numerical and descriptive classifications of the channels. This is a positive step that should be extended to other parts of the watershed system. For example, a rating system or classification of sediment sources and yields from watersheds would be helpful in describing this important aspect. Simons et al. 1979) have specified important sources of sediment in watersheds.

It is possible, using experience gained from years of field inspections and modeling efforts, to develop a framework of watershed classification. This proposed framework, Table 10 is based on the primary needs of modeling and quantifying descriptions as determined from past experience.

As Table 10 indicates, many different combinations of components are possible for watershed classification. However, such a system would provide a common basis for comparison of different water-

Table 10. PROPOSED FRAMEWORKS FOR CLASSIFICATION OF WATERSHEDS

KEY COMPONENT	POSSIBLE DESCRIPTORS
Location	State, county, town, or township
Geometry	Area, slope, drainage density, length of streams, channel characteristics
Soil	Type, distribution, dominant size, erosivity, depth, texture
Geology	Bedrock and surficial types and distribution, structure, effects on hydrology and soils
Vegetation	Overstory and understory types and distribution, interception, transpiration, stage of succession
Climate	Precipitation type, seasonal occurrence, duration, frequency, temperature, evaporation
Hydrology	Surface water, ground water, infiltration interflow, peak discharges, yearly hydrographs, ephemeral or perennial discharge
Sediment Yield	Sources, erosion and transport mechanisms, sediment hydrograph characteristics
Human Influence	Degree of development, type of development or construction activity, impacts on other components

sheds. For example, a watershed may be classified as listed in Table 11. Although Table 11 gives general descriptions, it serves to show the many pieces of information needed to fully describe and classify a watershed.

River Classification

In contrast to the lack of a widely used watershed classification system, rivers have many classification schemes. Davis (1899) first suggested that

Table 11. EXAMPLE OF A WATERSHED CLASSIFICATION USING COMPONENT SYSTEM PROPOSED IN TABLE 9

KEY COMPONENT	SELECTED DESCRIPTORS
Location	Any state, any county, any town or township
Geometry	Area = 1.6 sq km Average channel slope = 0.05 Average overland slope = 0.25 Narrow, deep channel; width/depth = 3
Soil	Gravelly sand loam predominates, sand sizes predominate Moderately erodible (gravel offers protection) Typically 50-75 cm deep
Geology	Granitic rocks — primarily granodiorite. Some basalt flows. Major fault in watershed and basalt-granodiorite contact provides zones of groundwater discharge
Vegetation	Ponderosa pine (second growth) overstory Minimal understory from recent fire Sixty percent canopy cover Forty percent ground cover
Climate	Winter snowfall = 70% of yearly total Spring rain showers = 15% of yearly total Summer thunderstorms = 15% of yearly total Yearly precipitation average= 63.5 cm Average January minimum temperature = 10° F Average July maximum temperature = 83° F
Hydrology	Mixed ground and surface water flow Geologic controls force most of groundwater to surface at basin mouth Ephemeral stream Storm flow controlled by interflow
Sediment Yield	Primarily surface and rill erosion A few small landslides, little channel erosion Bedload transport of sand size particles
Man's Influence	Increased erosion near an unprotected roadway Some soil disturbance on logged areas

rivers could be divided into three stages: youth, maturity, and old age. Thornbury (1969) presented common valley classifications based on their development on the surface of the land. Schumm (1963, 1971) quantified stream types by using discharge and type of sediment load. Rivers can also be classified broadly in terms of channel pattern. Simons et al. (1975) discusses how rivers or segments of rivers, can be generally classified as straight, meandering, braided, or some combination of these. Culbertson, et al. (1967) proposed a classification scheme based on vegetation pattern, sinuosity, and bank characteristics. Other more complex classification schemes are available, many of which have been compiled by Rundquist (1975).

The river classification scheme recommended here is a modification of that presented in Brice, Blodgett, et al. (1978). The modified version, shown in Figure 33 is a detailed classification scheme oriented to reflect aggradation and degradation potentials in rivers. It provides an excellent quick-reference guide to the types of alluvial channels. The common geomorphic terms for the various types of streams (meandering, braided, incised, etc.) are shown in the figure. Each term is defined by the small sketches.

Summary

Watershed and river channel classification systems are a reflection of the physical processes that govern their observable characteristics. As an example, consider the nature and stability of straight, braided, and meandering channels as discussed in Report No. FHWA/RD-80/038. Each behaves in a slightly different way when subject to man-related or natural impacts. A knowledge of this behavior is important in anticipating and understanding gradation problems. However, no matter what classification scheme is used, it should be remembered that the classification only applies to that segment of the system being analyzed at the time. Because river channels are dynamic, the river form can change significantly from location to location and thus change its classification.

IDENTIFICATION OF CHANNEL CONTROLS

The identification of natural or man-made control points in rivers is essential to aggradation/degradation analysis. Control points form the upstream and downstream limits for the analysis; they also define the reach length over which the analysis must be carried out. Channel controls are well defined, permanent sections having some form of base level and width control where the depth/discharge and discharge/sediment load relationships are well established.

Channel controls can be both natural and man-made. Natural controls include bed-rock outcrops, heavily armored channel conditions, clay plugs, buried vegetated material, fixed or controlled water surface levels, and in some cases tributary junctions. Heavily vegetated bank lines also provide lateral control. Most hydraulic structures built on rivers represent man-made controls. Typical examples include dams, weirs, spillways, free overfalls, underflow gates, sluce gates, culvert crossings, and some bridge crossings. Submerged pipe lines can also act as controls.

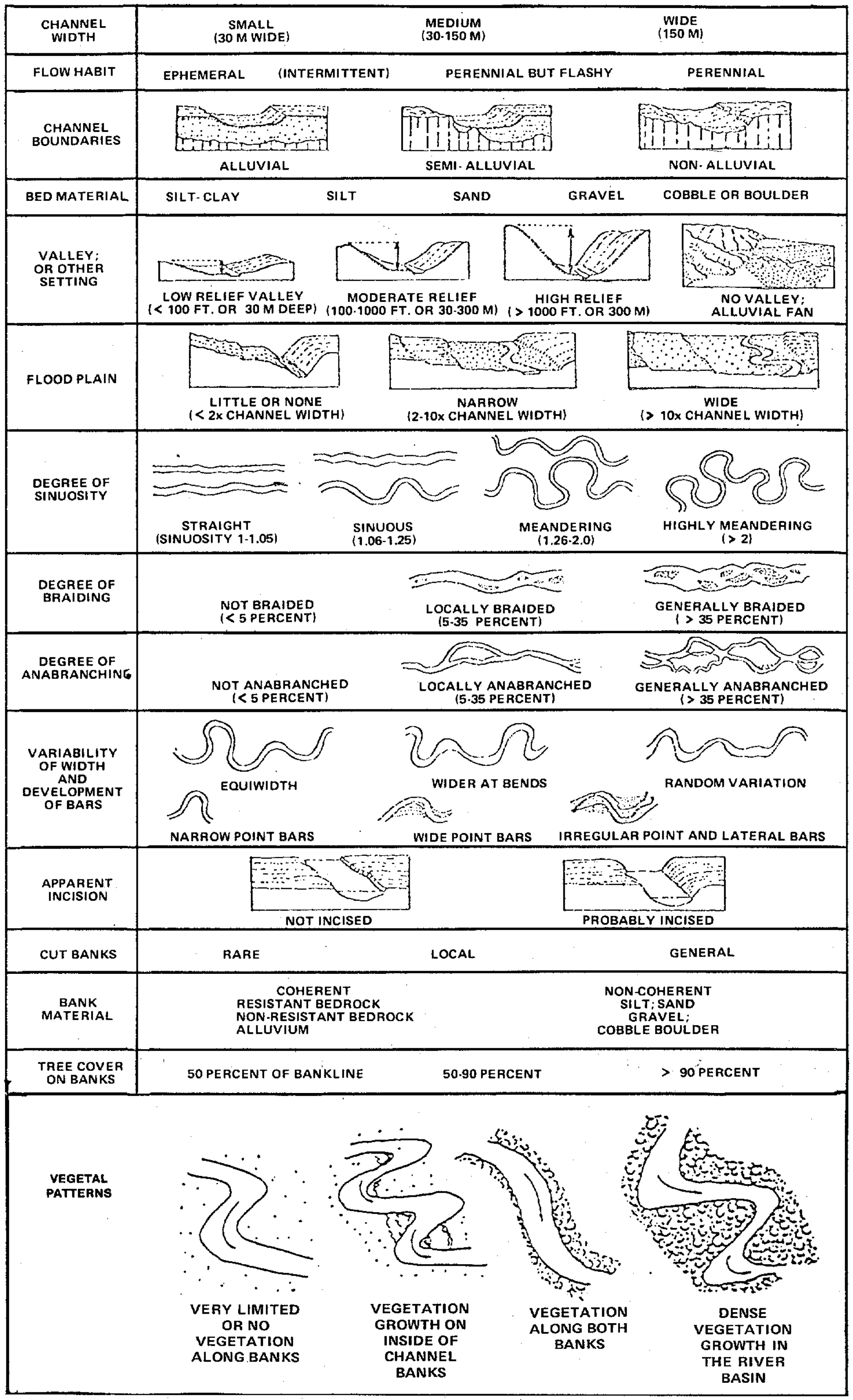
The most efficient means of locating channel controls is through field reconnaissance and the study of low-level aerial photography. A common channel control evident from aerial photographs is rock ledges; these rock ledges extend across channel beds and often outcrop at the bank.

OPEN-CHANNEL FLOW CONCEPTS

Although a basic understanding of open-channel flow is assumed, it will be helpful to review some general information required for the correct application of various gradation change prediction techniques. This information includes flow classification, bed configurations, flow regimes, flow resistance, water surface profile computations, and flow in bends.

Classification of Flow

Establishing the state of flow is essential to the evaluation of the hydraulic factors within aggrading or degrading reaches. In open-channel flow, gravity forces predominate and three basic flow conditions – subcritical (tranquil), supercritical (rapid), and critical flow – are possible. The state of flow in a given reach influences the depth and velocity within



Source: Modified from Brice, Blodgett, et al. (1978)

Figure 33. STREAM PROPERTIES FOR CLASSIFICATION STABILITY ANALYSIS

the flow field. Bed shear stress, which is one of the most important parameters for defining sediment transport characteristics, is directly dependent on flow depth and velocity.

The state of flow also determines the direction of the computational procedure. For subcritical flow, backwater and sediment routing is done from the downstream control to an upstream control. However, supercritical flow requires that computations start at the upstream control and proceed to the downstream control. Detailed descriptions of flow type and their influences on highway crossing design are included in Richardson et al. (1974).

Resistance to Flow

To evaluate the hydraulics of river channels, it is essential to determine the resistance of flow, which is a function of many variables involving channel geometry, river stage, bed configurations, size and characteristics of bed and bank material, supply of sediment, etc. The purpose of flow resistance investigations is to relate the flow velocity to all other factors that might affect that velocity. The two primary forms of resistance are due to bedforms and grain size.

The basic problem in flow resistance work is the determination of the resistance coefficient. Two approaches can be used, but neither is completely satisfactory.

The first approach is to assign values based on experience. Aids to this method are tables of values that have been found to be common among open channels and sets of photographs of various channels allowing visual comparisons. Potential errors of this approach are large when relied on as sole source; subjective judgement leads to the assumption that the chosen coefficient applies to all discharges despite the large changes in flow resistance that actually occur as the discharge varies.

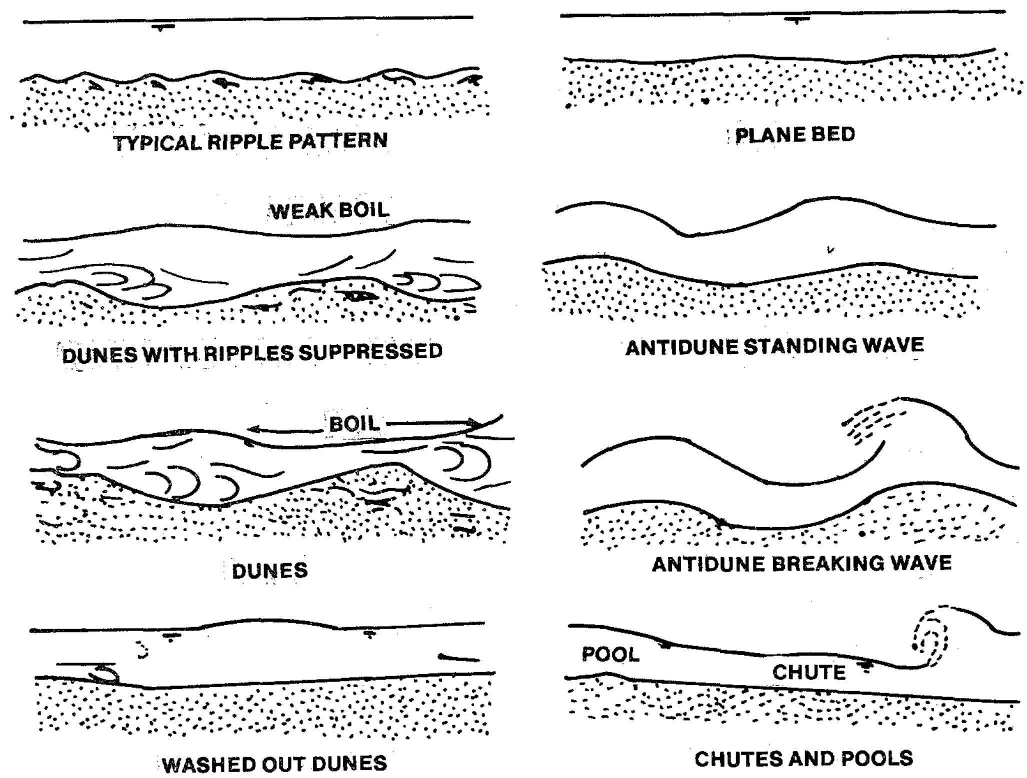
In the second approach, the coefficient is calculated by some equation based on a theoretical description of the relevant process of flow. The potential for accuracy is greater, but the method is inevitably more complex. At the present time, only simple flows have been described in this fashion.

It is recommended here that a combination of both approaches be used. Theoretical computations can be verified with resistance values based on experience.

Bed Forms

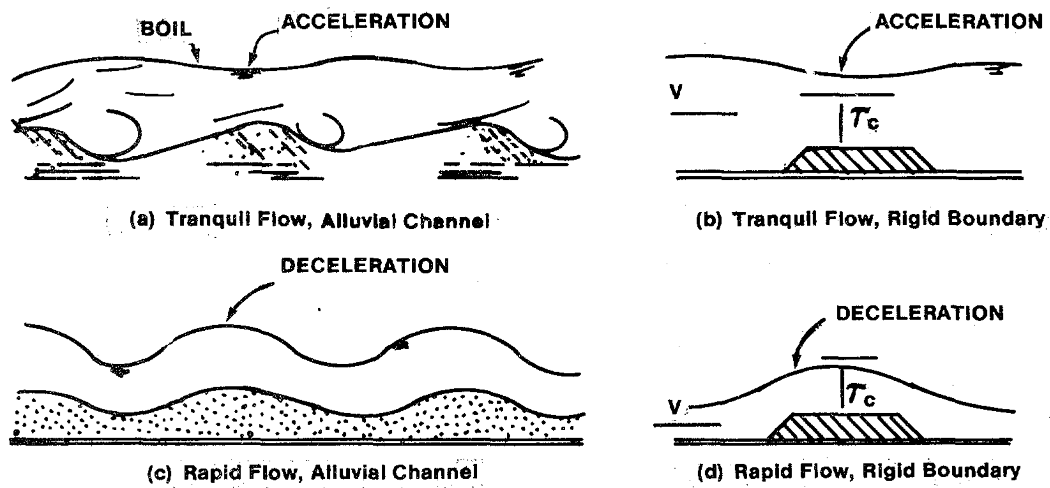
For flow in channels composed of erodible granular material, a strong physical interrelationship is known to exist among the friction factor, the sediment transport rate, and the geometric configuration assumed by the bed surface. The changes in bedforms result from the interaction of the flow, fluid, and bed material. Thus, the resistance to flow and sediment transport are functions of the slope and depth of the stream, the viscosity of the fluid, and the size distribution of the bed material. The interaction between the flow and bed material and interdependency among the variables makes the analysis of flow in alluvial streams extremely complex. However, with a general understanding of the different types of bedforms that may occur and a knowledge of the resistance and sediment transport associated with each bedform, the engineer can begin to analyze and understand alluvial channel flow.

The bed configurations (roughness elements) that may form in an alluvial channel are plane bed without sediment movement, ripples, ripples on dunes, dunes, plane bed with sediment movement, antidunes, and chutes and pools. Sand bed alluvial streams can experience all the bed configurations listed. However, gravel bed streams are known to only experience plane bed without bed movement, dunes, plane bed with bed movement, antidunes, and, in very steep streams, chutes and pools. The typical forms of each bed configuration are shown in Figure 34, and the relationship of bedform to water surface is shown in Figure 35.



Source: Richardson et al. (1974)

Figure 34. FORMS OF BED ROUGHNESS IN SAND CHANNELS



Source: Richardson et al. (1974)

Figure 35. RELATIONSHIP BETWEEN WATER SURFACE AND BED CONFIGURATION

The different forms of bed roughness are not mutually exclusive in time and space in a stream. Bed roughness elements may form side by side in a cross section or reach of a natural stream, giving a multiple roughness, or they may form in time sequence, producing variable roughness.

Changes in bedform are related to stream power (V γ d S; where V is the flow velocity; γ is the specific weight of fluid; d is the flow depth; and S is the energy slope often assumed to equal the channel bed slope). Figure 36 shows a relationship between stream power, median fall diameter, and bed configuration.

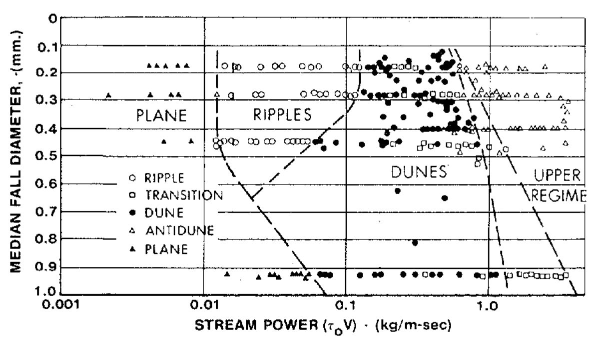


Figure 36. RELATIONSHIP AMONG STREAM POWER, MEDIAN FALL DIAMETER, AND BED CONFIGURATION

Since Manning's roughness coefficient is the most commonly used measure of flow resistance in open channels, the resistance to flow reflected by the various bedforms is related to this parameter. Table 12 lists the various bed configurations along with representative values of Manning's roughness coefficient. The range of Manning n coefficients given reflects changes in bed material grain size.

A detailed anaysis of bedforms and resistance to flow caused by bedforms can be found in Richardson et al. (1976), Simons and Senturk (1977), and Graf (1971).

Grain Resistance

Flow resistance caused by various grain sizes and particle distributions is well documented in the literature. Chapter 6 of Simons and Senturk (1977) presents an exhaustive review of flow resistance caused by grain size and other factors, Richardson et al. (1974) also provides coverage of flow resistance. Two classic equations for the computation of flow resistance caused by various bed material sizes are given in Equations 2 and 3. Equation 2 (form of Strickler equation) is for use in sand and small gravel bed channels, and Equation 3 (developed by Lane and Carlson) is for use in gravel and cobble bed channels.

$$n = \frac{D_{65}^{1/6}}{29.3} \quad (D_{65} \text{ in ft}) \tag{2}$$

$$n = \frac{D_{75}^{1/6}}{39} \quad (D_{75} \text{ in inches}) \tag{3}$$

Table 12. VARIATION OF MANNING'S n WITH BED CONFIGURATION

BED CONFIGURATION	MANNING'S n*
Plane Bed – No Movement	0.012 – 0.020[a]
Ripples	0.018 – 0.030
Dunes	0.020 – 0.045[b]
Plane Bed – Movement	0.012 – 0.020
Antidunes	0.012 – 0.025
Chutes and pools	0.018 – 0.040

*Variation in n due to grain size range.
[a]Resistance due solely to grain roughness.
[b]Form roughness equal to or larger than grain roughness.

Water Surface Profiles

Another important consideration in the analysis of aggradation and degradation problems is the computation of water surface profiles. Water surface profiles are determined by calculating the water surface elevation, depth, velocity, and head losses along a channel for steady, nonuniform flow. Engineers responsible for highway crossing design are generally familiar with these computations.

The first step in calculating water surface profiles is to determine the type of backwater curves that would exist. Classification of backwater curves is presented in detail in Chapter II of Richardson et al. (1974). The second step is to perform the numerical computations. Determining the type of backwater curve involves identifying the type of controls operating in a given subreach and then qualitatively determining the backwater profile. The numerical computations use conventional backwater calculations, such as the standard step method [Henderson (1966)]. The calculations can be performed by hand-held calculators or programmed for computer solution; however, they are limited to rigid boundary channels. Additionally, the calculation difficulty increases as the channel geometry changes from a prismatic section to irregular channel sections.

For complex situations involving many bridges, culverts, and long reaches of river, it is often necessary to use a computer program, such as the U.S. Army Corps of Engineers' HEC-2, to compute a water surface profile. This procedure generally requires a considerable amount of manpower to organize and create large input data files describing the river conditions and can result in a costly analysis. However, the use of such models is the only feasible way to analyze large, complicated systems. Additionally, in HEC-2 there is provision for modifying the resistance as the stage changes. This provision is important since the resistance value used in calculation has a significant effect on the results. However, HEC-2 and similar models are rigid-boundary models and care must be applied when using them to analyze alluvial systems.

For a large event such as a 50- or 100-year flood, the sediment transport capacity in the stream and the sediment supply from the upland watersheds will be large. Sediment movement can significantly alter the channel geometry by the process of erosion and sedimentation. A realistic bridge design and flood level analysis must consider the channel as an erodible bed and bank system when passing the flow from significant storms.

Flow through channel bends produces a superelevation effect (the water surface is higher at the concave bank than at the convex bank) that is not accounted for in most techniques for water surface profile evaluation. If it is considered significant, a number of procedures are available to estimate this effect quantitatively after evaluating the water surface profile without superelevation [Richardson et al. (1974)]. However, in alluvial channels the added complexity from erosion on the concave bank and deposition on the convex bank inhibits accurate evaluation of superelevation effects.

When neither sufficient time nor money are available to conduct a thorough backwater analysis, a water surface profile may be synthesized by a series of normal depth calculations. Normal depth may be calculated using the Manning equation:

$$Q = \frac{1.0}{n} A R^{2/3} S^{1/2} \tag{4}$$

where

- n is the Manning resistance parameter,
- R is the hydraulic radius (m),
- A is flow area (m^2),
- S is slope, and
- Q is discharge (cms).

For a trapezoidal channel, Equation 4 can be expressed as

$$Q = \frac{1.0}{n} \frac{(Z y^2 + by)^{5/3} S^{1/2}}{[b + 27 (1 + Z)^{1/2}]^{2/3}} \qquad (5)$$

where

- Z is the side slope angle (or horizontal to vertical),
- b is the bottom width (m), and
- y is the depth (m).

Equation 5 may be solved for y in terms of the other known parameters by a trial-and-error method such as that of Newton Raphson. The solution technique can be performed by hand, programmed on hand-held programmable calculators, or solved on a computer. The Newton method requires an initial estimate relatively close to the actual answer for rapid convergence.

Water surface profiles and other hydraulic computations need continued updating when significant changes take place within a watershed. These changes are brought about by the dynamic nature of rivers and are the result of both natural forces and human activities. These factors are discussed in Chapter II.

GEOMORPHIC RELATIONSHIPS

Geomorphic relationships have been developed that describe hydraulic geometry and longitudinal stream profiles. These relationhips can be used to predict the width, depth, and slope of rivers in equilibrium (or quasi-equilibrium).

Hydraulic Geometry

Hydraulic geometry is a general term used to denote relationships between bankfull discharge, channel morphology, hydraulics, and sediment transport. In natural alluvial channels, the morphologic, hydraulic, and sedimentation characteristics of the channel result from a variety of factors. Generalized hydraulic geometry relationships apply to channels having similar hydraulic and geomorphic characteristics. Generally these relationships can be evaluated using data available from rivers within similar physiographic regions. Hydraulic geometry relationships express the integral effect of all hydrologic, meterologic, vegetative, and geologic variables in a drainage basin.

The development of quantitative relationships for river geometry has been attempted by numerous investigators, notably by Leopold and Maddock (1953) Leopold and Wolman (1957), Blench (1966), Langbein et al. (1964), and Engelund and Hansen (1967), not to mention the regime canal researchers. In the most general form, regime formulas can be given as functions of the discharge as follows:

$$W = a Q^b; \qquad (6)$$

$$y_o = c Q^f; \qquad (7)$$

$$V = kQm; \qquad (8)$$

$$Q_t = PQ^j; \qquad (9)$$

$$S = tQ^z; \text{ and} \qquad (10)$$

$$n = rQ^y. \qquad (11)$$

where

W is the channel width,

y_o is the channel depth,

V is the average velocity of flow,

Q_t is the total bed material load,

S is the energy gradients,

n is Manning's roughness coefficient, and

Q is the discharge.

Leopold and Maddock (1953) have shown that in a drainage basin, the two types of hydraulic geometry relations that can be defined are (a) those relating W, y_o, V, and Q_t to the variation of discharge at a station and (b) those relating these same variables to the discharges of a given frequency of occurrence at various stations on a channel. Because the total bed material load was not available, Leopold and Maddock (1953) used the suspended load transport rates, Q_s. The former relationships [(a) above] are called at-station relationships and the latter [(b) above], downstream relationships.

Hydraulic geometry relationships were theoretically developed by Li et al. (1976). These relationships are almost identical to those proposed by Leopold and Maddock (1953). The at-station relationships developed by Li et al. (1976) are:

$$W \propto Q^{0.26} \tag{12}$$

$$y_o \propto Q^{0.46} \tag{13}$$

$$S \propto Q^{0.00} \tag{14}$$

$$V \propto Q^{0.30} \tag{15}$$

Equation (14) implies that slope is constant at a cross section, which is not precisely true. At low flow, the effective channel slope is determined by the thalweg that flows from pool through crossing to pool. At higher stages, the thalweg straightens somewhat, thus shortening the path of travel and increasing the local slope.

The derived downstream relationships for bankfull discharge, as presented by Li et al. (1976) are:

$$y_b \propto Q_b^{0.46} \tag{16}$$

$$W_b \propto Q_b^{0.46} \tag{17}$$

$$S \propto Q_b^{-0.46} \tag{18}$$

$$V_b \propto Q_b^{0.08} \tag{19}$$

where

b indicates the bankfull condition.

An additional degree of freedom must be considered for channels that are not laterally confined. To describe meandering tendencies, the following relationships have been suggested by Blench (1966):

$$L = 10\,W$$

$$M_R = M_b/L$$

where

W = channel width,

L = the meander wavelength,

M_R = the meander ratio, and

M_b = the meander breadth.

For sinusoidal flood plain meanders, M_R is approximately equal to 0.5. For incised meanders, a value of 1.5 is suggested.

Longitudinal Stream Profile and Grain Size Distributions

The longitudinal profile of a stream shows its slope or gradient. It is a representation of the ratio of the fall of a stream to its length over a given reach. Because a river channel is often steepest in its upper regions, most river profiles are concave upward. As with other channel characteristics, the shape of the profile is a result of a number of interdependent factors, representing a balance between the transport capacity of the stream and the size and quantity of the sediment load supplied.

Shulits (1941), among others, provided an equation describing the concave horizontal profile of a channel in terms of distance along the stream as

$$S_x = S_o\, e^{-\alpha x} \tag{20}$$

where

S_x is the slope of any station a distance x downstream of a reference station,

S_o is the slope at the reference station, and

αx is a coefficient of slope reduction.

Similarly, grain size of the bed material decreases in a downstream direction. Transport processes reduce the general size of sediment particles by abrasion and hydraulic sorting. Abrasion is size reduction by mechanical actions such as grinding, impact, and rubbing, while hydraulic sorting results in differential transport of particles of varying sizes. For sedimentary particles of similar shape, roughness, and specific gravity, the end result of these processes is the observed reduction of bed material size along the direction of transport. The change in particle size with distance downstream can be expressed as

$$D_{50x} = D_{50o}^{\;e^{-\beta x}} \tag{21}$$

where

D_x is size of bed material at distance x downstream of a reference station,

D_0 is median size of bed material at the reference station, and

β is a wear or sorting coefficient.

This trend is found in large and small channels.

The longitudinal profile of an alluvial river is dynamic, continually adjusting to changed input conditions of water and sediment discharge. Altered input conditions change the channel geometry, roughness,and other parameters, including channel gradient.

Additional information related to these concepts is presented in Simons and Senturk (1977) and Richardson et al. (1974).

SEDIMENT TRANSPORT THEORY

Understanding the basic concepts of sediment transport is essential to analyzing aggradation/degradation problems. Sediment transport theory is presented in numerous texts. Some of the more thorough coverages are presented in ASCE (1971), Simons and Senturk (1979), and Richardson et al. (1974). To provide a background for the analysis techniques to be presented, sediment transport concepts will be briefly reviewed.

Factors Affecting Sediment Transport and Deposition

The amount of material transported or deposited in a stream under a given set of conditions is determined by the interaction of two groups of variables. The first group influences the quantity and character of sediment brought down to a section of a stream, and the second group influences the capacity of the stream to transport that sediment. A list of these variables is given in Table 13.

It is difficult to compute the supply of sediment that will be brought down to the stream because of the complexity of the variables involved. The most accurate method to make such information available is to measure the sediment flow over a considerable time period. A few of the more commonly used methods for determining sediment yield were summarized by an ASCE Task Committee (1970).

The variables in the second group that deal with capacity of the stream to transport solids are more subject to mathematical analysis and prediction. They are closely related to the hydraulic variables controlling the capacity of the stream to carry water, which in turn are closely related to the forms of bed roughness of the stream and their resistance to flow. Sediment transport equations that determine transport capacity of streams are discussed later.

Table 13. FACTORS THAT AFFECT SEDIMENT TRANSPORTATION AND DEPOSITION

GROUP	TYPE VARIABLE	VARIABLE
1. Sediment brought down to stream	Qualitative	Geology and topography of watershed; magnitude, intensity, duration, distribution, and season of rainfall; soil condition; vegetal cover, cultivation, and grazing; surface erosion; bank cutting
	Quantitative	Size; settling velocity; specific gravity; shape; resistance to wear; state of dispersion; cohesion
2. Capacity of stream to transport sediment	Geometric properties	Depth; width; form; alignment
	Hydraulic properties	Slope; roughness; hydraulic radius; discharge; velocity; velocity distribution; turbulence; tractive force; uniformity of discharge.

Incipient Motion Considerations

Before soils can be transported through a system, some critical condition must be reached above which sediment motion will occur. This incipient motion condition is reached when the hydrodynamic forces acting on the grain of sediment particle have attained a value that, if increased even slightly, will move the grain. Under critical conditions, or at incipient motion, the hydrodynamic forces acting on the grain are just balanced by the resisting forces of the particle.

Sediments are broadly classified as cohesive and noncohesive. The resistance of cohesive sediment to erosion depends on the strength of the cohesive bond of the particles. Cohesion may far outweigh the influence of other physical characteristics of the individual particles. However, once erosion has taken place, cohesive material may become noncohesive with respect to transport. Sediment characteristics may also change through chemical or physical reactions. On the other hand, noncohesive sediments generally consist of larger discrete particles than cohesive soils. Noncohesive sediment particles react to fluid forces, and their movement is affected by the physical properties of the particles, such as size, shape and density.

Cohesive Materials

Before cohesive soils can be eroded, the interparticle bond must be broken. Thus, a critical shear stress has to be exceeded before erosion of a cohesive soil can occur. Considerable research has been conducted in recent years on the factors that influence the critical shear stress of saturated cohesive soils. An excellent review of the literature is made in Alizadeh (1974) and Kandiah (1974).

The resistance of a cohesive bed to erosion by flowing water depends upon (a) the types of clay minerals that constitute the bed; (b) the structure of the bed (which in turn depends on the environment in which the aggregates that formed the bed were deposited), time, temperature, and the rate of gel formation; (c) the chemical compositions of the pore and eroding fluids; (d) stress history, i.e., the maximum overburden pressure the bed had experienced and the time at various stress levels; and (e) organic matter and its state of oxidation.

To model the transport process, it is necessary to know the critical shear stress of each stratum of the bed and also the erosion rate if the erosive mechanism is surface erosion. At present, laboratory measurements must be made to obtain these parameters. The critical shear stress for scour and rates of erosion may be measured in a flume for beds of relatively low strength. Stronger beds may be tested in the rotating cylinder apparatus by the method described by Sargunam et al. (1973), although this method is not suitable for thin layers.

In cases in which it is not possible to conduct the detailed laboratory measurements described above, one of several regression equations can be used to relate the physical properties of the cohesive material to its critical shear stress. The physical properties include plasticity index, in-place maximum soil density, soil gradation, and liquid limit. Although these properties must also be obtained through laboratory analysis, the analysis procedures are less complex than those discussed above.

A review of regression equations for the analysis of critical conditions in cohesive materials is presented in ASCE (1968). The most reliable of the equations presented was

$$\tau c = 9.35 \times [-0.00124 + 0.00081\,PI + 0.00030\,D\% + 0.00022\,M_\Phi\,\delta_\Phi\,K_\Phi] \qquad (22)$$

where

τc = critical shear stress (kg/m^2),
PI = plasticity index
D% = in place maximum soil density,
$M_\Phi\,\delta_\Phi$
K_Φ = a measure of soil gradation

Equation (22) was developed by C. W. Thomas and P. F. Enger (1961). Its limits of applicability are:

$0 < PI < 22$,
$65 < D\% < 100$,
$-12 < M_\Phi, \delta_\Phi, K_\Phi < 40$, and
$13 <$ liquid limit < 42.

Care must be taken when applying an equation that was developed through regression techniques to be sure that the material being analyzed has characteristics similar to that of the data base on which the equation was developed. The data base was made up of rivers from west of the Mississippi River only.

Noncohesive Materials

The beginning of motion of noncohesive bed material is affected by the physical properties of the particles, such as size, shape, density, and gradation.

Of the various sediment properties, size has the greatest significance to the hydraulic engineer, not only because size is the most readily measured property but also because other properties, such as shape and specific gravity, tend to vary with particle size. In fact, size alone has been found to be sufficient to describe the sediment particle for many practical purposes.

Size may be measured by calipers, optical methods, photographic methods, sieving, or sedimentation methods. While the size of an individual particle is important, the size distribution or gradation of the sediment that forms the bed and banks is of primary importance.

The beginning of motion of bed material is known to be a function of the dimensionless number

$$\tau_c/(\gamma_s - \gamma)\,D_s \tag{23}$$

where

τ_c is the critical boundary shear stress,

γ_s and γ are the specific weights of the sediment and water, respectively, and

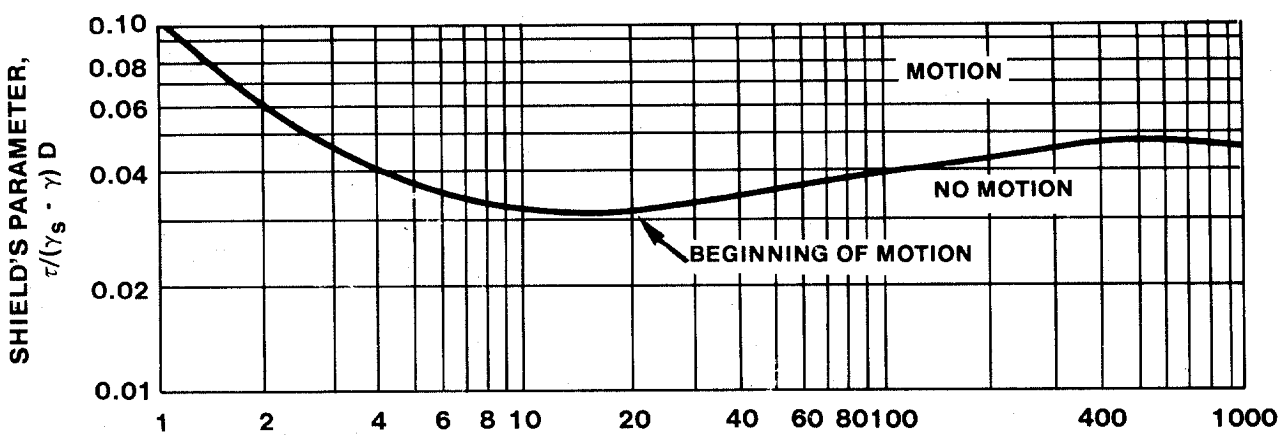
D_s is a characteristic diameter of the sediment particle.

This number is often referred to as the Shields parameter. Shields determined the graphical relationship that is given in Figure 37 by measuring bed load transport for various values of Equation (23) at least twice as large as the critical value and then extrapolating to the point of vanishing bed load. This indirect procedure was used to avoid the implications of the random orientation of grains and variations in local flow conditions that may result in grain movement even when the matnitude of Equation (23) is considerably below the critical value.

Shields' diagram (Figure 37) can be divided into three regions:

$$\textit{Region } I^{\dagger}: \quad \frac{U_* D_s}{\nu} < 3.63 - 5.0 \tag{24}$$

†The limit 3.63 is sometimes replaced with 3.3, 3.5, or 2.0.



Source: Gessler (1971)

Figure 37. SHIELD'S RELATION FOR BEGINNING OF MOTION

where

U_*D_s/ν = shear Reynold's number,
U_* = shear velocity, and
ν = kinematic viscosity of fluid.

In Region 1, $D_s < 3\delta$, the boundary is considered hydraulically smooth (δ is the thickness of the laminar boundary layer). The portion of the diagram for $U_*D_s/\nu < 2$ was estimated by Shields who did not perform any experiments in that region.†

According to Shields, when the value of

$$\frac{\tau_c}{(\gamma_s - \gamma)\, D_s} = 0.1 \tag{25}$$

then (approximately)

$$\frac{U_*D_s}{\nu} = 1 \tag{26}$$

Region 2: $$3.63 - 5.0 < \frac{U_*D_s}{\nu} < 68.0 - 70.0 \tag{27}$$

In this region the boundary is in a transitional state and $\delta/3 < D_s < 6\delta$. The Shields diagram has a form similar to the relationship of Darcy-Weisbach's resistance coefficient, f, versus Reynolds number, R_e. It is also similar in form to the relationship between the drag coefficient, C_D, and Reynolds number for cylindrical bodies.

Region 3: $$\frac{U_*D_s}{\nu} > 68.0 - 70.0 \tag{28}$$

The critical shear stress is given by

$$\tau_c = \gamma RS \tag{29}$$

where

R is the hydraulic radius.

†The original curve given by Shields ends at around $U_*D_s/\nu=1$. Points with the shear Reynold's number at lesss than 1.00 have been provided by other researchers.

It can also be expressed as

$$\tau_c = \rho U_*^2\,. \tag{30}$$

where

ρ is the fluid density.

The criterion of beginning of motion established U_* and τ; the shear stress, τ, is assumed constant at a point in the channel when the hydraulic characteristics of a flow and channel geometry remain constant, but it varies in the cross section (Figure 38). This variation has been reported by Lane (1953) and others. The shear distribution is based upon a theoretical analysis that utilizes a membrane analogy and field data. However, if the channel is curved or if the geometry is different, the coefficients 0.75 and 0.97 as shown in Figure 39 are not valid. For example, in bends, coefficients as high as 2.5 were measured by Ippen and Drinker (1962).

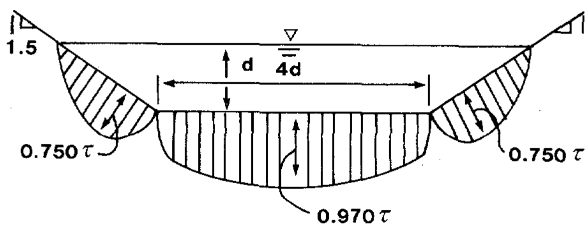


Figure 38. MAXIMUM UNIT TRACTIVE FORCE VS. b/d

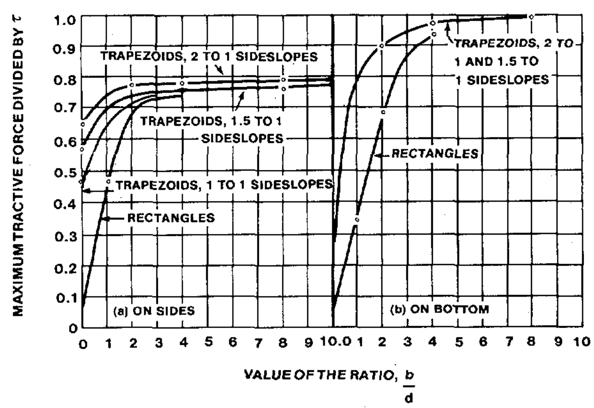


Figure 39. VARIATION OF τ IN A TRAPEZOIDAL CROSS SECTION

Based on the foregoing analysis and field studies, Figure 39 was prepared by Lane and Carlson (1953). For different values of width-to-depth ratio, b/d, values of $\tau_c/\tau dS$ (d is assumed equal to R for wide channels) are given for the side slopes and the channel bottom. The critical shear stress, τ_c, is given in Equation (29), b/d is assumed or known, and it is possible to estimate the shear on the side slopes and the channel bottom from Figure 39.

Transport Concepts

Sediment particles are transported by one of a combination of the following mechanisms: (a) rolling or sliding on the bed (surface creep), (b) moving in short steps but periodically coming to rest on the bed (saltation), and (c) supported by the surrounding fluid during its entire motion (suspension). Sediments that move as surface creep or saltation and are supported by the bed are called bed load. Sediments that are suspended and supported by the flow are called suspended load. The sum of the bed load and suspended load is the total bed material load.

That part of the sediment load smaller than representative bed material supplied from upstream sources is termed wash-load. There is no clear distinction between wash load and bed material load. As a rule of thumb, it is sometimes assumed that bed-material load size is equal to or greater than 0.0625 mm, the division point between sand and silt. A more reasonable criterion, although not necessarily theoretically correct, is the use of a sediment size finer than 10 percent of the bed sample as the division point between wash load and bed material load. It is assumed that most of the wash load is transported through the system by stream flow and little is deposited on or in the streambed. Wash load thus deposited with the coarse material is usually a fraction of the total bed material.

The sum of bed material load and wash load becomes the total sediment load. Since the sediment transport equations can determine merely the bed material load, total sediment load can be predicted only if the wash load is estimated by measurement, empirically, or by analytical relations.

Bed Load

When the flow over movable boundaries of a channel has hydraulic conditions that exceed the critical condition for motion of the bed material, sediment transport starts. When the motion of entrained particles is one of rolling, sliding, and sometimes jumping in the bed layer, it is commonly referred to as bed load transport or contact load.

Generally, the amount of bed load transported by a large river is about 5 to 25 percent of the suspended load. Although the amount of bed load may be small compared with total sediment load, it is important because it shapes the bed and influences channel stability, bed roughness form, and other factors.

DuBoys (1879) initiated the idea of the bed shear stress or tractive force in the analysis of bed load. Since then, numerous bed load equations for relating the bed load discharge, flow condition, and composition of the bed material have been proposed. Many of these equations look similar because they are derived under conditions of steady flow in graded channels and thus are only applicable under those conditions. There is a great need for research on the transport of sediment when flow is unsteady, nonuniform, and both slowly and rapidly varying with time.

The goal of bed load equations is to predict the amount of bed load transport in a natural channel. With limited knowledge, accurate prediction of the amount is an extremely difficult task. Most bed load equations were derived from laboratory flumes, and none has been thoroughly tested with field data because of insufficient field data, the cost, and measuring difficulty. Therefore, the accuracy of applying the bed load equations presented here to large rivers cannot be determined. In other words, the bed load equation selected must be based on the investigator's experience, usually without actual bed load measurements to verify it.

Suspended Load

Part of the sediment transported by the flow in streams is suspended in the flow. The weight of these sediment particles is continuously supported by the surrounding fluid. Turbulence is the most important factor in suspension of sediment. The particle weight causes settling that is counterbalanced by the irregular motion of the fluid particles introduced through turbulent velocity components.

Most early studies concerned with suspension of sediment are unacceptable when considered in relation to the present knowledge of fluid motion. Generally acceptable aspects of suspended sediment transport will be discussed in subsequent sections.

Only that part of suspended load composed of bed material load can be evaluated by applying sediment transport theory since the wash load transport is determined by the available upslope supply rate. Thus, the relationships derived in the following sections for determining sediment concentration and suspended load are restricted to bed material load.

Sediment Transport Equations

Many transport equations are available, ranging in complexity from those that require simple graphical procedure to those that require many calculations. In most cases the development or derivation of the equations is based on some specific bed material type and/or transport mechanism. There is no one transport equation that is applicable to all situations [ASCE Task Committee (1975)]. Therefore, it is important that bed material type and transport mechanism be considered before applying transport equations to a given situation.

Several of the more generally accepted transport equations will be reviewed here. These equations are most clearly presented by considering those used for bed load, suspended load, and total bed material load. A transport equation for cohesive materials is also presented. Detailed derivations of the equations presented are available in various texts and publications, specifically Simons and Senturk (1977) and ASCE Task Committee (1975).

Bed Load Sediment Transport Relationships

Many bed load equations are based on excess shear stress, i.e., $\tau_o - \tau_c$ or $\tau_o' \tau_c$. For example, the DuBoys' (1879) relation

$$q_{bv} = K \tau_o (\tau_o - \tau_c) \tag{31}$$

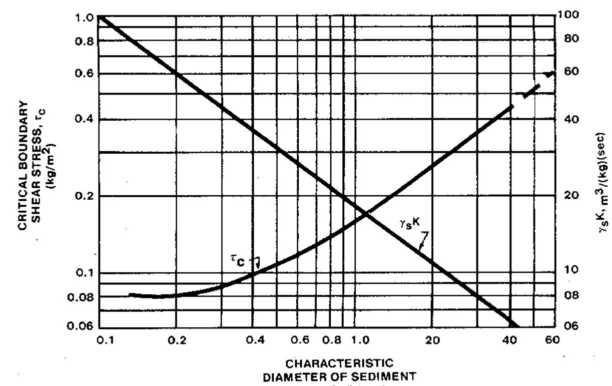
where

q_{bv} is the volume transport rate of bed load,

K is the constant, and

τ_o and τ_c are the unit tractive force for boundary shear stress and the critical tractive force, respectively.

This was one of the first bed load expressions developed. Values of K and τ_c obtained by Straub and reported by Brown (1950) are given as functions of median size of bed sediment, D_{50}, in Figure 40. DuBoys' work has been criticized mainly because all the data that he utilized was obtained using small laboratory flumes.



Source: Straub (1935)

Figure 40. SEDIMENT COEFFICIENT AND CRITICAL TRACTIVE FORCE FOR DUBOY'S BED LOAD EQUATION

Schoklitsch (1930) stated that the average bed shear stress in DuBoys' equation is a poor criterion when applied to field computations because the shear distribution in the channel cross section is nonuniform. He suggested two equations. Schoklitsch's critical value, q_c, for the discharge after which sediment motion begins was defined as

$$q_c = 0.26 \left(\frac{\gamma_s'}{\gamma}\right)^{5/3} \frac{D_s^{3/2}}{S^{7/6}} \tag{32}$$

where

D_s is the characteristic sediment diameter.

Using this value

$$q_{bw} = 2500.00\, S^{3/2} (q - q_c) \tag{33}$$

where

q is the total discharge per unit width of channel was suggested to compute the bedload. q_{bw} represents the sediment discharge on a dry weight basis.

The Schoklitsch formula was based mainly on data from experiments by Gilbert (1914) in small flumes with well-sorted and also graded sediments with median sizes ranging from 0.3 to 5 mm. Sediment discharges calculated with the formula also agreed well [Shulits (1935)] with bed load discharges measured with samplers in two European rivers that have gravel beds. This suggests that it is a bed load formula that should not be applied to sand-bed streams that carry considerable bed sediment in suspension.

Meyer-Peter and Muller (1948) developed a formula similar in form to the DuBoys' formula based on experiments with sand particles of uniform size, sand particles of mixed sizes, natural gravel, lignite, and baryta. The results are similar to DuBoys' formula, but the method of solving problems is different.

The Meyer-Peter and Muller equation is

$$\gamma_w \left(\frac{Q_s}{Q}\right)\left(\frac{k_s}{k_r}\right)^{3/2} YS =$$

$$A\ \gamma_s''\ D_m + B \left(\frac{\gamma_w}{g}\right)^{1/3} q_{bw}^{2/3} \tag{34}$$

where

γ_w = specific weight of water (metric tons per cubic meter);

Q_s = water discharge quantity determining the bedload transport (liters per second);

Q = total water discharge quantity (liters per second);

k_s = the bed roughness (meters$^{1/3}$ per second);

k_r = particle roughness $= \dfrac{C}{D_{90}^{1/6}}$ (meters$^{1/3}$ per second); where C = 26 (meters$^{1/2}$ per second);

Y = depth of flow (meters);

S = slope of the energy line;

A = dimensionless constant $\approx$ 0.047;

γ_s'' = specific weight of sediment in water $= \gamma_s - \gamma_w$ (metric tons per cubic meter);

D_m = effective diameter of bed material (meters), $D_m = \dfrac{\Sigma D \Delta p}{100}$ where D is average size of particles in a size fraction, and p is percent in that size fraction;

B = dimensionless constant $\approx$ 0.25;

g = acceleration due to gravity (meters per second^2); and

q_{bw} = the specific bed load transport weighed under water (metric tons per meter of width per second).

The U.S. Bureau of Reclamation (1960) reduced Equation (34) to the form

$$q_{bv} = \frac{12.85}{\sqrt{\rho \gamma_s}} (\tau - \tau_c)^{1.5} \tag{35}$$

in which

$$\tau_c = \delta_s (\gamma_s - \gamma) D_s \tag{36}$$

where

q_{bv} is the bedload transport rate in volume per unit width for a specific size of sediment,

τ_c is critical tractive force,

ρ is density of water,

γ_s is specific weight of sediment,

γ is specific weight of water, and

δ_s is a constant dependent on flow conditions.

Gessler (1965) showed that δ_s should be 0.047 for most flow conditions. If rilling develops on the overland flow surface, the value of δ_s should be reduced.

The Meyer-Peter and Muller formula is based on data from experiments in flumes ranging in width from 15 cm to 2 m, with slopes varying from 0.0004 to 0.02 and water depths ranging from 1 to 120 cm.

The mean sizes and effective diameters, d_m, of the sediments ranged from 0.4 to 30 mm. The advantage of this formula is that it can be used for graded sediments under flow conditions that give rise to dunes and other bed forms. Most of the data upon which the formula is based were obtained in flows with little or no suspended load that suggests that the formula is not valid for flows with appreciable suspended loads. The Meyer-Peter and Muller equation is a simple and commonly used bed load equation. It can be used to predict total bed load transport or transport of individual size fractions.

Although it is not necessary to use the Meyer-Peter and Muller equation, an equation should be chosen that is applicable to the field conditions. The general bed load function can be expressed in the functional form of

$$q_b = a_4 (\tau_o - \tau_c)^{b_4} \tag{37}$$

where

a_4 and b_4 are constants.

Einstein (1942, 1950) departed from the concepts of the DuBoys-type and Schoklitsch-type equations. In his method, the sediment discharge is computed for individual size fractions of the bed material. The technique is extremely complex and is best suited for computer application. The equations and relationships used in the calculations follow:

$$q_{bv} = \Sigma\, q_{bv_i} \tag{38}$$

$$Q_s = b q_{bv_s} \tag{39}$$

$$q_{bv_i} = q_{sb_i} [P_r I_1 (N_{oi}, Z_i) + I_2 (N_{oi}, Z_i) + I] \tag{40}$$

In these equations

q_{bv} = discharge of bed sediment per unit width,

Q_s = total bed sediment discharge,

b = bed width of stream,

q_{bv_i} = discharge of bed sediment of mean size d_{si} per unit width, and

q_{sb_i} = discharge of bed load of mean size d_{si} per unit width.

The terms in Equation (40) are defined by figures and graphs [ASCE Task Committee (1975)].

The application of the Einstein procedure to a particular water course can be subdivided into three parts. The first part involves choice of a river reach and collection of required field data; the second part is to determine the required hydraulic parameters; and the third part uses the results of the other parts to compute total bed material discharge. The procedure requires many calculations and is usually arranged in tabular form. The development and use of the equations and example calculations are presented in Simons and Senturk (1977).

The development of Einstein's equations were based on flume data and then verified through field application. The flume experiments were conducted using graded to well-sorted fine sand to small gravel size bed materials. The experiments were run producing high transport rates. Therefore, this technique is well suited for application under conditions of high suspended as well as bed load transport in file sand to gravel bed rivers.

Although there are many bed load equations available, Laursen (1956) found that they could all be reduced to essentially the same simplified form. The form arrived at was

$$q_{bw} = K\, U^r D_s^x \tag{41}$$

where

K = an empirical coefficient that is a function of the characteristics of channel and bed sediment,

U = the flow velocity, and

r and x = exponents

where

$r > o$ and $x < o$.

Equation (41) can be used in two ways. First, if enough transport data are available for a river, the coefficients in Equation (41) can be calibrated producing a simple, accurate transport equation for a given river. Second, the parent equation could be used to generate a set of transport data that could then be fitted to Equation (41).

Suspended Load Transport Relationships

Part of the sediment transported by the flow in streams is suspended in the flow. The weight of these sediment particles is continuously supported by fluid. Turbulence is the most important factor in suspension of sediment. Because of the particle weight, there is settling that is counterbalanced by irregular motion of the fluid particles introduced through turbulent velocity components.

Detailed discussions of the theory of suspended load transport are contained in Simons and Senturk (1977). Some general considerations are included here.

The suspended load discharges per unit width of Channel q_s for two-dimensional flow is given by the relationships

$$q_{sv} = \int_a^d \bar{u}\ \bar{C}\ dy \tag{42a}$$

and

$$q_{sw} = \gamma_s \int_a^d \bar{u}\ \bar{C}\ dy \tag{42b}$$

where

- $\bar{u}$ and $\bar{C}$ are the time-averaged flow velocity and concentration at y, respectively,
- y is distance above the bed,
- a is the bed layer thickness,
- C is the sediment concentration by volume, and
- q_{sv} is calculated as the volume of sediment per unit time and width [Equation (42b)].

The discharge of sediment for the entire stream cross section, Q_s, is obtained by integrating Equation (42) over the cross section. Then,

$$Q_{sv} = Q\ \bar{C} \tag{43a}$$

and

$$Q_{sw} = \gamma_s\ Q\ \bar{C} \tag{43b}$$

where

- $\bar{C}$ is the average suspended sediment concentration by volume.

To integrate Equation (42), $\bar{u}$ and $\bar{C}$ must be expressed as a function of y. The velocity distribution can be expressed as

$$U = U_* \left(\frac{1}{\kappa}\ \ln y/y_o\right) \tag{44}$$

where

- κ = Van Karman constant,
- U_* = shear velocity = $\sqrt{\tau/\rho}$,
- y = depth, and
- y_o = depth at turbulent boundary.

The suspended sediment distribution equation has been presented by Roose as

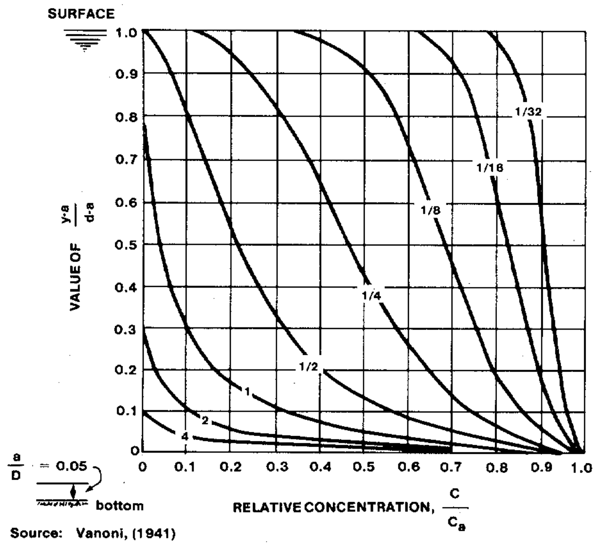
$$\frac{C}{C_a} = \left[\frac{d - y}{y}\ \frac{a}{d - a}\right]^{z_1} ; z_1 = \frac{w}{\beta \kappa U_*} \tag{45}$$

where

- β is a correlation coefficient.

Figure 41, shows a family of curves obtained by plotting Equation (45) for different values of the exponent z_1.

One of the most widely recognized methods used to compute suspended sediment load is that proposed by Einstein (1950). The procedure utilizes a compiicated expression based on Equations (45) and (42b) and must be solved by numerical integration or graphic techniques. The resulting expresssion is



Source: Vanoni, (1941)

Figure 41. SUSPENDED LOAD DISTRIBUTION FOR a/D=0.05 AND SEVERAL VALUES OF z_1

$$q_{sw} = 11.6\, U'_* \, C_a \, a \left[2.303 \log \left(\frac{30.2\, d}{\Delta}\right) \cdot I_1 + I_2\right] \quad (46)$$

where

- Δ is a correction factor,
- U'_* is the shear velocity due to grain roughness, and
- I_1 and I_2 are the expressions that must be numerically or graphically solved.

Einstein (1950) further assumed that the bed layer thickness was

$$a = 2D$$

and within this layer, the suspension of material is impossible. However, this bed material was assumed to be the source of the suspended load and the lower limit of the reference concentration C_a. From these assumptions, an equation relating bed load transport to suspeneded load transport (for all size fractions for which the bed load function exists) was developed. Detailed descriptions outlining the application of this technique are presented in Simons and Senturk (1977).

Total Load Sediment Transport Relationships

The total load is the sum of bed load and suspended load or the sum of bed material load and the wash load. At low transport rates when most of the sediment moves in contact with the bed or in shallow flow, the bed load may approximate the total load. Conversely, in a deep river, such as the Mississippi River, bed load may only total 10 to 20 percent of the total load.

As discussed previously, bed material load and wash load in a uniform flow are normally dealt with separately because the wash load is determined by the available upslope supply rate and cannot be predicted by the transport capacity of the stream. Thus, only the bed material load that is composed of those solid particles consisting of grain sizes represented in the bed can be predicted by summing up the bed load and the suspended load.

Indirect and Direct Determination of Total Bed Material Load

An indirect approach to determining the bed material load by summing up bed load and suspended load was initiated by Einstein (1950). While, there is no sharp line dividing bed load and suspended load, at least two points warrant this division: (a) the difference in forms of transport requires two physically different models and (b) the two loads are measured by different methods. The bed load is measured using suitable traps placed on or in the bed, and the suspended load is measured by sampling the water sediment mixture.

The bed load function is defined in Einstein (1950) as a function that estimates the rate at which flow of any magnitude in a given channel will transport the individual sediment sizes that compose the channel bed. Application of Einstein's bed load function is too complicated and laborious for practical use. In the original paper, Einstein (1950) gave a detailed sample calculation. His method represents one of the most detailed and comprehensive treatments of bed material transport from a fluid mechanics viewpoint that is currently available.

Most researchers agree with Einstein's approach and believe that bed material load is the summation of fractional loads. Despite this, a group of investigarors feels there is no need to distinguish bed load from suspended load since (a) the hydraulic forces involved are the same for both types of transport, and (b) thickness of bed layer or proper demarcation between the bed load and suspended load is difficult to define. Furthermore, direct approaches have some advantage over indirect approaches because of simpler application procedures, especially for those relationships based upon available data. This procedure eliminates questionable assumptions involved in rational approaches. Furthermore, an empirical relationship between sediment rate and flow is relatively easy to apply and can provide the design engineer with an estimation that is of the correct order of magnitude, assuming the problem in question falls within the range of conditions used to establish the empirical relationships.

After investigating the effect of mean flow velocity, shear, shear velocity computed from mean velocity, stream power of flow, flow depth, viscosity, water temperature, and concentration of fine sediment on the bed material discharge per foot of channel width, Colby (1964) developed four graphic relationships shown in Figure 42 and 43 for determining the bed material discharge. In developing his computational curves, Colby was guided by Einstein's bed load function [Einstein (1950)] and a great amount of data from streams and flumes [Simons and Richardson (1966)]. However, it should be understood that all curves for the 100-ft depth, most curves for the 10-ft depth, and some of the curves for the 1.0-ft and 0.1-ft depths (Figure 42) are not based entirely on data but are extrapolated from limited data and theory.

In utilizing Figures 42 and 43 to compute the bed material discharge, the following procedure is proposed:

- The required data are mean velocity, U; depth, d; median size of bed material, D_{50}; water temperature, T; and wash load, C_f.
- Uncorrected sediment discharge, q_{Ti}, for the given U, d, and D_{50} can be found in Figure 42 by first reading q_{Ti} knowing U and D_{50} for the two depths that bracket the desired depth. Then a logarithmic scale of depth versus q_{Ti} is used to interpolate in order to determine the bed material discharge per unit width for the actual d, U, and D_{50}.
- Two correction factors, k_1 and k_2, shown in Figure 43 account for the effect of water temperature and fine suspended sediment on the bed material discharge. If the bed material size falls outside the 0.2 to 0.3 mm range, factor k_3 from Figure 43 is applied to correct for sediment size effect.

True sediment discharge, q_T, corrected for water temperature effect, presence of fine suspended sediment, and sediment size, is given by

$$q_T = [1 + (k_1 k_2 - 1)\, 0.01\, k_3]\, q_{Ti} \qquad (47)$$

As shown in Figure 43, $k_1 = 1$ when the temperature is 60°F, $k_2 = 1$ when the wash load is negligible, and $k_3 = 1$ when D_{50} lies between 0.2 mm and 0.3 mm.

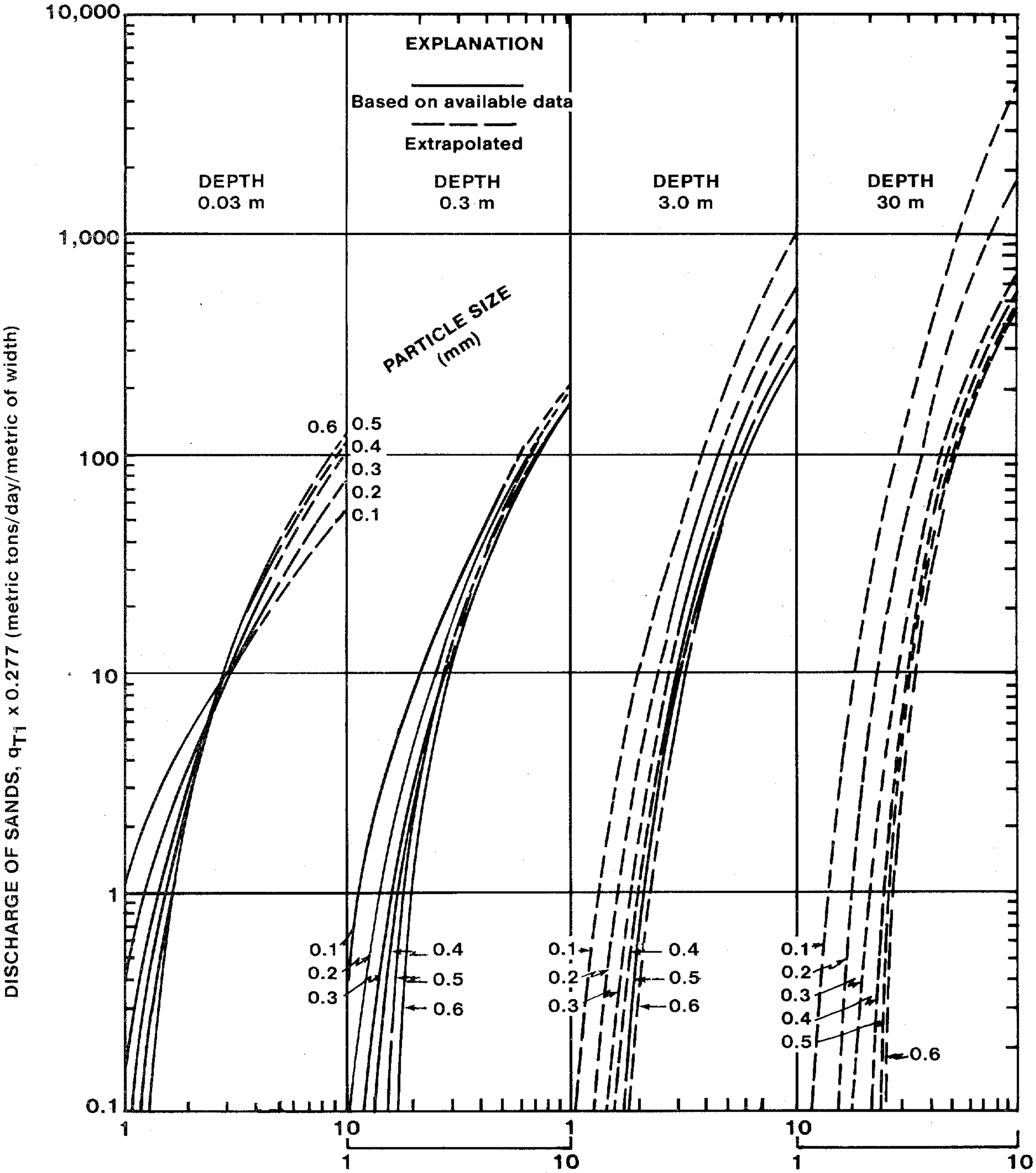
Colby's relations for sediment discharge apply only for sand bed channels.

Calculation of Sediment Discharge from Stream Measurements

Normal measurements made on many streams include discharge measurements as well as suspended load measurements. Discharge measurements consist of determining depth and mean velocity at selected verticals; from these determinations, the total discharge, Q, and the cross sectional area, A, for the channel cross section may be calculated. Suspended load samples give the mean measured sediment concentration, C'_s, at each or at least several of the verticals in which mean velocity is measured. Concentration C'_s includes sediment of all grain sizes, wash load, and bed material load.

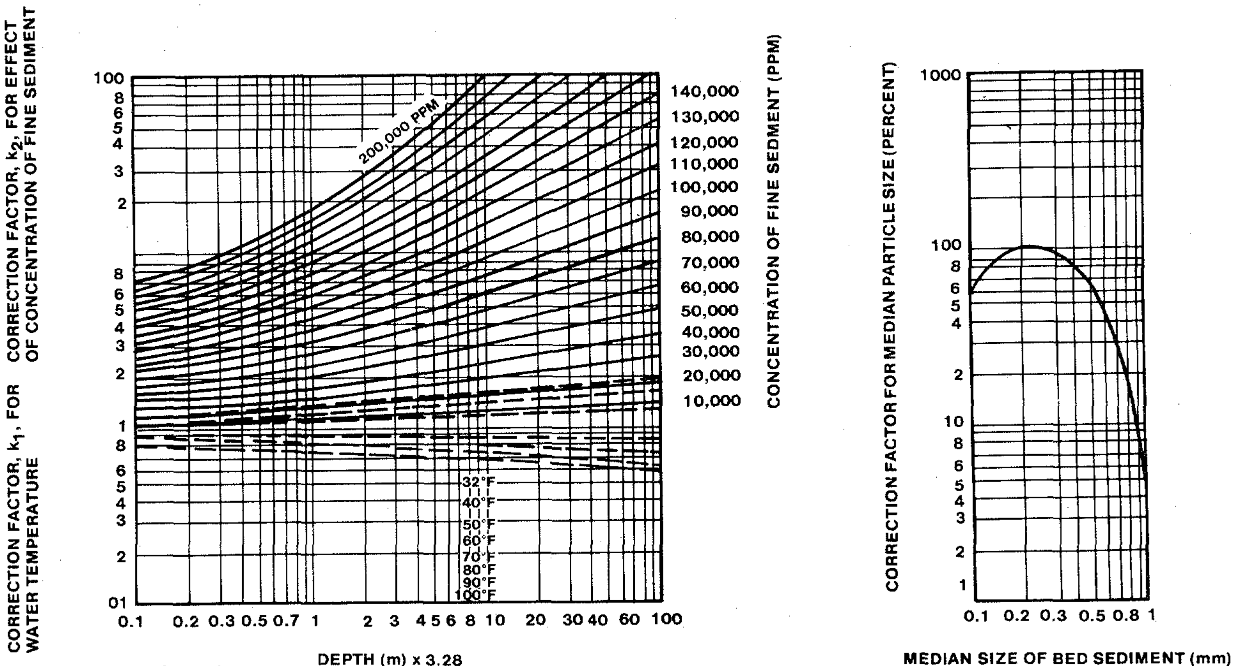
Suspended sediment measurements do not sample the sediment load near the bed which may include bed load and part of the suspended load. The modified Einstein procedure developed by Colby and Hembree (1955) is a means for estimating the total sediment discharge once suspended load samples and other required data have been acquired.

Colby and Hembree (1955) and subsequently others proposed a modified Einstein procedure to



Source: Colby (1964)

Figure 42. RELATIONSHIP OF DISCHARGE OF SANDS TO MEAN VELOCITY FOR SIX MEDIAN SIZES OF BED SANDS, FOUR DEPTHS OF FLOW, AND A WATER TEMPERATURE OF 60°F



Source: Colby (1964)

Figure 43. APPROXIMATE EFFECT OF WATER TEMPERATURE AND CONCENTRATION OF FINE SEDIMENT ON THE RELATIONSHIP OF DISCHARGE OF SANDS TO MEAN VELOCITY

obtain the total sediment transport rate in a river. The term "modified Einstein procedure" usually gives the impression that it serves the same purpose as Einstein's (1950) procedure. Actually, these two procedures, although based on similar principles, serve entirely different purposes. Einstein's procedure, used mainly for design purposes (as well as other equations that predict the bed material discharge), estimates bed material discharges for different river discharges based on channel cross section and sediment bed samples in selected uniform flow river reaches. The modified Einstein procedure, on the other hand, esitmates total sediment discharge (including wash load) for a given water discharge from measured depth-integrated suspended sediment samples, streamflow measurements, bed material samples, and water temperature for specific discharge at the cross section.

The major differences between the modified Einstein in Colby and Hembree (1955) and the original Einstein (1950) procedure include:

- The suspended load exponent, z, used in the original Einstein procedure is determined from the observed z value for dominant grain size. Values of z for other grain sizes are derived from that of dominant size and are assumed to vary with the 0.7 power of their fall velocity.
- A slight change in the hiding factor coefficient (used to reflect the interaction of various grain sizes in a sediment mixture) in the original Einstein procedure is introduced.
- The depth, d, is used in the modified Einstein procedure to replace the hydraulic radius.
- The value of Einstein's original intensity of bed load transport is arbitrarily divided by a factor of two to fit the observed transport data more closely.

Data needed are stream discharge, Q; mean velocity, U; cross sectional area, A; stream width, B; mean value, d_v, of the depths at verticals at which suspended sediment samples were taken; measured sediment discharge concentration, C'_s; size distribution of the measured load, i_s; size distribution of bed material at the cross section, i_b; and water temperature, T. Simons and Senturk (1977) detail the application of the modified Einstein procedure.

That the modified Einstein procedure rendered very good predictions of the total load for the Niobrara and Loup Rivers is demonstrated by Schroeder and Hembree (1956). This procedure should also be generally applicable to streams of different character because much of the sediment discharge is actually measured, particularly for streams of significant depth.

Bed Material Load Transport Relationship For Cohesive Materials

The mode of cohesive sediment transport is different from that of noncohesive sediment transport principally because cohesive sediments move entirely as suspended load whereas noncohesive sediments move as both bed load and suspended load. In addition, cohesive sediments form interparticle bonds as they settle on the bed and consolidate with increasing overburden pressure. Noncohesive sediments, on the other hand, depend only on their weight to resist transport.

To model the transport process it is necessary to know the critical shear stress of each stratum of the bed and also the erosion rate if the erosive mechanism is surface erosion. As mentioned previously, laboratory measurements must be made to obtain these parameters.

The erosion rate for particle erosion is given by Parthenaides (1962) as

$$\left(\frac{d_m}{d_t}\right)_e = M\left(\frac{\tau_b}{\tau_{ce}-1}\right) \qquad (48)$$

where

$\left(\frac{d_m}{d_t}\right)_e$ = mass rate of erosion per unit area,

τ_b = bed shear stress,

τ_{ce} = critical shear stress for erosion, and

M = erodibility constant.

τ_{ce} and M must be evaluated experimentally. If d is the local depth of flow then the rate of change of particle concentration is

$$\left(\frac{d_c}{d_t}\right)_e = \frac{M}{d}\left(\frac{\tau_b}{\tau_{ce}-1}\right) \qquad (49)$$

Equations (48) and (49) have been verified in flume and field tests [Ariathurai and Krone (1976) and Ariathurai (1980)].

Comparison of Bed Material Load Equations and Field Applications

The bed material load equations are established for different conditons, and their use should be restricted to the conditions for which they are applicable. Very few direct measurements of total sediment discharge of rivers have been made. Most of the growing body of data on sediment discharge from the rivers is obtained by adding an estimate of the unmeasured sediment discharge to that measured with suspended load samplers. Although few attempts are discussed, a meaningful comparison of measured and calculated bed material load is difficult.

An ASCE Task Committee (1971) used the field data from the Colorado River at Taylor's Ferry and the Niobrara River near Cody, Nebraska, to evaluate 13 sediment transport equations [Vanoni et al. (1960)]. They include formulas by Blench (1966), Colby (1964), DuBoys (1879), Einstein (1950), Brown and Einstein (1950), Englelund and Hansen (1967), Inglis (1968), Laursen (1958), Meyer-Peter (1934), Meyer-Peter, Muller (1948), Schoklitsch (1934), Shields (1936), and Toffaleti (1969).

On the basis of the comparisons made, sediment discharge formulas cannot be expected to present precise results, and equations should only be applied to cases similar to those for which they have been verified.

Because of the tremendous uncertainties in estimating sediment transporrt, it is difficult to make clear cut recommendations. However, the following procedures are suggested.

When measured data are available, the suitable procedures are

- The modified Einstein method should be used to estimate unmeasured suspended load and bed load based on measured data.
- Bed material load should be separated from wash load and analyzed.
- When only a very limited amount of measured data are available, sediment transport equations that best agree with these data should be selected to estimate the sediment transport load for the design flow.

In the absence of measured data, the following relationships are recommended:

- The Einstein (1950) procedure should be used in sand-bed rivers when the bed load is more than 25 percent of the total bed material load.
- The Colby (1964) method should be used for sand-bed rivers with significant amounts of suspended material.
- The Meyer-Peter, Muller technique should be used for large sand and gravel bed streams containing little or no suspended bed-material load.
- Cohesive bed channels should be analyzed using the techniques of Parthenaides (1962) and Ariathurai and Krone (1976).

Table 14 shows the applicability of some sediment transport relationships to various conditions. The relationships presented do not comprise a complete list of available transport equations.

SEDIMENT YIELD

The volume of sediment delivered to a channel from overland areas is an important input to channel stability considerations. The quantity of sediment delivered to the channel depends on various watershed characteristics such as climate, land use, vegetal cover, soil type, overland slope, etc. It also depends on upland erosion rates, which are functions of the type or source of erosion. These sources include sheet erosion, rill erosion, and gully erosion.

Table 14. APPLICABILITY OF SEDIMENT TRANSPORT RELATIONS

MATERIAL	METHOD	WASH LOAD	BED MATERIAL LOAD		
			SUSPENDED	BED LOAD	TOTAL LOAD
Wash Load	Direct Measurement	X	X		
	Estimated from watershed and channel response	X	X		
Sand	Einstein		X	X	
	Modified Einstein	X	X	X	X
	Colby	Measured or estimated	X	X	X
	Schoklitsch			X	
Gravel and Cobble	Schoklitsch			X	
	Meyer-Peter, Muller			X	
Cohesive	Ariathurai, Krone/ Parthenaides	Measured or estimated	X		

A complete description of the various techniques available for predicting rates of sediment yield is beyond the scope of this report. A review of such techniques is given in Chapter IV of "Sedimentation Engineering" [ASCE Task Committee (1975)]. The most universally used technique is the Universal Soil Loss Equation (USLE). A review of a modified version of the USLE is presented below. The USLE is typical of the regression type equations used to predict sediment yields for upland areas. A modified version of the USLE is used to compute sediment yields from watershed. The modifications allow for prediction of soil loss resulting from individual storms. The rainfall energy factor, R, is replaced by a runoff factor that is closely related to soil loss. The equation is:

$$Q_w = 95\,(Q\,q_p)^{.56}\,K\,LS\,C\,P \tag{50}$$

where

- Q_w is the soil loss in tons,
- Q is the volume of runoff in acre-feet,
- q_p is the peak runoff from the storm in cfs, and
- K, LS, C, and P are universal soil loss equation parameters.

If records of Q and q_p are available, Equation (50) can be used to estimate sediment yield. If flow records are not available, Q and q_p must be determined from watershed characteristics, basic land use information, and rainfall data.

CHAPTER VI

METHODS OF DETERMINING GRADE CHANGES

The three basic procedures for predicting grade changes are (a) use of geomorphic techniques; (b) combining geomorphic techniques and basic engineering relationships; and (c) mathematical modeling.

A qualitative analysis can provide insight and direction to quantifying complicated river response problems. Application of a strict quantitative analysis under these conditions results in many tables and charts summarizing the results of many calculations. Understanding and applying those results to the problem solution is difficult. A qualitative analysis indicates those variables and relationships that are actually significant to the given problem. Some key elements of the mathematical modeling of extremely complicated physical processes governing watershed and river responses are illustrated in Figure 44.

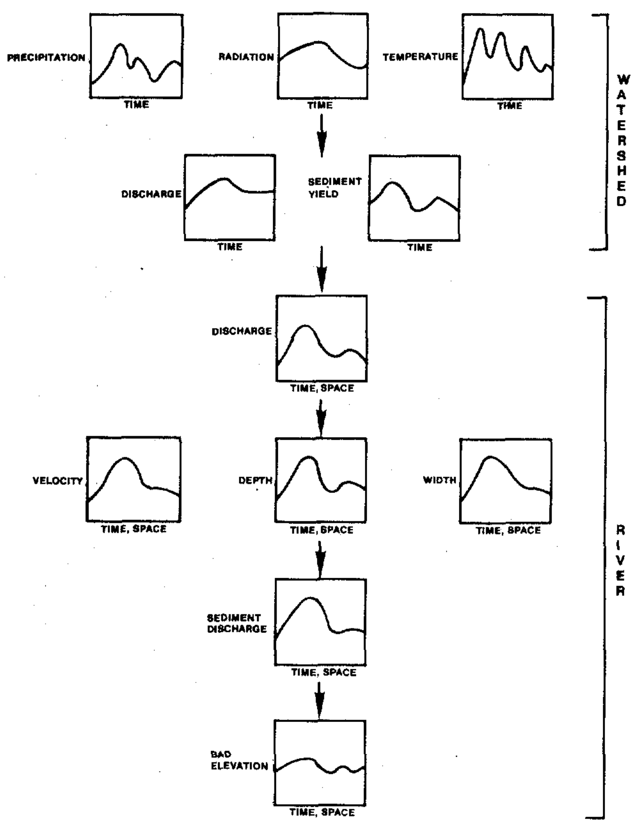


Figure 44. KEY ELEMENTS IN WATERSHED AND RIVER ANALYSIS

GEOMORPHIC TECHNIQUES

Geomorphic techniques are based on an understanding of the physical laws governing watershed and river response. Interpretation of aerial photographs also provides valuable information on river hydraulics and channel geometry problems. Sequential aerial photographs document historic trends and changes in a river. Properly applied photographic interpretation can provide an abundance of accurate and useful information.

Evidence of bank cutting, shifting of the thalweg, lateral migration/meander tendencies, vegetation changes, and sediment deposition can be documented by studying photographs for different years. Changes in stream widths also can be documented by measuring the minimum, maximum, and average stream width in each reach. Photometric techniques can be used to make measurements depending on the availability of adequate aerial photographs and viewpoint requirements.

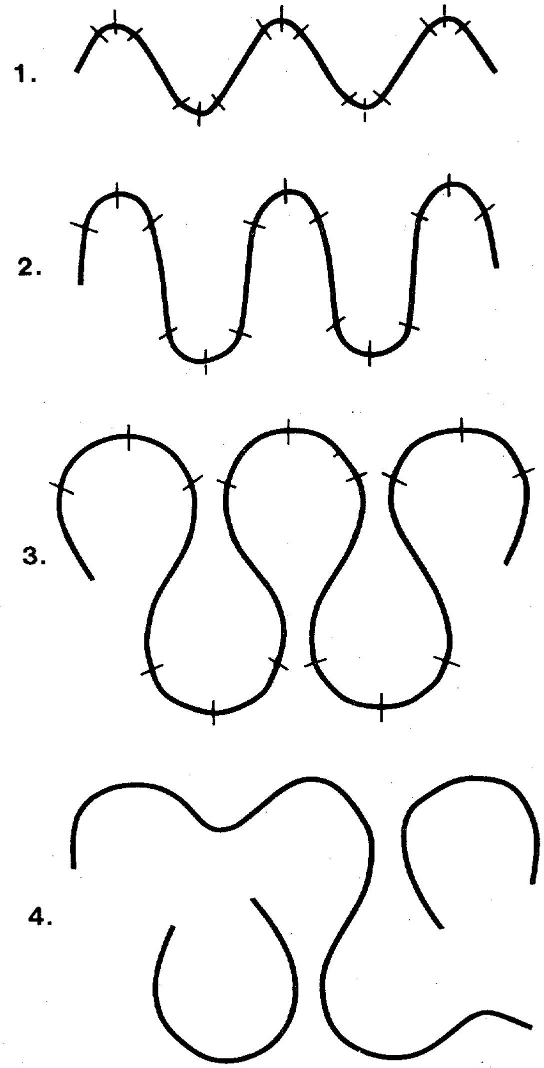
Lateral Migration

Evaluation of lateral migration tendencies is based on a knowledge of geomorphic concepts. Alluvial channels deviate from straight alignment, and the thalweg oscillates transversely and initiates the formation of bends. In general, the river engineer concerned with channel stabilization should not attempt to develop straight channels. In a straight channel, the alternate bars and the thalweg change continuously; thus, the current is not uniformly distributed through the cross section but is deflected first toward one bank and then toward the other. When the current is directed toward a bank, the bank is eroded in the area of impingement, and the current is deflected away and may impinge upon the opposite bank further downstream. The angle of deflection of the thalweg is affected by the curvature formed in the eroding bank and the lateral depth of erosion.

In general, bends are formed by the processes of erosion and deposition. Erosion without deposition to assist in bend formation would result only in escalloped banks, and the channel would widen until it became so large that the erosion would terminate. The material eroded from the bank is normally deposited over a period of time on the point bars that are formed downstream. These point bars constrict the bends and enable erosion in the bend to continue, accounting for the lateral and longitudinal migration of the meandering stream. Erosion is greatest across the channel from the point bar. As the point bars build out from the downstream sides of the points, the bends gradually migrate down the valley. The bar generally is streamlined, and its largest portion is oriented downstream. If there is very rapid caving in the bendways upstream, the sediment load may be sufficiently large to cause middle bars to form in the crossing.

As a meandering river system moves laterally and longitudinally, the meander loops move at an unequal rate because of the unequal erodibility of the banks. This inequality causes a top or bulb to form, and it is ultimately cut off. After the cutoff tip or bulb has formed, a new bend may slowly develop. Its geometry depends upon the local slope, the bank material, and the geometry of the adjacent bends. Over time, the local steep slope caused by the cutoff is distributed both upstream and downstream. It may be years before a configuration characteristic of average conditions in the river is attained.

When a cutoff occurs, an oxbow lake is formed, and it may persist for a long time before filling in. Usually the upstream end of the lake fills quickly to bank height. Overflow during floods carries fine materials into the lake. The lower end remains open and the drainage and overland flow entering the system can flow out from the lower end. The oxbow gradually fills with fine silts and clays. The fine material that ultimately fills the bendway is plastic and cohesive. As the river channel meanders, it encounters old bendways filled with cohesive materials (referred to as clay plugs) that are resistant to erosion and serve essentially as geologic controls. Clay plugs can drastically affect river geometry. The development of cutoffs and oxbow lakes is shown schematically in Figure 45.



Source: Netherlands Engineering Consultants

Figure 45. DEVELOPMENT OF NATURAL CUTOFFS

The variability of bank materials and the fact that the river encounters such features as clay plugs cause a wide variety of river forms coincident with a meandering river. The meander belt formed by a meandering river is often 15 to 20 times the channel width.

Over time, the highlands of an area are worn down. Streams erode their banks, and the material that is eroded is utilized further downstream to build banks and to further the meandering process. Streams move laterally, pushing the highlands back, and low flat valley land and floodplains are formed. As the streams transport sediment to areas of flatter slopes and in particular to bodies of water in which the velocity and turbulence are too small to sustain the transport of the material, the material is deposited, forming deltas. As deltas build outward, the up-river portion of the channel is elevated through deposition and becomes part of the floodplain, the stream channel is lengthened, and the slope is further reduced. The upstream riverbed is filled in, and average flood elevations are increased. As it moves across the river valley, this meander development causes the flood-plain elevation to rise. Hence, even old streams are far from static. Old rivers meander, are affected by changes in sea level, are influenced by movements of the Earth's crust, are changed by delta formations or glaciation, and are subject to modifications from climatological changes and human development activities.

Based on a knowledge of the geomorphic concepts and the study of aerial photographs for different years, lateral migration tendencies can be qualitatively evaluated.

Lane's Relationship

Qualitative geomorphic analysis of actual gradation changes is based on the concept of equilibrium. The qualitative approach assumes that in the long run, rivers strive to achieve a balance between the product of water flow and channel slope and the product of sediment discharge and size. The most widely known geomorphic relation embodying the equilibrium concept is known as Lane's principle.

Lane (1955) studied the changes in river morphology caused by modifications of water and sediment discharges. Similar but more comprehensive treatments of channel response to changing conditions in rivers are presented in Leopold and Maddock (1953), Schumm (1971), and Santos-Cayado and Simons (1972). All research results support the following general statements:

- depth of flow is directly proportional to water discharge and inversely proportional to sediment discharge;
- width of channel is directly proportional to water discharge and to sediment discharge;
- shape of channel expressed as a width/depth ratio is directly related to sediment discharge;
- meander wavelength is directly proportional to water discharge and to sediment discharge;
- stream channel slope is inversely proportional to water discharge and directly proportional to sediment discharge and grain size;
- stream channel sinuosity is proportional to valley slope and inversely proportional to sediment discharge;

These relationships will help to determine the response to change of any water-conveying channel.

A relationship among the above principles may be derived as follows. Sediment bed material transport, Q_s, can be directly related to stream power, $\tau_o V$, and inversely related to the fall diameter of bed material, D_{50}.

$$Q_s \sim \frac{\tau_o VW}{D_{50}/C_f} \tag{51}$$

where

τ_o is the bed shear,

V is the cross-sectional average velocity,

W is the width of the stream, and

C_f is the final material load concentration.

Equation (51) can be written as

$$Q_s \sim \frac{\gamma y_o SWV}{D_{50}/C_f} = \frac{\gamma QS}{D_{50}/C_f} \tag{52}$$

where

γ is the specific weight,

Q is water discharge, and

S is the channel slope.

If the specific weight is considered constant and the concentration of fine material can be incorporated in the fall diameter, the relationship can be expressed as

$$QS \sim Q_s D_{50}, \tag{53}$$

which is the relationship originally proposed by Lane (1955), except he used the median diameter of the bed material as defined by sieving instead of the fall diameter. The fall diameter includes the effect of temperature on the transportability of the bed material and is preferable to the use of sieve diameter.

As an example of the application of the Lane relationship, consider the interaction of a dam and reservoir with a river. Aggradation upstream of a dam results from deposition of sediment in the reservoir, which produces a relatively clear water discharge downstream of the dam. The existing downstream channel before dam closure is the product of its interaction with normal water-sediment flows over a long period of time. Hence, the clear-water discharge that comes when the dam is closed has the capability of eroding large quantities of sediment and, in fact, has a higher scouring potential than bed material-laden flow. This results in sediment entrainment from the bed and banks downstream of the dam. These changes can be predicted by considering Equation (53). The sediment discharge immediately below the dam will be less than it was previously as a result of the reservoir; in other words, Q_s is reduced to $\overline{Q_s}$. Assuming D_{50} and Q are constant, to maintain the proportionality of Equation (53), the slope S must decrease downstream of the dam. This response requires that degradation downstream of the dam be induced by the clear-water discharge. The reduction in channel slope lowers the baseline of the river downstream of the dam and can modify the downstream hydrograph. Additionally, the lowering of the baseline influences tributaries downstream by increasing the immediate water surface slope in the tributary. Applying Equation (53) to the tributary, the increase in S must be balanced by an increase in Q_s. Therefore, the increased slope induces headcutting and increased water velocities in the tributary, which lead to an increase in bank instability, increased local scour, and increased sediment delivered to the main channel.

GEOMORPHIC TECHNIQUES AND BASIC ENGINEERING RELATIONSHIPS

Geomorphic Principles

Geomorphic principles can be qualitatively applied to predict river response. This approach does not require detailed data; it requires only a general understanding of the direction of change of the river conditions. Geomorphic principles can also be applied to available data to evaluate gradation problems more accurately, but usually data records for at least several years are needed.

Detection and verification of a gradation problem by short-term observations is difficult for two reasons. First, gradation problems usually occur over long periods of time although an extreme flood event can produce rapid changes in river conditions. However, the gradation problems of concern here are aggradation-degradation changes that occur over several years. Second, the changes generally occur in the channel bottoms, which in many parts of the country are covered with water. Additionally, natural flow variations and man-controlled flow regulation frequently vary the flow depth in a river. Casual observation of a lower flow depth does not indicate a channel bottom that has degraded. The severity of the problem near a bridge is often not evident until the pilings are undermined, and even that may not be detected if the observations are made at high flows. Therefore, casual or infrequent observation is not an adequate way to detect gradation problems.

Quantitative geomorphic analysis requires collection and analysis of data over several years. It is necessary to establish the temporal changes in the water surface elevation for a given discharge, the elevation of the channel bottom, and the slope of the channel bottom or the bed material size at a given location. However, in many cases, these data may not be available. Occasionally, information on elevation changes can be gained from a series of maps prepared at different times. Data from railroad and pipeline crossing surveys upstream or downstream of a highway bridge may also be helpful in determining bed elevation as a function of time.

Analysis of gaging station stage trends provides useful information on long-term trends. On many occasions the U.S. Geological Survey (USGS) and U.S. Army Corps of Engineers have already performed the analyses. Gaging stations provide excellent records, and many stations have been in existence for 30 years or longer.

Gaging stations can often provide useful information on suspended sediment load. Although only a few stations can provide continuous sediment data, the data that are available can often provide clues to the presence of gradation problems. By definition, aggradation occurs when sediment inflow to a river reach exceeds sediment outflow. Any change in the long-term sediment load signals an imbalance in the stream system that tends to produce lateral movement, bank sloughing, and gradation problems.

Another method for verifying gradation changes is stream profile evaluation, a method that requires considerable surveying effort. The approach is similar to measuring the change in bed elevation from a bridge deck, but in this case a longitudinal profile of the thalweg is surveyed and compared to a historic profile.

Profile analysis requires considerable effort if the survey must be performed. Rough profile analysis can occasionally be performed by plotting the elevations of cross sections at pipeline crossings and railroad bridges and other similar data as a function of time. That procedure may be required when gaging station data or other data are not available. The Corps of Engineers conducts potamology surveys and maintains sediment ratings on many major streams. Data from these sources may be useful in determining bed level changes with time.

If no historic profile data are available, the analysis of a single recent profile can provide clues to aggradation/degradation problems. Sudden profile breaks or discontinuities can signal the advance of headcuts or significant aggradation problems. However, these irregularities might merely reflect changes in topography and/or geology and must be examined with aerial photographs and field visits to verify the condition. These profile irregularities can also signal the location of channel control points.

Geomorphic Relationships for Analysis of Channel Geometry Changes

The general properties of a river, such as geometry and alignment, may be expected to be changed by floods during the design lifetime of most projects. It is necessary, therefore, to establish information on the river morphology to estimate these changes. Hydraulic geometry relationships are presented in Chapter V.

The representative cross section in the vicinity of a proposed project can be developed by averaging the nearby cross sections where the channel geometries have been surveyed. From this representative cross section, channel geometry relationships can be evaluated.

The relationships between the meandering dimension parameters can also be established if sufficient data are available. These relationships include meander length to channel width, meander amplitude to channel width, and meander length to the radius of curvature. Slope relations also contain significant information about river morphology and are valuable to qualitative geomorphic analysis.

As an example of geomorphic relationships, consider the Jim River in Alaska, which is a meandering, coarse-bed stream. During high flows, it can adjust its channel goemetry in six degrees of freedom: width, depth, slope, meander wavelength, amplitude, and radius of curvature. A project was undertaken to analyze a proposed pipeline crossing for the lowest bed levels and also for the possible extremities of river course that can occur during the economic life of the project. The six-degree-of-freedom variables were treated as time dependent in the analysis.

The representative cross section of the river was established by averaging seven nearby cross sections for which survey data were available. From this representative cross section, the following channel geometric relationships were established (Figure 46):

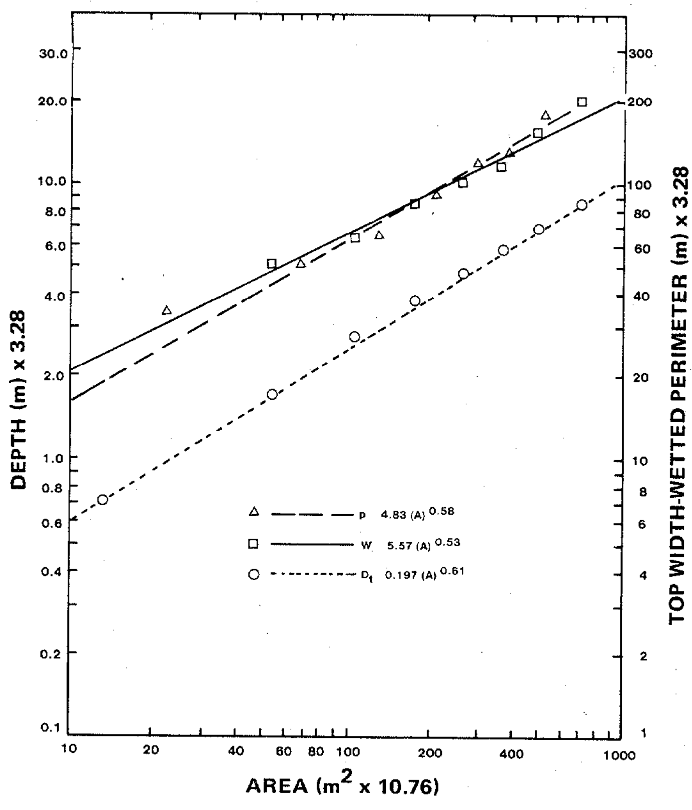


Figure 46. CHANNEL GEOMETRY RELATIONSHIPS

- the wetted perimeter, P (m), versus flow area, A (m^2):

$$P = 4.83 (A)^{0.58}$$

- the top width, W (m), versus flow area:

$$W = 5.57 (A)^{0.53}, \text{ and}$$

- the depth from thalweg level, D_t (m), versus flow area:

$$D_t = 0.197 (A)^{0.61}.$$

The analysis of 13 meander loops located in the reach 9.65 km upstream to about 32.2 km downstream of the proposed crossing shows that the average meander wavelength, L, is 750 m; the average wave amplitude, A_m, is 380 m; the average radius of curvature, r, is 185 m; and the average sinuosity is 1.70. The sharpest bend has a radius of curvature of about 60 m, and the relationships between meandering dimension parameters are estimated as follows:

- the meander length to channel width relationship is

$$L = 13.67 W;$$

- the meander amplitude to the channel width relationship is

$$A_m = 6.944 W; \text{ and}$$

- the mender length to the radius of curvature relationship is

$$L = 4.053 r.$$

By averaging meander belt widths at ten locations near the proposed crossing, the average meander belt width was found to be 890 m and the meander belt width in the immediate vicinity of the crossing to be 825 m.

The average water surface slope or the approximate energy gradient in a 1890 m reach near the proposed crossing is 0.00319. The channel bed slope for a 550 m reach immediately downstream of this site has a slope of 0.00128 (Figure 47). The average channel bed slope as taken from USGS topographical maps is 0.00419 for a 7.40 km reach near the crossing (Figure 48).

The energy slope is usually less than the channel bed slope. For a safer design, however, it is reasonable to assume the energy slope to be equal to channel bed slope. The design bed slope of 0.004 that was adopted by pipeline technologists for computing design flood is satisfactory for design purposes.

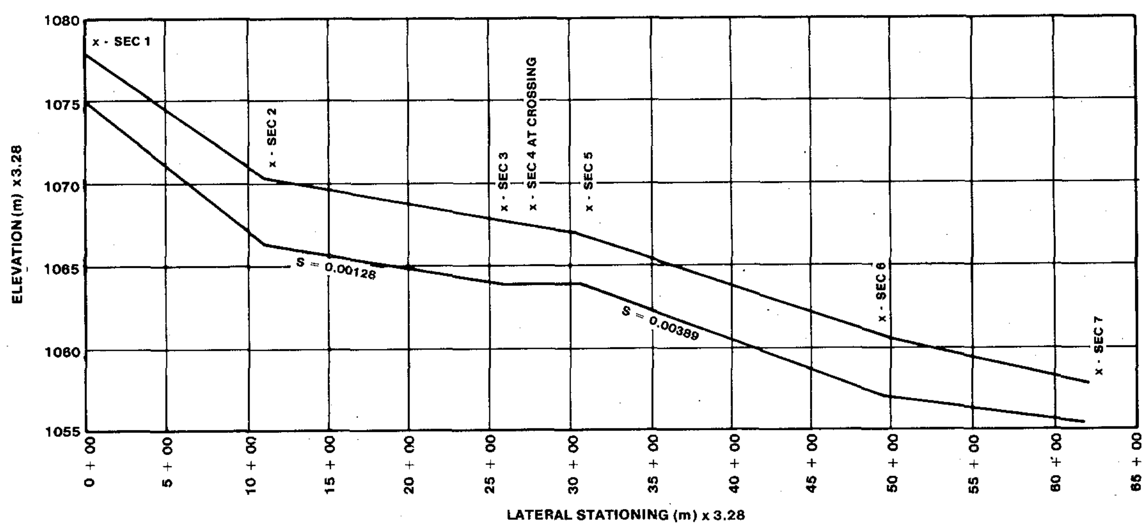


Figure 47. BED AND WATER SURFACE PROFILES

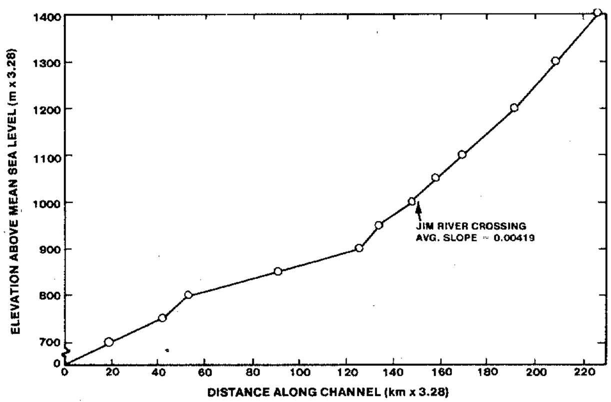


Figure 48. AVERAGE CHANNEL BED SLOPES

Incipient Motion Considerations

Incipient motion concepts are discussed in Chapter V. These considerations can be used to estimate a limiting slope to which a stream may degrade. This technique is best explained through the use of an example.

An example of Shield's relationship for incipient motion concerns construction of a bridge with a span of 152 m across an alluvial stream. A dam has been constructed 11.57 km upstream of the bridge, and its normal daily power releases are 283.2 m^3/sec from the power plant for the six high-demand hours and nominal releases for the remainder of the day to maintain fish stock. The 100-year design flood is 1133 m^3/sec with the dam in place. The natural flow of sediment in the river has been checked at the dam, and the downstream control is 14.48 km downstream where the river joins a much larger river.

The dam changes the time distribution of the flow but not the total volume. The flood peaks are reduced and the sediment transport is cutoff. The average flow has been increased from 198.2 m^2/sec to about 283.2 m^3/sec, ignoring periods when the flows are very low. Increasing the mean discharge shifts the river toward the braided stream classification, generally a destabilizing trend. The channel will probably widen, and this effect may be estimated by one of Blench's (1969) regime equations,

$$b \sim Q^{0.26}. \tag{54}$$

The new width is

$$b_n = 152.4 \left(\frac{283.2}{198.2}\right)^{0.26} = 167.6 \text{ m}. \tag{55}$$

The depth of flow at 1133 m^3/sec is computed from Manning's equation, or

$$y_o = \left(\frac{Vn}{S_f^{1/2}}\right)^{3/2}, \tag{56}$$

but since

$$V = \frac{q}{y_o} \text{ and} \tag{57}$$

$$y_o = \left(\frac{qn}{S_f^{1/2}}\right)^{3/5}, \tag{58}$$

the unit discharge is

$$q = \frac{Q}{b} = \frac{1133}{167.6} = 6.75 \text{ m/m}. \tag{59}$$

The n in Manning's equation is estimated as 0.028. The friction slope is assumed equal to the bed slope, which Richardson et al. (1974) gave as 0.00138. Then,

$$y_o = \left(\frac{6.75 \times 0.028}{0.00138^{1/2}}\right)^{3/5} = 2.65 \text{ m}. \tag{60}$$

The average velocity is

$$v = \frac{q}{y_o} = \frac{6.75}{2.65} = 2.56 \text{ m/sec, and} \tag{61}$$

the average bed stress is

$$\tau_o = \gamma y_o S_f = 1000\,(2.65)\,(.00138) = 3.66 \text{ kg/m}^2. \tag{62}$$

The Froude number for the channel flow is

$$\mathrm{Fr}_1 = \frac{2.56}{\sqrt{9.86(2.65)}} = 0.50. \tag{63}$$

The bed level is expected to degrade as a result of the cutoff of sediment by the dam. Degradation starts at the dam and progresses downstream with time, stopping only when it reaches a rock or gravel ledge or when the river enters a lake or enters into confluence with a larger river (as in this case). The river scours its bed to establish an ultimate gradient such that the shear is below the critical level for transport of sediment. This shear is not necessarily the critical shear for D_{50} because the large sizes in the bed material tend to remain to armor the bed. The D_{90} size is sometimes considered appropriate for armoring.

The critical tractive force for the D_{90} material is given by Shields (1936). Assume the flow is fully turbulent at the bed. Then,

$$\frac{V+D}{\upsilon} \geqslant 400$$

$$\frac{\tau_c}{(\gamma_s\ \gamma)D_{90}} = \frac{\tau_c}{(S_s\text{-}1)\gamma D_{90}} = 0.047, \tag{64}$$

and

$$\begin{aligned}\tau_c &= (0.047)\,(2.65\text{-}1)\,(1000)\,(.015)\\ &= 1.174\ \mathrm{kg/m^2}.\end{aligned} \tag{65}$$

The normal daily power release discharge will degrade the channel. This flow is 283.2 m^3/sec, so

$$q = \frac{Q}{W} = \frac{283.2}{167.6} = 1.69\ \mathrm{m^3/sec/m}. \tag{66}$$

Richardson et al. (1974) estimate a final n in Manning's equation for the degraded stream as

$$n = 0.028.$$

Manning's equation (given in metric units) states that

$$q = \frac{1.0}{n}\ y^{5/3}\ S_f^{\ 1/2}, \tag{67}$$

where q and n are known because

$$\tau_c = \gamma y S_f = 1.174\ \mathrm{kg/m^2}, \tag{68}$$

where

$$\gamma = 100\ \mathrm{kg/m^3}.$$

Then,

$$S_f = \frac{.00174}{1.11y} = .0016/y \tag{69}$$

Put this expression for S in Manning's equation so that

$$\begin{aligned}1.69 &= \frac{1.0}{0.028}\,(.0417)y^{7/6} \text{ or}\\ y &= 1.11\ \mathrm{m}.\end{aligned} \tag{70}$$

It follows that the limiting slope that the stream may reach is

$$S_f = \frac{1.174}{1000y} = \frac{0.00174}{y} \tag{71}$$

or ~ 1.3 m/km. The slope before degradation was known to be 2.23 m/km.

The existing profile and the ultimate profile as computed by Richardson et al. (1974) are shown in Figure 49, along with an intermediate profile during the degradation process. These profiles are ultimately controlled at the larger river, which controls the water surface level of the river at the point of confluence. Degradation at the bridge crossing can be easily estimated from the figure.

Lateral Migration

A quantitative estimate of the lateral migration tendencies can be made by analyzing the river mor-

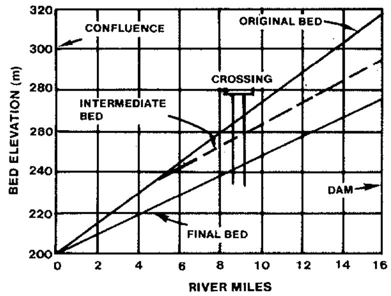


Figure 49. DEGRADATION FROM DAM UPSTREAM OF CROSSING

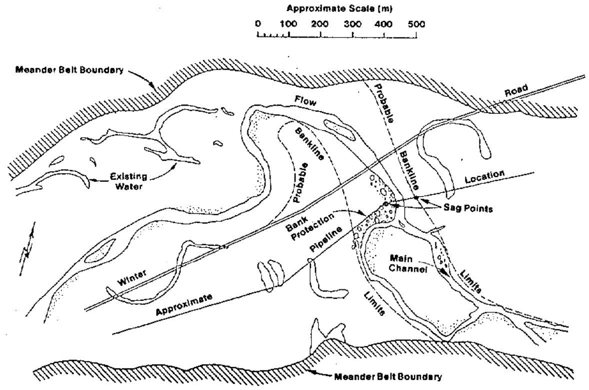


Figure 50. PROBABLE LIMITS OF JIM RIVER BANKLINES

phology relations discussed in Chapter V. The application of this procedure is best described by an example.

A meandering alluvial river has three additional degrees of freedom to adjust its geometry beyond those for a straight alluvial river, i.e., meander wavelength, wave amplitude, and radius of curvature may all change with time. The Jim River is a meandering river; therefore, these three additional degrees of freedom should be considered in estimating possible lateral channel migrations. The extent of possible lateral channel migration depends upon: (a) the meander belt width, (b) the rate and direction of lateral migration of the channel, and (c) the possible change of channel alignment brought about by the development of cutoffs upstream and downstream of the site.

The probable maximum meander wavelength, wave amplitude, and radius of curvature during the project's lifetime are estimated by the relationships between meander dimension parameters. The input is the probable maximum bankfull width, which is determined for the design flood. The estimated extremities of these three quantities are as follows: the meander wavelength is 1290 m, the wave amplitude is 655 m, and the radius of curvature is 320 m. The sinuosity estimated from the above quantities is 1.5, which is less than the current average value of 1.7. This indicates a current tendency to straighten the channel reach.

The average meander belt width near the crossing site is 890 m and has a value of 825 m in the immediate vicinity of the crossing (Figure 50). From the figure it can be seen that the proposed crossing is near the center of the meander belt. The estimated probable maximum wave amplitude during high flows is 655 m and is less than the local meander belt width. Hence, the probability of the river's moving the entire meander belt distance during the project's lifetime is very small.

The channel may migrate in a direction along the axis of the meander belt. This is shown in the studies of the crossings by James Brice. A change of 38 m has occurred in the direction the pipeline over a 14-year period. Continued migration in this direction at a similar rate would definitely jeopardize the pipeline during its economic lifetime. The meander wavelength of the loop in the crossing vicinity before the development of the cutoff is about 915 m, and the probable maximum wavelength as deduced in this study is 1290 m. This shows the possibility for channel migration in the direction along the axis of the meander belt, but the development of a cutoff upstream of the site in the past 10 years has largely reduced this possibility. The 1973 survey shows that the flow has concentrated in this cutoff path (Figure 50). However, for a safer and more economical design, riverbank protection works are recommended for the left bank (looking downstream) near the crossing to prevent possible migration (Figure 50).

Judging from the information on the extremities of meander size parameters and available aerial photos, the probable limits of bank lines during the lifetime of the project have been estimated. They are shown as dashed lines in Figure 50. Sag points would require a deep-burial distance of 200 m for the pipe.

With bank protection or with close surveillance so bank protection could be installed in the event channel migration endangers the crossing, sag points can be located 46 m to the left of the left bank and 30 m to the right of the right bank. The total deep-burial distance would be about 110 m. The right bank appears stable and probably will not require bank protection during the life of the pipeline. The left bank upstream of the crossing is unstable and probably will need bank protection. Plugs in the left channel upstream of the crossing would probably eliminate the need for bank protection works on the left side. These plugs would also improve the flow of the river through the crossing.

Slope/Discharge Relationship

The slope-discharge, SQ, relationship presented in Figure 51 can be used to analyze gradation problems when these parameters are known. The figure illustrates the dependence of river form on the channel slope and discharge and the possible dramatic change in river form if the slope/discharge relationship borders one of the transitional regions.

The significance of Figure 51 to the gradation problem is that it shows that vertical instability may result in lateral instability. That is, a dam on a stable meandering stream might cause sufficient aggradation upstream to cause braiding. Artificially constraining the channel to a meandering or straight course may increase the aggradation rate. Many other effects can be analyzed with Figure 51.

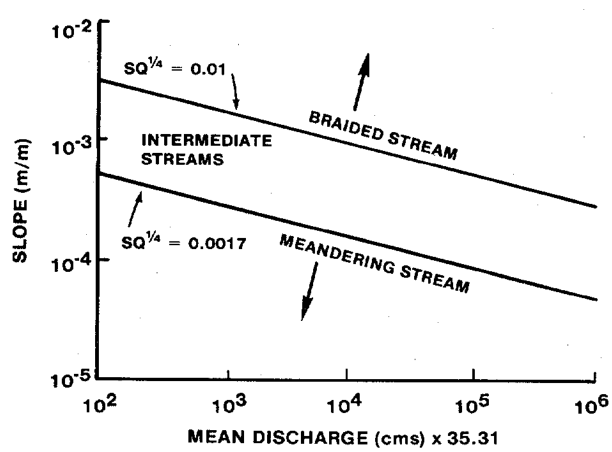


Figure 51. SLOPE-DISCHARGE RELATIONSHIP FOR BRAIDING OR MEANDERING IN SAND BED STREAMS

Analysis of Potential Bed Material Volume Change

A quantitative estimate of the amount of aggradation or degradation to be expected in the vicinity of a highway crossing can be estiamted by analyzing the potential bed-material volume change in the local reach. This technique involves analysis of the transport capabilities of upstream reaches as well as transport within the local reach of interest. This procedure can be subdivided into two steps; first, the required input data are collected and analyzed, and second, the bed/material volume change is computed using the necessary conditions of continuity.

Typical input requirements for this technique include average channel geometry, reach lengths and slopes, estimates of channel roughness, bed material size gradations for reaches of interest, upland sediment yields (if found to be important), flow-duration and flow-frequency curves, and an approximate voids ratio for the bed material. Depending on local characteristics and the transport equation used, these input data could vary. Also, the required precision of the input data will depend on the desired accuracy of the results. For example, channel geometry could be obtained in several ways. It could be estimated from site inspections, measured using modern surveying techniques at one location per reach or surveyed at several locations within a reach and then the resulting sections averaged to produce the required average channel section within each reach. Similarly, various levels of analysis could be used to obtain other required information.

The second step of this analysis consists of applying, sediment continuity to the reach of interest. To maintain continuity, the following equation must be satisfied:

$$\frac{\Delta V}{\Delta t} = \frac{I_V - O_V}{\Delta t} \tag{72}$$

where

I_V = volume of sediment entering the reach,

O_V = volume of sediment leaving the reach,

Δt = time increment under consideration, and

Δt = sediment volume change within the reach.

The sediment continuity equation is applied with the aid of an appropriate sediment transport equation and an annual flow duration curve. The choice of an appropriate sediment transport equation should be based on characteristics of the bed material in the reach of interest as discussed in Chapter V. The annual flow duration curve should be constructed from streamflow records or from other approximate methods if the gage data do not exist.

Applying the sediment continuity equation as presented above requires the following steps:

- Divide the annual flow duration curve into incremental time steps. These time steps do not need to be the same length, but they should bracket some characteristic discharge for the period.
- With the channel geometry and bed material size gradation as input, compute the transported volume within each reach (using an appropriate sediment transport equation) for each time interval.
- Apply the sediment continuity equation at each time step to compute the volume of material eroded or deposited during each time step.
- Sum all the ΔVs to find the total volume of material deposited or eroded during one year.
- Estimate a depth of deposition/erosion based on channel geometry and typical bed material voids ratio.

The procedure outlined above is based on an average annual flow duration curve that gives an estimate of annual bed level changes at the site. However, in many cases the average flow conditions do not cause the major problem; rather, it is caused by large storm events. Therefore, this procedure should also be applied to the design storm for a given structure. For this case, the application of the sediment continuity equation would be the same as outlined above except that a design storm hydrograph would be analyzed instead of the average annual flow duration curve. The result would provide an estimate of the potential volume of material deposited or scoured during the design-storm event.

The above technique provides an estimate of the magntiude of a gradation problem at a local site on an annual basis and for a single storm event. It is not intended to give a "final" estimate of the base level of the channel at the site of interest. Such a final estimate would require repeated application of the procedure over several years with adjustments in channel slope and other hydraulic and geometric parameters at each step. This type analysis would be well suited for use with a digital computer and will be discussed later.

The technique described for estimating the potential bed-material volume change within a local reach provides a relatively quick method of estimating the magnitude of an aggradation/degradation problem. It is useful for establishing the size of a required maintenance program or for determining relative importance of single storm events and annual flow hydrographs to the grade change problem.

Engineering Relationships Developed for the Analysis of Specific Aggradation/Degradation Problems

In addition to the more-general techniques presented in the preceding sections, the literature contains a large number of prediction techniques based on assumptions that make them applicable only to specific cases. Among these techniques are procedures for estimating degradation downstream of dams, aggradation upstream of dams, and aggradation in streams from overloading. The analysis of case histories in Chapter I and II shows that these types of problems are common.

Degradation Downstream of Dams

The subject treated in the literature most often was degradation downstream of dams. Mostafa (1957) has presented a method of estimating an equilibrium profile or slope for a river undergoing degradation downstream of a dam. By combining a flow equation presented by Einstein (1942) and the critical tractive force equation from Shields (1936), Mostafa (1957) developed the following equation for the equilibrium slope:

$$S = (\gamma_s') \, 0.06 \, (k_s/Y)/\gamma R \qquad (73)$$

where

γ_s' = the submerged weight of sediment,
k_s = characteristic particle size,
Y = $\dfrac{0.06\,\gamma_s'\,k_s}{\tau_c}$,
τ_c = critical shear stress,
R = hydraulic radius, and
γ = specific weight of water.

The following simplifying assumptions are made in deriving this equation:

- the discharge is constant;
- the minimum tractive force required to move the bed material is the same as the maximum for which deposition occurs;
- Shield's (1936) criteria for incipient motion apply;
- channel cross sections are uniform and regular geometric sections;
- the bed material is composed of sand and small gravel size particles;
- the channel section and bed material properties are constant throughout; and
- the downstream limit of the reach is at some downstream control.

The following steps are recommended for the application of this technique:

Step 1: Assume X - Y = 1, where X = a correction factor proposed by Einstein (1942) (Figure 52).

Step 2: Estimate $k_s = d_{90}$.

Step 3: Compute R's for various Q's from

$$\frac{Q}{A} = 5.75\sqrt{\frac{0.06\,\gamma_s'\,k_s/Y}{\gamma}}\;\log\left(12.27\frac{R}{k_s}X\right). \tag{74}$$

Step 4: Compute corresponding S from R's using

$$S = [(\gamma_s\text{-}\gamma)0.06\,K_s/Y]\gamma R \tag{75}$$

Step 5: Verify X and Y using Figure 52 and

$$\frac{k_s}{\delta} = \frac{k_s\sqrt{gRS}}{\nu\,11.6}\;. \tag{76}$$

Step 6: If X and Y are not 1, assume a new X and Y until the final slope is reached.

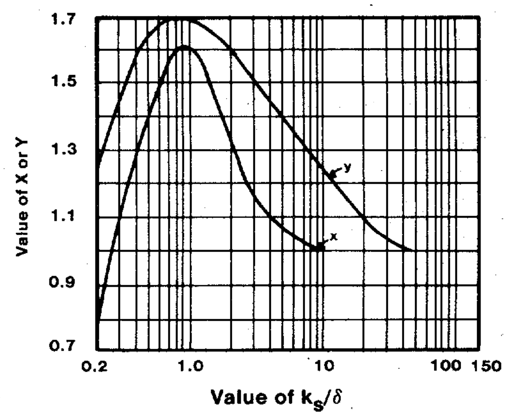


Figure 52. RELATIONSHIP BETWEEN X OR Y AND k_s/δ

Mostafa's (1957) equlibrium slope equation is recommended for use under conditions representative of the assumptions made in its derivation. Although many simplifying assumptions were made in its derivation, this relationship provides a quick method for evaluating the final equlibrium profile downstream of a dam or other structure that traps the normal sediment load.

A more detailed relationship is presented in Komura and Simons (1967). Using a sediment transport equation presented by Kalinske and Brown [Rouse (1949)] and the continuity equation, an equation describing bed degradation downstream of a structure that traps the normal sediment load was developed as follows:

$$Z_f = Z_o + \frac{14}{15}\left(\frac{C_s}{c}\right)\frac{D_{sfo}}{Y_{fo}}\left(e^{15C/14x'}-1\right) + Y_{fo}\left(1\text{-}e^{-c/14x'}\right) - \frac{Y_{fo}}{2}\left(\frac{Y_o}{Y_{fo}}\right)^3\left(e^{c/7x'}-1\right) \tag{77}$$

where

Z_f	=	final river profile,
Z_o	=	initial river profile,
C_s	=	an armoring coefficient,
c	=	a constant,
D_{sfo}	=	final representative particle size at origin,
Y_{fo}	=	depth at origin,
x'	=	distance measured upstream, and
Y_o	=	critical depth.

A definition sketch is given in Figure 53. The use of the Kalinske and Brown transport equation in deriving this equation limits this equation's use to streams having beds of sand and/or small gravel. Bed armoring was considered in the development of the equation. The relationship presented will give a final equilibrium profile but no information on the rate of degradation.

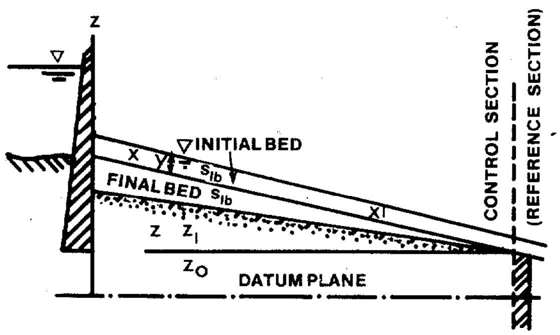


Figure 53. DEFINITION SKETCH

A tabular step method was presented in Komura and Simons (1967) for the solution of their equation. The application of the technique is demonstrated below with the aid of an example first presented by Komura and Simons (1967).

Degradation below the Milburn Diversion Dam on the Middle Loop River in Nebraska will be computed numerically using the data obtained by Miller (1953) and the U.S. Bureau of Reclamation (USBR) (1963). In the numerical example it is assumed that

- the sediment transport is completely arrested by the dam,
- the river banks are not erodible,
- seasonal variations in discharge and temperature of the water do not occur,
- sediment injections by tributaries do not occur, and
- meandering and growth of vegetation do not occur.

The relationships between the sediment sizes and the distance from a reference section are shown in Figure 54. The reference section is taken at the Lower Sediment Range No. 3, which was installed by the USBR. The distribution of final equilibrium sediment size is shown in Figure 54. Data for sediments in the river bed in the Lower Sediment Ranges No. 1, 2, and 3 are given in Table 15. The values of constant a_c in Table 14 were obtained from Figure 55. The value of C_s is 0.413, because $a_c = a_c\,\sigma_\phi - 0.25$, where σ_ϕ = the standard deviation of the particle size distribution and $\delta\rho = 2.65$ (assumed).

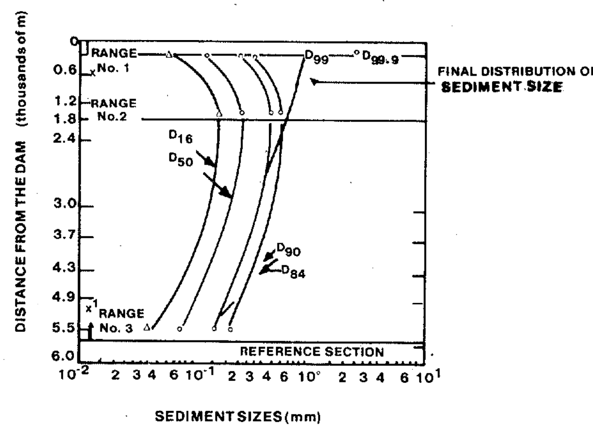


Figure 54. SEDIMENT SIZE DISTRIBUTIONS

Taking $Z_o = 0$, a final equilibrium profile is given by

$$Z_{fn} = \sum_{n=0}^{7} \left\{ 0.413 \left(\frac{d_{sf}}{Y_f}\right)_n \Delta x'_n + \frac{1}{14}\left[1-\left(\frac{y_c}{y_f}\right)_n^3\right]\left(\frac{y_f}{d_{sf}}\right)_n \Delta d_{sfn} + \frac{1}{7}\left(\frac{y_f}{b}\right)_n \left[6+\left(\frac{y_c}{y_f}\right)_n^3\right]\Delta b_n \right\} \qquad (78)$$

Table 15. SEDIMENT PROPERTIES OF RIVER BED

RANGE NO.	DISTANCE MEASURED UPSTREAM x' (ft)	SEDIMENT SIZE				σ_ϕ	a_c
		D_{16} (mm)	D_{50} (mm)	D_{84} (mm)	D_{90} (mm)		
1	17,850	0.052	0.115	0.25	0.30	4.8	0.036
2	14,000	0.150	0.240	0.43	0.52	2.9	0.050
3	0	0.036	0.061	0.13	0.17	3.6	0.110
Mean			0.139			3.8	0.065

Source: Komura and Simons (1967)

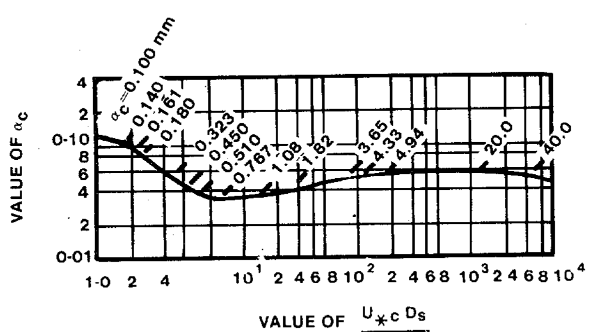


Figure 55. VARIATION OF a_c WITH $U_* cD_s/\nu$

where

d_{sf} = mean diameter of bed material at final equilibrium state,

Δd_{sfn} = final difference between the mean diameters of bed material at the two end sections of $\Delta x_n'$,

b = width of river,

Δb_n = difference between the widths at the two end sections of $\Delta x_n'$,

y_c = critical depth of flow,

y_f = final depth of flow,

$\Delta x_n'$ = distance of reach divided,

Z_{fn} = final river-bed elevation at a reach measured from a datum plane,

ΔZ_{fn} = final difference between the river-bed elevations at two end sections of $\Delta x_n'$, and

subscript n = mean value in a divided reach, $\Delta x_n'$.

Equation (78) is a modified version of Equation (77). The transformation puts the equation in a form that allows easy step solution. The step solution for this example is outlined in Table 16. The final equilibrium profile obtained is shown in Figure 56. The discharge for this example was 22 cms. For this example, the widths of various reaches were not the same.

Other techniques for estimating degradation downstream of dams have been presented in Ashida and Michiue (1971) and Aksoy (1971). Their techniques were more detailed in their application and more restrictive in use than the techniques presented above. Therefore, they are not recommended for general use.

Aggradation Upstream of Dams

Several authors have presented techniques to evaluate aggradation upstream of dams. These include Harvey et al. (1971), Bruk and Milorodov (1971), and Garde and Swamee (1943). These techniques are all similar to the technique presented subsequently; the major difference is the assumption here that the entire sediment load is trapped in the reservoir. Therefore, these techniques are not discussed here.

Table 16. COMPUTATION BY THE STEP METHOD

η (1)	x' in feet (2)	$\Delta x'_{\eta}$ in feet (3)	B, in feet (4)	$\Delta B_{\eta'}$ in feet (5)	$D_{sf'}$ in feet (6)	$\Delta D_{sf\eta'}$ in feet (7)	(yf) m in feet (8)
7	18,600	750	300	0	0.00328	0.00020	0.82
6	17,850	1,750	300	-50	0.00308	0.00059	0.81
5	16,100	2,100	350	-50	0.00249	0.00048	0.76
4	14,000	2,900	400	50	0.00201	0.00057	0.83
3	11,100	2,500	350	50	0.00144	0.00036	1.02
2	8,600	2,500	300	-50	0.00108	0.000258	1.11
1	6,100	6,100	350	0	0.000822	0.000396	1.20
0	00	0	350	0	0.000426	0.000000	1.28

η (1)	(y_c^3) m, in cubic feet (9)	$\left(\frac{y_c^3}{y_f^3}\right)_m$ (10)	$\left(\frac{yf}{B}\right)$ m (11)	$\left(\frac{yf}{D_{sf}}\right)$ m (12)	$YZf_{\eta'}$ in feet (13)	$\Delta Zf_{\eta'}$ in feet (14)
7	0.231	0.418	0.00273	258	1.20	12.26
6	0.198	0.373	0.00249	290	2.40	11.06
5	0.148	0.337	0.00203	338	2.49	8.66
4	0.148	0.258	0.00221	480	2.61	6.17
3	0.198	0.187	0.00314	808	1.44	3.56
2	0.198	0.144	0.00341	1,164	0.76	2.12
1	0.170	0.098	0.00343	1,925	1.36	1.36
0	0.170	0.081	0.00365	3,010	0.00	0.00

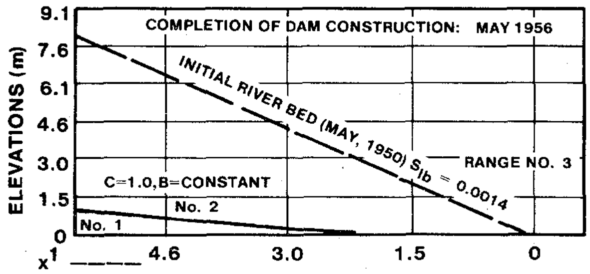


Figure 56. FINAL EQUILIBRIUM PROFILES

Aggradation in Streams from Overloading

A procedure for estimating aggradation in streams from overloading has recently been proposed by Soni et al. (1980). During the initial derivation of this procedure it was assumed that

- Z_o (depth of aggradation at origin) is independent of time;
- a sediment transport law of the form $Q_s = a\,U^b$ is used for total sediment load;
- a suspended load is constant in time;
- the sediment transport equation obtained from uniform flow considerations is used in an unsteady flow situation;
- the channel is of uniform slope and cross-section;
- deposition comes only from bed load; and
- the flow is uniform at all times.

Considering these assumptions, an analytical approach was taken to the problem of estimating aggradation from overloading. The analytical solution was later calibrated to match experimental results through the use of an aggradation coefficient. This coefficient has only been calibrated for uniform fine to medium sand bed channels, and for that reason, the solution technique should only be used on that type stream. Data requirements include (a) an estimate of Z_o (the depth of deposition at the sediment injection section after some known time), (b) hydraulic properties (flow roughness, channel sections, etc.) throughout the reach of interest, (c) an estimate of the influent sediment transport rate, and (d) an estimate of bed porosity.

The procedure to be outlined was designed to predict a final channel profile downstream of the overload point. However, after the final profile is estimated, intermediate profiles can be computed. Again, the assumption is made that the entire reach under study has a uniform cross section and slope. The procedure is as follows:

Step 1: Using surveyed cross sections within the reach, construct an average channel section and compute its hydraulic properties relating to sediment transport and backwater conditions.

Step 2: Compute, measure, or in some way estimate the overload bed transport rate (ΔQ_s).

Step 3: From known Z_o at a given time, t, and ΔQ_s, compute K from

$$\frac{Z_o}{\sqrt{Kt}} = 0.885\,\frac{(\Delta Q_s)}{K(1-\lambda)}\text{, or} \tag{79}$$

$$K = \frac{0.783\,(\Delta Q_s)^2 t}{(1-\lambda)^2\,Z_o^{\,2}} \tag{80}$$

where

K = the aggradation coefficient and

λ = the porosity of the sediment.

Step 4: Aggradation will occur to some water surface control such as a reservoir, lake, or another river. Measure or compute this length.

Step 5: Compute the time required for aggradation to reach the downstream control using

$$t = \frac{L^2}{13.40\,K} \tag{81}$$

Step 6: Compute Z_o from

$$\frac{Z_o}{\sqrt{Kt}} = 0.885\,\frac{\Delta Q_s}{K\,(1-\lambda)} \tag{82}$$

Step 7: Compute the final longitudinal profile from

$$\frac{Z}{Z_o} = 1 - \frac{2}{\sqrt{\pi}} \int_u^n e^{-u^2} du \qquad (83)$$

where

$$n = \frac{x}{2\sqrt{Kt}}$$

This relationship is represented graphically in Figure 57 as Z/Z_o vs. $x/2\sqrt{Kt}$

Application of the procedure in Soni, et al. (1980) in this manner gives the final bed profile and a time to equilibrium for some length of channel. An alternative approach is to select a time span of interest and compute the respective profile.

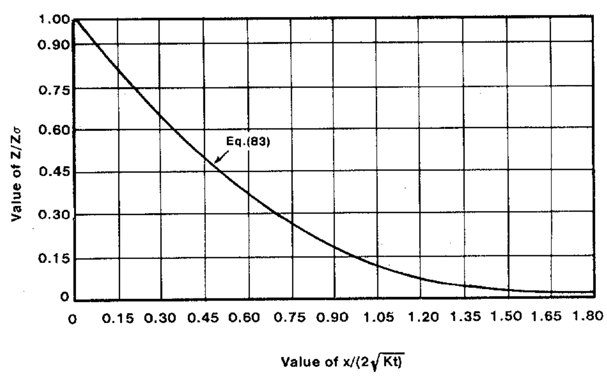


Figure 57. DIMENSIONLESS PLOT OF TRANSIENT BED PROFILES

Summary

The techniques presented for the analysis of specific aggradation and degradation problems are extremely limited in scope. Each technique was developed for use under a given set of conditions. Care must be taken when applying these techniques that the specific reach of river under consideration meets the criteria under which the technique was developed. When these criteria are matched by field conditions, these engineering relationships provide a method of estimating the magnitude of aggradation and degradation problems.

MATHEMATICAL MODELING

Many processes and factors govern watershed and river responses (Figure 44). The interaction of the various processes and the effects of other influencing factors make the complete modeling of river systems virtually impossible. However, under certain assumptions, models have been developed that reproduce the behavior of rivers and river systems very well. These models range from methods employing hand calculation procedures aided by small computers or desk-top calculators to full-blown dynamic computer models capable of unsteady flow routing as well as sediment routing.

The basic computational process in all sediment transport models is essentially the same. This process involves the repetition of five basic steps outlined by Gessler (1971) and listed below:

Step 1: Set up the model.

Step 2: Evaluate the water surface and energy grade line profiles for current geometric conditions.

Step 3: Estimate sediment transport rates within each reach.

Step 4: Apply the continuity equation for sediment transport over segments of the channel to determine depth of aggradation or degradation.

Step 5: Compute the new bed profile.

After Step 5 is completed the procedure is recycled to Step 2 and the steps are repeated until an equilibrium profile is reached. The procedure is best implemented with the use of a digital computer to perform the necessary iterations. However, with enough simplifying assumptions, this procedure can be applied manually with the aid of a small, desk-top, programmable calculator. The numerical process described in the latter four steps is straightforward, but it requires a tremendous amount of numerical computation. The process would be extremely tedious without the aid of a programmable calculator or computer.

The evaluation of this process is presented on three levels. First, a hand computational technique is presented; then, a technique employing hand computation of sediment routing combined with a computer model for flow routing is discussed; and finally, complex computer models employing both sediment and flow routing are documented. These latter models include both quasi-dynamic models and fully dynamic ones.

Hand/Desk-Top Calculator Method

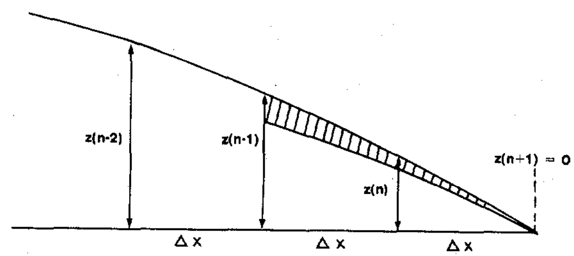
The model to be developed for applying the hand/desk-top calculator method of solution will be set up in a fashion similar to the other levels of mathematical modeling that are discussed later. The primary difference between the various modeling technique is in the detail and sophistication of the model.

Model Setup

The first step in developing a model is to locate the channel controls with respect to the aggradation or degradation problem under consideration to define the length of river that will be affected. Channel controls are discussed in Chapter V. With the affected reach of river defined, field surveys should be conducted to define the geometric and hydraulic properties characteristic of the reach. These properties include channel geometry, bed profiles, channel roughness, and bed material characteristics. Variances in these properties should also be documented.

The reach of interest is then divided into n subsections. The criteria for selecting the subsections within the reach of interest are uniformity among the hydraulic and geometric properties. The number of subsections or subreaches used will depend on conditions within the given river reach and the level of accuracy required of the results. Theoretically, the value of n could range from 1 into the hundreds; however, because of the tremendous volume of numerical computations and the analysis time that would be required, it is recommended that the number of subreaches be limited to five or fewer. If, for accuracy reasons, more than five subreaches are selected one of the more sophisticated modeling techniques should be used.

The next step is to obtain average hydraulic and geometric properties for the subreaches. Based on field observations and surveyed cross sections, average channel geometry for each reach can be estimated. Bed samples taken during field visits provide the required sediment size gradation and other bed material properties within each reach. Field observations should also provide a base for estimating average channel roughness within each reach. The channel profile is then given in tems of n + 1 bed elevations as documented in Figure 58. The Δx's shown in Figure 58 do not have to be equal, but for best result, they should not vary by more than 35 percent.



Source: Gessler (1971)

Figure 58. DIVISION OF REACH INTO FINITE ELEMENTS

It is also necessary to establish the boundary conditions for the model. Sediment transport loads entering the model at the upstream control and any other influent locations (at tributaries or other point load sources) must be estimated. This estimate would include the computation of sediment yields from tributary watersheds as discussed in Chapter V. Sediment loads entering the model at the upstream end can be computed using an appropriate transport equation assuming that the volume of material transported into the reach of interest is equal to the capacity of the upstream reaches to transport the material. If some other condition is known to exist, the transport volume must be estimated by some other method. Similarly, the downstream sediment control must be evaluated.

In setting up the aggradation/degradation model, special care must be taken because during the first time interval, degradation can only occur in the most upstream section. If the time interval selected is too long, the slope in this section will drop below the final equilibrium slope. (This final equilibrium slope is defined by incipient motion considerations as documented in Chapter V.) The interval should be determined such that less than 2/3 of the material is eroded (in the case of degradation) or deposited (in the case of aggradation) between the profile at the beginning of the time interval and the hypothetical new profile from incipient motion considerations in first section.

The time interval (Δt) selected does not have to remain the same for all steps; in fact, it is best to increase the Δt with each computaional step because degradation or aggradation processes proceed most rapidly in the period of time immediately following the initiation of the grade change. The value of Δt used will depend on the type and cause of the gradation problem as well as the hydraulic and geometric characteristics of a given river system. The initial time interval is based on trial estimates of the initial grade change rate. Subsequent changes in Δt at each step of the process are based on the actual rate of change documented in the preceding step. Consideration must also be given to the time steps in the discharge hydrograph to be used for the analysis.

New Profile Computation

With the model set up, the repetative steps in computing the new profile can begin.

The first step is to compute a water surface profile along the reach. For the level of analysis being discussed here, the backwater profiles can be estimated using normal depth considerations or calculated using step-backwater techniques. The backwater profiles established in this manner provide the required hydraulic properties within each reach for use in sediment transport equations. These properties include flow depth, flow velocities, and energy gradeline slopes.

The second step involved in computing the new profile is to evaluate the sediment transport along the channel within each subreach. The choice of an appropriate transport equation should be based on information presented in Chapter V.

With the sediment transport rates established for each subreach, the continuity equation for sediment load can be applied over segments of the channel for the given time interval in order to find the amount of aggradation. The sediment continuity equation is

$$\frac{\partial g_s}{\partial x} = -C_3 \frac{\partial z}{\partial t} \tag{84}$$

where

g_s	=	sediment transport rate per unit width,
x	=	channel distance,
Z	=	elevation,
t	=	time, and
C_3	=	a constant converting eroded volume to eroded weight.

Equation (84) is applied for each section starting with the section between cross sections (n + 1) and (n) in Figure 58. In the figures, the hatched area represents the volume to be eroded, which is

$$1/2 \cdot \Delta Z\,(n) \cdot \Delta x$$

which, according to Equation (84) equals the difference in sediment load at (n + 1) and n times the time interval Δt and times a coefficient converting weight into corresponding volume. The resulting equation

$$-1/2\ \Delta Z\,(n) \cdot \Delta X = g_s\,[(n+1) - g_s\,(n)] \cdot \Delta t \cdot C_3 \tag{85}$$

is solved for Z (n) (which in the case of degradation will be negative and in the case of aggradation will be positive) producing

$$\Delta Z\,(n) = -2 \cdot \frac{g_s\,(n+1) - g_s\,(n)}{\Delta X} \cdot \Delta t \cdot C_3 \cdot \tag{86}$$

A similar procedure is then applied to the next section, yielding

$$\Delta Z\,(n-1) = \Delta Z\,(n)\ -2\frac{g_s\,(n) - g_s\,(n-1)}{\Delta X} \tag{87}$$

and so on until the ΔZ's of all cross sections are determined. The geometry of all cross sections is then adjusted to reflect the degradation or aggradation.

When adjusting degraded sections, bank stability must be checked. If the degradation produces a bank angle steeper than the angle of repose of bank material, the bank will slump to the angle of repose or some lower angle. The volume of material eroded this way must be included in the channel geometry adjustment. A new profile is then determined by adding ΔZ to the previous Z.

The procedure just described is then repeated until the limits of the final equilibrium profile (usually 95 percent) are reached. This final equilibrium profile can be evaluated separately, using the incipient motion criterion.

Several parts of the above procedure could easily be programmed on small desk-top calculators or computers. They include the sediment transport equation, the hydraulic properties of cross section, normal depth or backwater computation techniques, and the sediment continuity equation. Small programs for each of these procedures would greatly reduce the number of individual computations required and would, therefore, reduce the time required to apply the technique.

Computer-Aided Flow Modeling with Manual or Computer-Aided Computation of Sediment Routing

The second level of sophistication in the modeling of aggradation and degradation processes involves computer-aided flow modeling with manual or computer-aided computations of the accompanying sediment movement. The foundation for this technique is the same as that for the technique presented for the hand/desk-top calculator method in the preceding section. However, in this case, a more detailed analysis of channel characteristics is involved and fewer simplifying assumptions are made. Although the title of this technique implies manual computation of sediment routing, the sediment transport mechanism could be computerized as was recommended in the preceding section.

The additon of computerized water routing to the model described in the previous section reduces the number of individual computations required and makes the approach easier to manage on more complex hydrologic systems. This technique is suited for use in situations in which

- very long reaches of river must be analyzed,
- a large number of subreaches must be analyzed,
- numerous flow obstructions exist such as encroachments or bridges, and
- any combinations of the above conditions exist.

The procedural steps for this technique are the same as those given in the section on "Hand/Desk-Top Calculator Method." Descriptions of the more detailed water routing and sediment calculations are presented in the following paragraphs.

Water Routing

One of the most often applied one-dimensional channel water-routing techniques is the U.S. Army Corps of Engineers HEC-2 computer model. This program computes the water surface profile under subcritical or supercritical conditions for rivers of any cross section. The program incorporates the effects of hydraulic structures (bridges, culverts, weirs, embankments, and dams) and varying Manning's "n" values. It is generally used to evaluate profiles for various frequency floods under natural or modified conditions.

The theory of the HEC-2 model is based on Bernoulli's theorem for the total energy at each cross section and Manning's formula for the friction head loss between cross sections. The average friction slope for a reach between two cross sections is determined from the average of the conveyances at the ends of the reach. Other losses (bridge crossings, etc.) are computed by other methods. The critical water surface elevation corresponding to minimum specific energy is computed using an iterative process.

The HEC-2 model can be applied to a gaged or an ungaged river system. In a gaged river system the inflows from tributaries and other rivers are known and used as input to HEC-2. For an ungaged river system, the inflows from tributaries and other rivers can be scaled from the discharge in the main river according to drainage basin area. According to Leopold et al. (1964), the flood or bankfull discharge can be expressed as a power function of drainage area, and the exponent of the equation varies from 0.65 to 0.8. For mean annual flow in humid regions, the exponent is about 1.0. This technique provides reasonable results when actual data are not available.

A more-sophisticated technique for estimating inflow rates in an ungaged river system requires evaluating the watershed flows. Three simplified techniques to accomplish this are the rational method [Schwab et al. (1966)], the Soil Conservation Service (SCS) method [Mockus (1957)], and the unit hydrograph method [Sherman (1952)]. Computerized methods have also been developed by the SCS and the Corps of Engineers to estimate design hydrographs. These methods combine the theoretical approaches and allow more complicated problems to be considered.

Sediment Computations

After evaluating the hydraulic conditions of the river, the sediment transporting capacity can be established. Sediment transport equations are used to determine the sediment transport capacity for a specific set of flow conditions. Different transport capacities can be expected for different sediment sizes. For each sediment size, the transport rate includes the bed load and the suspended load transport rates.

A method that uses hydraulic conditions from HEC-2 or an equivalent program and computes sediment transport based on the Meyer-Peter and Muller bed load equation and the Einstein suspended sediment transport equation was developed by Simons and Senturk (1977). This procedure computes the transport of bed material load by sediment size fractions. The data requirements are the same as those for a HEC-2 backwater analysis insofar as channel geometry and bed resistance are concerned. Additional data requirements include bed material size distributions and upstream sediment supply rates. Using the generated hydraulic conditions, the transport capacity for each sediment size at each cross section can be determined.

Actual transport rates depend on transport capacity as well as supply rate. The change in transport capacity between two cross sections can be used to estimate aggradation or degradation based on availability. For example, if sediment is in ample supply to meet the transport capacity at an upstream cross section and at the next cross section downstream, the transport capacity is only one-half as much and the other half of the sediment passing the upstream cross section must be deposited between the upper and lower cross sections. This comparison of transport capacities continues reach by reach and size fraction by size fraction through the entire stream segment. The disadvantage of this simplified approach is that the hydraulic conditions are not readjusted for aggradation or degradation. However, at least some idea of bed elevation changes can be developed with this approach without resorting to more-complex techniques. Some details of the procedure combining the Meyer-Peter and Muller bed load equation and the Einstein suspended load procedure are outlined in the following paragraphs.

Sediment bed material load consists of bed load and suspended load. The bed load discharge may be fit to the following function:

$$q_b = \beta_1 (\tau - \tau_c)^{\beta_2} \tag{88}$$

where

- q_b is the bed load discharge per unit width of channel,
- τ is the boundary shear stress,
- τ_c is the critical shear stress related to particle size, and
- β_1 and β_2 are constants.

β_1 and β_2 may be calibrated if data exist, or they can be set to values given in Meyer-Peter and Muller (1948), where

$$\beta_1 = \frac{8w}{\sqrt{\rho \gamma'_s}} \quad ; \quad \beta_2 = 1.5$$

where

- w is the width,
- ρ is the mass density of water, and
- γ_s is the specific weight of submerged sediment.

The boundary shear stress is determined by

$$\tau = \frac{1}{8} f \rho V^2 \tag{89}$$

where

- f is the Darcy-Weisbach friction factor for grain resistance and
- V is the average velocity of the flow.

The critical shear stress is given by

$$\tau_c = 0.047 \gamma (S_s - 1) d_s \tag{90}$$

where

- S_s is the specific gravity of the sediment and
- d_s is the representative sediment size.

Following a procedure similar to that of Einstein for determining the suspended load,

$$q_s = \frac{q_b}{11.6} \frac{G^{w-1}}{(1-G)^w} \left[\left(\frac{V}{U_*} + 2.5 \right) \int_G^1 \left(\frac{1-r}{r} \right)^w dr + 2.5 \int_G^1 \ln r \left(\frac{1-r}{r} \right)^w dr \right] \tag{91}$$

where

- q_s is the suspended load transport per unit width of channel;
- U_* is the shear velocity, defined as τ/ρ;
- ρ is the density of water;
- V is the mean velocity of flow;
- r is ξ/y;
- G is d_s/y;
- y is the flow depth;
- w is $V_s/(0.4U_*)$;
- V_s the settling velocity of the sediment; and
- ξ is the distance measured from the bed.

These two integrals are evaluated numerically. Multiplying q_b and q_s by the channel width results in the total bed load (Q_b) and suspended load (Q_s) for the entire width. Therefore, the total sediment transport for a reach will be

$$Q_T = Q_b + Q_s + Q_w \tag{92}$$

where

Q_w is the wash load.

Quasi-Dynamic Sediment Modeling

The previous sediment routing model based on a gradually varied flow backwater program assumes a rigid boundary system. The methodology can be extended to account for unsteady flow and alluvial channel boundaries without going to a fully unsteady water and sediment routing model. The model can include a watershed system as shown in Figure 59.

The quasi-dynamic sediment model uses the same gradually varied flow backwater program for hydraulic computations. However, the flow is assumed constant for a given time increment, Δt. A flow event, either short- or long-term, can be broken into a number of time increments, each with a different flow, but during each increment the flow is considered steady.

To account for cross-section geometry changes during aggradation and/or degradation processes, the channel section must be periodically adjusted to reflect changes in the alluvial boundary. This is accomplished by distributing the computed aggradation or degradation during each time-step to the channel sections. The volume of sediment aggradation or degradation within a reach for a given time period is given by

$$\Delta V_s = \text{(sediment supply - sediment transport)} \; (1-\lambda)$$

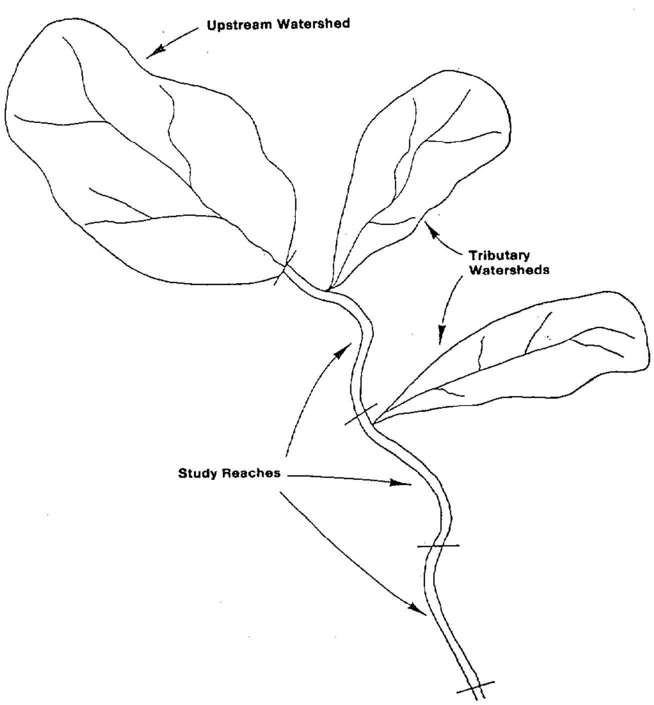


Figure 59. WATERSHED RIVER SYSTEMS

where

ΔV_s = the change in sediment volume is the reach and

λ = the porosity of channel bed material.

The change in sediment volume is assumed to be uniformly distributed through the reach. Change in area for each cross section is determined by a weighting factor based on the conveyance in adjacent segments of the cross sections. The changes in elevation are used to generate a new HEC-2 data file for the next time period. Therefore, during any given time period, the channel boundary is assumed rigid and the HEC-2 analysis is valid. After evaluating the hydraulic conditions and the sediment transport capacity, the channel boundary is modified to reflect the aggradation-degradation changes occurring throughout the river and to establish the new channel configuration for the next time step.

This methodology has been successfully applied to a number of practical engineering problems. It provides a feasible and cost-effective approach to design problems in alluvial rivers.

Dynamic Mathematical Modeling

Dynamic mathematical modeling of water and sediment routing is the next level of sophistication and complexity in determining gradation changes. It involves unsteady nonuniform flow routing to determine the hydraulic conditions so that sediment transport, aggradation, and degradation can be calculated.

Unsteady nonuniform flow routing solves equations governing the motion of water in open channels. These equations are mathematical descriptions of physical principles or phenomena. Continuity states that water coming into a reach is either stored in the reach or passes on downstream without creating or destroying water. A mathematical expression of the continuity equation is

$$q_L = \frac{\partial Q_s}{\partial x} + \frac{\partial A}{\partial t} \qquad (93)$$

where

- Q is the discharge,
- x is the downstream distance,
- A is the cross sectional area,
- t is time, and
- qL is the lateral inflow or outflow.

The momentum principle balances the forces and accelerations acting on flowing water and can be described by the full dynamic momentum equation

$$S_f = S_o - \frac{\partial y}{\partial x} - \frac{1}{gA}\frac{\partial(Q^2/A)}{\partial x} - \frac{1}{gA}\frac{\partial Q}{\partial t} \qquad (94)$$

where

- S_f is the friction slope,
- S_o is the bed slope,
- y is the depth, and
- g is the acceleration of gravity.

In some cases, such as channels with steep slopes, the momentum equation can be simplified under the kinematic wave assumption, which is

$$S_f = S_o. \tag{95}$$

Generally these equations, along with a resistance-to-flow equation involving Manning's n or Chezy's c, are solved numerically in finite difference form. The results are the hydraulic variables of velocity, depth, and width for unsteady nonuniform flow. These variables are then used in the sediment routing scheme. Sediment movement is controlled by the shear forces acting on the bed, transport capacity of the flow, and availability or supply. Equations used in these calculations were previously described. To compute aggradation and degradation, the sediment continuity equation is used.

$$q_{Ls} = \frac{Q_s}{\partial x} + \frac{\partial CA}{\partial t} + (1 - \lambda)\frac{\partial PZ}{\partial t} \tag{96}$$

where

Q_s is the total sediment transport rate in volume per unit time, the concentration $C = \frac{Q_s}{Q}$;

λ is the soil porosity;

P is the wetted perimeter;

z is the depth of loose soil; and

q_{Ls} is lateral sediment inflow or outflow.

This equation can be applied for each size class of sediment in a given analysis and can successfully account for armoring of the bed.

This type of approach can given excellent results but is not often required to solve practical problems. Most aggradation and degradation problems related to such things as highway crossings can be solved by the previous methods.

COST, EAST OF APPLICATION, ACCURACY, AND ABILITY REQUIRED

The cost of any engineering analysis is a function of the level of effort, which in turn depends on the accuracy required, the time available, and the analysis techniques that are used in reaching the solution. Generally, the more complicated the analysis, the more accurate the solution becomes; however, the cost also increases.

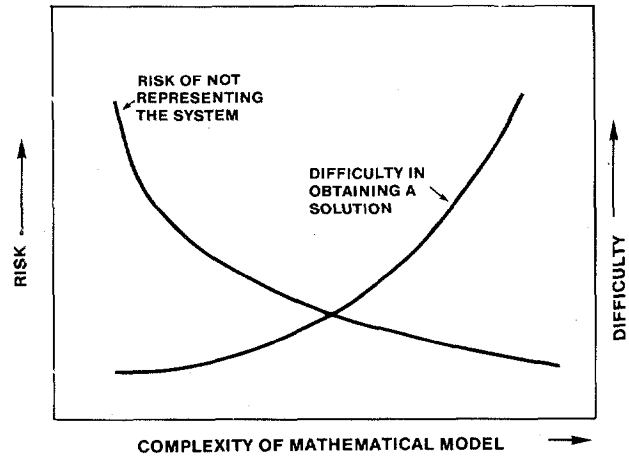
The three level-of-solution procedures – geomorphic techniques, geomorphic techniques and basic engineering relationships, and mathematical modeling – described in this chapter are illustrative of this generalization. A qualitative analysis based on geomorphic concepts provides an insight into and an understanding of complicated problems at a fraction of the cost of a quantitative approach. Such an analysis usually takes experienced engineers approximately 1 to 2 weeks. A quantitative solution based on geomorphic concepts and basic engineering relationships provides design figures and numbers at a reasonable cost. The manpower requirement for such a solution usually ranges from 4 to 10 weeks. However, a more accurate quantitative assessment can only be made by using mathematical models considering the simulation of water and sediment movement on a real-time basis. In a conventional method or analysis, a series of manual calculations may be required. With the advance of numerical techniques and computer technology, a series of tedious computations can be conducted efficiently and repeatedly through the formulation and construction of a mathematical model.

Using a well-developed model, a whole array of "what if" questions can be answered with minimum time and effort. Since no process can be completely understood and observed, any mathematical expression of a process will involve some level of uncertainty. This uncertainty can be minimized if the governing physical processes are considered in the analysis and the model is properly designed, calibrated, and verified. Model developed, verification, and application to real-world problems requires the consideration of the nature of the problems, the physical environment, the objectives of the study, and the time, manpower, and money available. If mathematical modeling is involved, the manpower requirement for a typical job ranges from 6 to 20 weeks.

Since time, manpower, and money are always limited resources, decisions must be made by the model users and developers as to the degree of complexity the model is to have and the extent of the verifications that are to be performed. According to Overton and Meadows (1976), if a highly complex mathematical representation of the system under study is made, the risk of not representing the system will be minimized but the difficulty of obtaining a meaningful solution will be maximized. Much data will be required, programming effort and computer time will be significant, and the general complexity of the mathematical handling may even render the problem formulation intractable. Furthermore, the resource constraints of time, money, and manpower may be exceeded.

Conversely, if a greatly simplified mathematical model without proper examination of physical significance is selected or developed, the risk of not representing the physical system will be maximized but the difficulty in obtaining a solution will be minimized. Figure 60 shows the general concept of "tradeoffs" considering model complexity. The knowledge of governing physical processes and the sensitivity of system response plays the most important part in deciding an appropriate level of analysis. It is possible to select or develop a model that is simple to use and involves a minimum level of risk if the governing physical processes are emphasized in the analysis.

The application of all analytic procedures requires trained personnel to evaluate and interpret the results. Geomorphic concepts do not require extensive mathematical or computer analysis; however, they do require a very well-founded knowledge of the significant physical processes. Engineering relationships and mathematical models can be used as "black box" calculations by support personnel. However, the design engineer must provide the



Source: Overton and Meadows (1976)

Figure 60. MODEL COMPLEXITY TRADE-OFF DIAGRAM

proper input and be capable of interpreting the output.

The dynamic nature of river and watershed systems requires that local problems and their solutions be considered in terms of the entire system. Natural and man-induced changes in a river frequently initiate response that can be propagated for long distances both upstream and downstream [Simons and Senturk (1977)]. Successful river utilization and water resources development require a general knowledge of the entire watershed and river system and the processes affecting it. This complexity inhibits simplified solutions to design problems. There are no easy answers when performing engineering design in a river or watershed system.

CHAPTER VII

APPLICATION OF METHODS

This chapter outlines a procedure for analysis of gradation changes in a river system. In the previous chapters, various geomorphic and engineering concepts, methods for estimating gradation changes in unstable river systems, and the general information required for such an analysis were presented. That information forms a foundation from which to draw the necessary techniques for conducting a stability analysis. However, because each river system is unique and because the factors that produce gradation changes in these systems are also, for the most part, unique, a "cookbook" approach to the solution of gradation problems is not possible. However, a general outline documenting a recommended approach to the solution of each problem is presented as an aid for applying the procedures and techniques presented in the previous chapters.

The approach outlined here has been applied to case histories and the results are given in Appendix C.

GENERAL APPROACH

The recommended approach is designed to follow a logical process considering the potential for a grade change, or some other instability, at a crossing site. The first step in the process is to determine whether the specific river reach under consideration is stable with respect to its water-sediment complex. If it is found to be potentially unstable, the magnitude of the potential instability must be documented and the potential impacts analyzed. The latter consideration includes evaluation of an appropriate countermeasure to protect the crossing.

Table 17 presents an outline of the approach for analyzing the potential for and magnitude of a grade change at a crossing site. The outline is divided into three regions: qualitative system stability analysis, analysis of the potential grade change magnitude, and impacts on crossing and countermeasure design. Each of these three regions corresponds to the respective level in the process. The outline is described in the following sections.

Table 17. SYSTEM ANALYSIS OUTLINE

PHASE I:	Qualitative System Stability Analysis
	1. Preliminary Data Acquisition
	2. Preliminary Data Analysis
	3. Stability Considerations
PHASE II:	Analysis of Potential Grade Channel Magnitude
	1. Level of Analysis
	2. Further Data Acquisition and Analysis
	3. Further Stability Considerations
	4. Analysis of Grade Change
PHASE III:	Impacts on Crossing and Countermeasure Design
	1. Impacts on Crossing Design
	2. Countermeasure Selection
	3. Evaluation of Other Impacting Factors

QUALITATIVE SYSTEM STABILITY ANALYSIS

The first step in the analysis of a river system is to establish from observations and historic data whether a reach of river in the vicinity of a highway crossing is stable with respect to its hydraulic geometry, bed elevation, and flow alignment. This analysis consists of three stages: obtaining the necessary background information and a history of the system, analyzing the data, and reaching a conclusion about the stability. A discussion of the recognition of grade change problems was included in Chapter V of Report No. FHWA/RD-80/038.

Preliminary Data Acquisition

The data required for preliminary stability considerations include maps, aerial photography, field observations, and other miscellaneous information available from various agencies. It is important to obtain historic as well as recent documentation of the specific reach and system under consideration so that the necessary comparisons can be made.

Types of maps needed include area maps, vicinity maps, site maps, geologic maps, and land use and soils maps. The purpose of an area map is to locate the river with respect to other systems. An unstable downstream river system is a possible cause of instability in tributary streams. Vicinity maps for each river crossing are needed to document more localized instablity problems. A sufficient length of the river reach should be included on the vicinity map to enable identification of stream type and other system classification parameters and to locate bars, braids, and potential channel controls. Site maps are needed to show more specific information on hydraulic conditions at the specific crossing site that could influence local stability. This specific information includes local flow alignment and location of bars and location of local tributaries. A geologic vicinity map on which geophysical features are indicated is a basic requirement because the rock formations, outcroppings, and river deposits that form control points are valuable for the analysis of river stability. Soils and land use maps are also required because soil type, vegetative cover, and land use have important effects on the availability and characteristics of sediment transport material.

Along with the maps, aerial photographs are required. Multi-image or simple black and white photographs can be used. Modern multi-image cameras use different ranges of the light spectrum to assist in identifying various features such as types of vegetation, sizes and heights of sandbars, river thalwegs, river controls and geologic formations, existing bank protection works, old meander channels, and other features. The resolution required in the photographs is a function of the river system size; larger resolution can be used with larger river systems. In general, the resolution should be small enough to distinguish channel characteristics and vegetation types.

A field inspection of potential highway crossing sites is the most important component of the initial stability analysis. It provides the best indication of the behavior of a river system. Observed conditions should be recorded photographically as well as with adequate notes. Field visits are advisable during both the low flow and high flow periods. The conditions of banks and beds should be described and the locations of bank cutting or slumping and deposition in the channel bed should be indicated. Structures encountered should be inspected for signs of aggradation or degradation and the locations and implications of impacting activities. Vegetation types and densities should also be documented.

Preliminary stability data may also be available from such government agencies as the U.S. Army Corps of Engineers, U.S. Geological Survey, U.S. Soil Conservation Service, U.S. Forest Service, local river basin commissions, and local watershed districts. Pertinent information that these agencies can provide includes historic streambed profiles, stage-discharge relationships, and sediment load characteristics. These agencies also often have records of past systems activties that might have impacted the stability and might give an indication of future instability characteristics.

Data sources for hydrologic and gradation studies were documented in Report No. FHWA/RD-80/038 and are reproduced in Table 18.

Preliminary Data Analysis

In reviewing procedures for recognizing the existence of grade change problems, Keefer, McQuivey, and Simons (1980) described techniques that included regional awareness, awareness of impacting activities, awareness of local geology/geomorphology, and various direct identification techniques including long-term streambed observations, observations of changes in stage-discharge relations, observations of changes in sediment load, and streambed profile analysis.

The first step in the stability analysis is to classify the wateshed and river according to classification schemes recommended in Chapter V. River and watershed classification provides insight into typical watershed behavior and response. It also provides information on impacting activities within the watershed. From the watershed and river classification and from field visits, channel stability can be interpreted. A summary of interpretations taken from Keefer, McQuivey, and Simons (1980) is given in Table 19.

In addition to the use of classification systems, the comparison of channel and system changes with time can provide evidence of channel instaiblity. From a comparison of time sequential maps and/or aerial photographs, extreme changes in channel

Table 18. DATA SOURCES FOR HYDROLOGIC AND GRADATION STUDIES

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 plats 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, Water Power and Resources Service.
(5)	Local areas – commerical 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, Water Power and Resources Service.
(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; commerical 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 regular quandrangle maps show geological 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, Water Power and Resources Service.
(4)	Hydraulic laboratory reports – U.S. Department of the Interior, Water Power and Resources Service.

Table 18. DATA SOURCES FOR HYDROLOGIC AND GRADATION STUDIES (Continued)

Climatological Data:

(1) National Weather Service Data Center.
(2) Hydrologic Bulletin – U.S. Department of Commerce, National Oceanic and Atmospheric Administration.
(3) Technial 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, Water Power and Resources Services Service.

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, Water Power and Resources Service.
(6) Owners of Reclamation.
(7) Corps of Engineers, U.S. Army, flood control studies.

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, Water Power and Resources Service; and U.S. Department of Agriculture and Soil Conservation Service.

Quality of Water Reports:

(1) Water supply papers – U.S. Department of the Interior, 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, Water Power and Resources Service.
(5) Environmental Protection Agency, regional office.
(6) State water quality agency.

Irrigation and Drainage Data:

(1) Agriculture 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 project – U.S. Deparrment of the Interior, Water Power and Resources Service.
(4) Reports and Progress reports – U.S. Department of the Interior, Water Power and Resources Service.

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, Water and Power Resources Service, Fish and Wildlife Service, and National Park Service;
(3) U.S. Department of Agriculture, Soil Conservation Service.

Table 18. DATA SOURCES FOR HYDROLOGIC AND GRADATION STUDIES (Continued)

Basin and Project Reports and Special Reports (Continued):	
(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 commission.
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 of 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.

alignment and flow habit with time can be determined. If historic channel profile data are available, a stream profile analysis can be conducted. If there is a stream gage along the channel under consideration, and analysis of stage trends would provide information on channel stability.

Of paramount importance to this phase of the analysis is an awareness of impacting activities. The most significant factors influencing system stability are those that are caused by activities that change some aspect of the natural morphology of a river. Through contacts with agencies involved with the river under consideration (U.S. Army Corps of Engineers, U.S. Soil Conservation Service, U.S. Forest Service, U.S. Geological Survey, local watershed districts, etc.) information can be found relating to human activities in the given watershed. Field observations and a review of maps and aerial photographs will also reveal the location of impacting activities.

Further indications of system instability can be made by various geomorphic analysis techniques, which are generally qualitative. After a highway engineer has determined that a number of impacting activities exist near a bridge, such techniques may be used to determine the trend of the problem. Geomorphology theory provides information on the direction and the extent of gradation problems but generally does not provide numbers useful in exact design. Nevertheless, a knowledge of geomorphology is very important for selecting a more exact hydraulic analysis method.

Geomorphic techniques were discussed in Keefer, McQuivey, and Simons (1980) and have been reviewed in this report. Lane's equilibrium principle and slope-discharge relationship, sinuosity-slope curves, and other techniques were reviewed. A summary of these techniques is presented in Table 20.

Thus far in this chapter, the first phase of the three-phase approach to solving aggradation-degradation problems has been discussed. Through observation and simple analysis, the bridge engineer can identify a gradation problem. If none exists, the bridge design can be completed. However, if evidence is found of a potential for a grade change in the system under consideration, an analysis of the potential magnitude of the change must be conducted as the second phase of the process shown in Table 17.

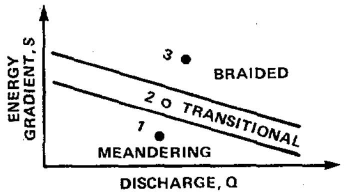
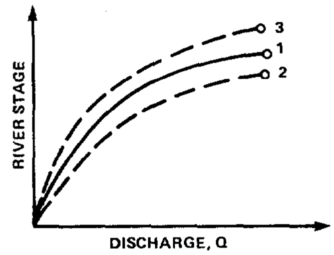
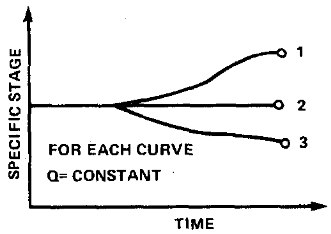
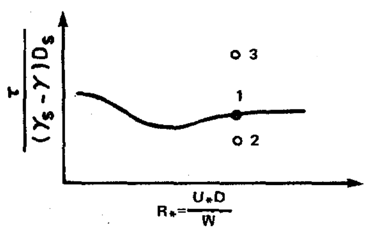
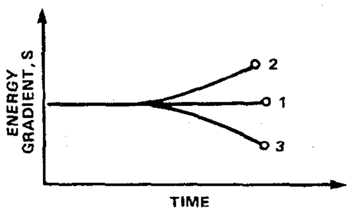
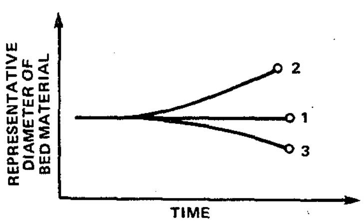
ANALYSIS OF THE POTENTIAL GRADE CHANGE MAGNITUDE

After the potential for a grade change problem is confirmed, the bridge engineer must determine the

Table 19. INTERPRETATION OF OBSERVED DATA

OBSERVED CONDITION	CHANNEL RESPONSE			
	STABLE	UNSTABLE	DEGRADING	AGGRADING
Alluvial Fan				
Upstream		√		√
Downstream		√	√	
Dam and Reservoir				
Upstream		√		√
Downstream		√	√	
River Form				
Meandering	√	√	Unknown	Unknown
Straight		√	Unknown	Unknown
Braided		√	Unknown	Unknown
Bank Erosion		√	Unknown	Unknown
Vegetated Banks	√		Unknown	Unknown
Head Cuts		√	√	
Diversion				
Clear water diversion		√		√
Overloaded with Sediment		√	√	
Channel Straightened		√	√	
Deforest Watershed		√		√
Drought Period	√			√
Wet Period		√	√	
Bed Material Size				
Increase		√		√
Decrease		√	Unknown	√

Table 20. GEOMORPHIC AND HYDRAULIC ANALYSIS OF RIVERBED LEVEL CHANGES

METHOD OF ANALYSIS	CHANNEL CONDITIONS		RIVER BED RESPONSE		
	STABLE	UNSTABLE	AGGRADING	DEGRADING	UNKNOWN*
	1	2,3			1,2,3
	1	2,3	3	2	
	1	2,3	2	3	
	1,2	3			1,2,3,
	1	2,3	3	2	
	1	2,3			1,2,3

*DETERMINED BY OTHER METHODS OF ANALYSIS.

extent or magntiude of the problem. This determination constitutes the second phase in the analysis and consists of the steps outlined in Table 17: level of analysis, further data acquisition and analysis, further stability considerations, and analysis of grade changes. Each of the steps is further outlined below.

Level of Analysis

From the various techniques and engineering concepts presented in Chapters V and VI, it is apparent that various levels of analysis exist for predicting the magnitude of grade changes in river systems. These levels of analysis range from the application of geomorphic and engineering concepts to various levels of mathematical modeling. Each analytic level involves a different degree of sophistication. Associated with the increased complexity, higher levels of analysis require more detailed input data and increased levels of engineering expertise. Furthermore, the more detailed the level of analysis, the higher the cost and the greater the time required.

The level of analysis chosen for a given job will be a function of several considerations. These considerations include the degree of instability indicated by the preliminary qualitative analysis, the size and character of the river system being considered, the availability of requisite data, the expertise of persons conducting the analysis, and the potential economic and social impacts of losing the bridge.

Based on these factors and knowledge of other impacting activities (such as potential debris and ice problems), a choice should be made as to the requried level of analysis of a given project.

Further Data Acquisition and Analysis

With the appropriate level of analysis chosen, additional data requirements can be evaluated. The types of information required in addition to that obtained as a part of the preliminary stability analysis includes hydrologic data, channel geometry properties, bed and bank material characteristics, and channel and bank roughness estimates. Depending on the solution techniques chosen, additional data might also be required.

The hydrologic data required for the analysis of grade changes include rainfall and/or runoff data to provide an estimate of typical channel discharges in the form of dominate discharges, flow duration curves, and flow frequency curves. Stream gaging stations have been established on many streams throughout the United States. Where there is a gage at or near the reach of interest, the daily discharge data recorded by the USGS in its water data reports can be used for the required analysis. However, in some streams, there is either no gaging station near the project site or none exists at all. In such cases, streamflows must be estimated on the basis of regionalized estimating procedures or from other prediction models using meterological and watershed data inputs. A discussion of these methods is beyond the scope of this report. Meteorological data are available from the National Weather Service (NWS) Data Center of the National Oceanic and Atmospheric Administration (NOAA), and estimates of average conditions can be made from rainfall data published by the NWS.

The characteristics of bed and bank materials are of extreme importance to any study involving sediment transport. Properties of bed and bank material particles indicate the behavior of the particles in their interaction with the flow. Several of the important bed-material properties are size, shape, fall velocity, cohesion, and angle of repose. These properties must be obtained and used as inputs to the various techniques described in Chapters V and VI. Extensive descriptions of procedures for analyzing these characteristics are documented in Richardson et al.(1974) and Simons and Senturk (1977).

Analysis of sediment transport requires the consideration of bed forms and bed roughness. A brief review of bed forms and their relationships to channel roughness is presented in Chapter V with most of the details referenced to Richardson et al. (1974). Roughness considerations are used as input to the calculation of sediment transport rates and backwater computations.

At times it is necessary to obtain field data to verify bed load and suspended load transport rates under various flow conditions. This practice is recommended when time and funds are available for a field data collection program. It is particularly important to have field transport data any time the sediment type and transport characteristics are different from those for which the transport equations were developed. Obtaining field transport data is also recommended when calculated transport rates differ from observed conditions. Techniques and equipment for obtaining suspended and bed load materials have been documented by the ASCE Task Committee for the Preparation of the Manual on Sedimentation of the Sedimentation Committee of the Hydraulic Division (1975).

Further Stability Considerations

A previous section of this chapter, *Preliminary Data Analysis,* discussed quantitative stability consideration that were based predominantly on historic comparisons of channel plan and profile and visual observations from field inspections. While these analytic techniques indicate past instability and are enough to signal the potential for a grade change problem, they do not give an indication of the potential degree of future instability. Such an indication requires a more quantative analysis. Tools for conducting that type of analysis are presented in Chapter VI and are briefly discussed below.

Relative stability can be analyzed using incipient motion considerations. Typically, incipient motion is based on boundary shear stress. The techniques used will depend on the type of material that constitutes the channel boundaries. Based on this criterion, the techniques are divided into two cateogries, those that apply to noncohesive boundaries and those that apply to cohesive boundaries.

For noncohesive boundaries of sand, gravel and cobble type material, Shield's technique as modified by Gessler is recommended. (This technique is presented in Chapter V.) The critical shear stress computed in this manner is compared with the actual boundary shear stress to verify stability or instability.

For cohesive boundary channels, a technique similar to that for noncohesive channels is used. However, the method for obtaining the critical shear stress is different. Ariathurai and Kandiah (1978) indicate that numerous physical and chemical factors (including the chemical composition of the water) affect the erosivity of saturated cohesive materials. Because of the variety of factors involved, the problem of predicting some critical condition is extremely complex. Ariathurai and Kandiah (1978) recommend laboratory analysis to determine the critical shear stress of a given material. The measured critical shear stress is then compared with the actual boundary shear stress to estimate the magnitude of the potential instability.

Actual boundary shear stresses for comparison with critical shear stresses were investigated by Lane (1953). The technique presented in Chapter V should be used to compute actual boundary shear stress for comparison.

Using the above mentioned techniques, both channel bed and banks can be evaluated for stability. When considering bank stability, the bank cohesion produced by the vegetative root structure should be considered. This is done by increasing the natural angle of repose of the bank material based on bank steepness in laterally stable reaches of the river.

Lane's slope-discharge relationship, discussed in Chapter VI, can also be applied to give a better indication of potential instability. Knowledge of typical discharges and channel slopes will allow use of Figure 51 in a more quantitative fashion to determine if there is the potential for a change in channel form that would lead to some degree of lateral or vertical instability.

Analysis of Grade Change

With the presence of the instability problem firmly established, the magnitude of the potential grade change can be determined using one or more of the techniques presented in Chapter VI. At this point, a final choice of the level of analysis should be made and any additional data needs should be filled.

The typical levels of analysis presented in Chapter VI range from the application of various engineering and geomorphic relationships to dynamic mathematical modeling techniques. At this point, it is important to realize that the higher levels of analysis should not be considered independent of the lower levels. The application of engineering and geomorphic techniques provides a check on the modeling results. Too often the significance of the results produced by a complex computer model is exaggerated. It is always good engineering practice to check computer results by applying the various geomorphic and engineering relationships available. Besides providing a check, this practice also gives the engineer better insight into the processes occurring in the system being investigated.

When applying the various grade change prediction techniques, the question of what discharge or series of discharges is to be used is important. For river reaches downstream of dams or in canal systems, the discharge is approximately constant and can be assumed as such for the analysis. However, in natural systems the situation is not so simple. Several situations must be considered. First, a dominant discharge can be computed. The dominant discharge is defined as the average discharge that is equivalent to the variable natural river flow that would produce a channel similar to the natural channel. The dominant discharge is used in the regime type formulas discussed in Chapter V and VI, which describe possible variations in channel geometry (size, shape, and meander characteristics).

For the computation of bed volume changes and other modeling techniques, however, it is necessary to consider some typical series of discharges. Three such series should be considered. The first is a typical annual flow duration curve. By taking a finite difference approach and breaking the flow duration curve into a series of typical discharges over a period of a year, the impact of average flow conditions on the rate and magnitude of the grade change can be evaluated. Analysis of a flow duration curve is best suited for perennial rivers where a gradual change in river grade as a response to changes in average flow conditions is expected.

The second flow series is the evaluation of a 50- or 60-year period by considering a flow frequency analysis. By hypothesizing the frequencies of occurence at 2-, 5-, 10-, 25-, 50-, and 100-year floods and considering these floods singly and in selected combinations, a long-term response can be predicted. This approach is recommended in the case of ephemeral rivers in the arid West and Southwest where channels are dry during most of the year and where flood events are known to have a major impact with respect to grade changes.

The third and most complex (and expensive) flow series which can be considered is to simulate a 50- or 60-year flow hydrograph. Such an analysis, while possible, is not recommended. The development of such hydrographs is filled with uncertainty and speculation. Also, to simulate water and sediment routing for such a hydrograph requires extensive computer time and becomes very expensive.

If one of the discharges or discharge series is known to be responsible for an expected grade change, only that flow situation need be evaluated. However, when this is not the case, all situations must be investigated to establish the possible impacts produced by different hydrologic situations.

IMPACTS ON BRIDGE AND COUNTERMEASURE DESIGN

Once the magnitude of the grade change has been established, the impacts of the change on bridge and countermeasures design can be evaluated. The process involves the following three steps:

- evaluate the impacts produced by the grade change on the current crossing or new crossing design;
- select an appropriate countermeasure, and
- evaluate other impacting factors.

These steps are briefly described in the following sections.

Impacts on Bridge Design

In Chapter III, the typical impacts of grade change problems at highway crossings are described. The components of bridge design most often influenced by aggradation and degradation are flow capacity, required bridge span, local flow velocities, backwater conditions, pier and abutment alignment, and pier and abutment foundation depths. From an estimate of the potential magnitude of the grade change, it is possible to evaluate how these components might be affected for new or existing bridges. With this knowledge, an appropriate design and/or countermeasure can be selected.

Countermeasures Selection

An analysis of the effectivness of various countermeasures for controlling aggradation and degradation problems and for protecting channel crossings is presented in Chapter VIII. The analysis is based on review of more than 200 case histories. Countermeasures selection should be based on the results of this survey and on engineering experience.

Other Impacting Factors

Other factors that affect the design of river crossing structures on aggrading or degrading channels include local and general scour, debris, and ice problems.

Local and general scour are defined in Chapter I. Aggradation and degradation processes in the vicinity of a highway crossing can have a significant impact on local and general scour if local flow alignment and velocities are drastically altered during the process. (These effects are discussed in Chapter III.) The important point is that both local and general scour must be added to the predicted grade change when estimating the maximum potential depth of scour.

Ice and debris problems are also discussed in Chapter III. Ice and debris affect river crossing structures by clogging the waterway opening and possibly damaging the bank protection and piers. These factors must also be considered when designing an adequately protected crossing.

CHAPTER VIII

COUNTERMEASURES: CLASSIFICATION AND ANALYSIS

GENERAL

For the purpose of this report, countermeasures are defined as those measures incorporated into the bridge design or installed separately at or near the bridge that serve to prevent or to control grade changes within a river system. These grade changes make themselves evident in the form of aggradation, degradation, and lateral erosion.

The classification and analysis of the effectiveness of these countermeasures in this report are based on approximately 200 case histories documenting hydraulic problems at crossings. The data base is made up of the 110 case histories documented as a part of this study (included as Appendix A) and 86 of the case histories reported by Brice, Blodgett, et al. (1978) in Report No.FHWA-RD-78-163.*

One hundred and six of the case histories document the use of various countermeasures to correct hydraulic problems. Countermeasures were not applied in the remaining case histories. The case histories documenting the use of countermeasures are first classified by the grade change type causing the problem (aggradation, degradation, or the associated lateral erosion) and then are broken down by the type of measure used to control the problem. The following is a list of the countermeasures documented in the data base:

- flexible revetment
  - riprap
  - rock-and-wire mattress
  - gabion
  - car bodies
  - planted vegetation
  - sacked concrete
- rigid revetment
  - concrete pavement
  - grouted riprap
  - concrete-filled fabric mats
  - bulkheads
- impermeable spurs
  - rock or earth embankment
  - sheet pile
- permeable spurs
  - timber pile
  - steel pile, pipe, or rail
- impermeable retards
  - rock or earth embankment
  - timber pile
  - sheet pile, pipe, or rail
- permeable retards
  - timber pile
  - steel pile, pipe, or rail
  - jack or tetrahedron fields
- drop structures
  - check dams
  - cutoff walls
- dikes
  - rock or earth embankment
  - steel pile, pipe, or rail
- spur dikes
  - revetted rock or earth embankment
  - rock or earth embankment not revetted
- pier protection
  - sheet pile casing
  - underpinning or jacketing of pier/abutment
  - driving of piles deeper/deeper placement of footings
- countermeasures built into new or replacement bridges
  - drop culvert/drop flume construction
  - fewer or no piers in channel
  - realignment of approach channels
  - deeper placement of piles and footings
  - lining bridge openings

*All detailed case histories are available from the Contract Manager, Mr.Stephen A. Gilje, HRS-42, Washington, D.C. 20590.

- increased bridge length
- increased bridge width
- raising bridge superstructure
- relocation of bridge

Table 21 (a) through (c) list the case histories by countermeasure type, specifying those applications that were successful and those that were not. Table 21 (a) lists countermeasures used at degrading sites; Table 21 (b) those used at aggrading sites; and Table 21 (c) those used at sites to counter lateral erosion that has resulted from either aggradation or degradation. Case histories Numbers 1 through 283 are documented in Report No. FHWA-RD-78-163.

Because countermeasures are often used in combination with other measures and because a particular crossing could be experiencing more than one type of grade problem (such as degradation and lateral erosion), the same case history number can be listed under several countermeasures as well as in more than one of the tables.

This chapter not only evaluates countermeasures built at existing bridges but also considers countermeasures built as a part of new crossings. Of particular interest is the performance of various new stream crossing designs that are intended to provide high safety factors against grade change problems. The design practices exemplified include minimizing the number of piers or other obstructions located in the stream channel, construction of very deep foundations, extensive bed armoring, etc.

The analysis of countermeasures is presented by the type of gradation problem. Because degradation is encountered more frequently, its countermeasures are discussed before those used for aggradation. Finally, countermeasures to control the lateral erosion problems often accompaning aggradation and degradation are reviewed. Within each of these groups, specific countermeasures are discussed and recommendations made for their use under various environmental and flow conditions.

DEGRADATION

In 56 of the case histories, some countermeasure was used to control channel degradation or reduce its impact on the crossing; in 11 of the case histories, more than one countermeasure was used. These 56 cases were divided into three categories: measures to control degradation, measures to minimize the hazards produced by degradation, and measures built as a part of new or replacement bridges to minimize the hazards of anticipated degradation. In all, 72 countermeasures are documented, and 43 were successful. If the success record for check dams (17 of 19 check dam countermeasures were successful) is removed from this comparison, only 26 of the remaining 55 countermeasures were successful — a 49 percent success rate.

As with countermeasures to protect crossings from other grade change problems, the effectiveness of the various degradation countermeasures is a function of the level of hydraulic analysis that goes into the design. In a significant number of cases, an adequate evaluation of the potential magnitude of the grade change was not anticipated, and as a result, the countermeasure design was not appropriate. This fact is reflected in the reported success rating.

Countermeasures To Control Degradation

Countermeasures used to control channel degradation documented in the case histories include the use of check dams and channel linings.

Check-dam and check-dam type structures have been given a variety of names. They include drop structures, cutoff walls, drop flumes, and flume bridges. A few of the typical check-dam type structures are shown in Figures 61 through 68. A check dam (or similar structure) is a low dam or weir constructed across a channel to prevent degradation. It is usually built of either rock riprap, concrete, sheet pile, gabion, or treated timber piles. The structure produces a bed-level control point in the system that will not allow a degradation profile to pass through it. When constructed just downstream of crossings, these dams have proved effective in protecting the bridge foundation system by maintaining the original streambed profile through the crossing reach.

Figures 61 through 65 show typical check dams. Figure 58 shows a sheet pile check dam (hidden by the flowing water) on Mosquito Creek in Shelby

Table 21(a). CASE HISTORIES WITH DEGRADATION COUNTERMEASURES

Key to Table 21

S = successful

U = unsuccessful

C = initial countermeasure

L = some subsequent countermeasure

M = moderate or limited success

B = undetermined success (usually, the countermeasure has not been in place long enough for evaluation)

A = countermeasure was used in combination with one or several other countermeasures

An asterisk indicates case histories from Report No. FHWA/RD-78/163

COUNTERMEASURE	RESULTS	CASE HISTORY NUMBERS
	FLEXIBLE REVETMENT	
Riprap	S	92*, 192M*, 37B*, 123B*, 124*, 177*, 205B*
	U	24C*, 42*, 173*, 198*, 222*, 14, 42, 53, 16
Railbank	S	85*
	U	
Rock-and-Wire Mattress	S	277*, 278*
	U	
Planted Vegetation	S	
	U	37*
Sacked Concrete	S	
	U	1C*
	RIGID REVETMENT	
Concrete Pavement	S	178*
	U	39*, 55*, 183*
Grouted Riprap	S	124*
	U	43

Table 21(a). CASE HISTORIES WITH DEGRADATION COUNTERMEASURES (Continued)

RIGID REVETMENT (Continued)		
Concrete-Filled Fabric Mats	S	183L*
	U	
Bulkheads	S	247*, 145M*
	U	
IMPERMEABLE RETARDS		
Steel Pile, Pipe, or Rail	S	
	U	1C*, 7C*
PERMEABLE RETARDS		
Timber Pile	S	
	U	261*
DROP STRUCTURES		
Check Dams/Cutoff Walls	S	10*, 85L*, 89*, 90*, 159*, 161*, 1, 5, 11, 16M, 31, 34, 46M, 95, 96, 98, 103
	U	85C, 87
SPUR DIKES		
Earth or Rock Embankment	S	57B*, 77A
	U	
PIER PROTECTION		
Sheet Pile Casing	S	
	U	174*
Underpinning or Jacketing of Piers/Abutments	S	
	U	148*, 247*, 261*, 282*, 20, 43
Driving of Piles Deeper/ Deeper Placement of Footings	S	1LB*, 227*, 241C*
	U	174*, 197B*
COUNTERMEASURES BUILT INTO NEW OR REPLACEMENT BRIDGES		
Fewer or No Piers in Channel	S	55*, 123*, 53*
	U	

Table 21(a). CASE HISTORIES WITH DEGRADATION COUNTERMEASURES (Continued)

COUNTERMEASURES BUILT INTO NEW OR REPLACEMENT BRIDGES (Continued)		
Deeper Placement of Piles and Footings	S	55*, 282*, 261*, 64
	U	
Lining Bridge Opening	S	123*
	U	
Increased Bridge Length	S	57*, 105, 107B
	U	3A
Raising Bridge Superstructure	S	57*, 71*, 40, 105, 107B
	U	3A
Relocation of Bridge	S	105
	U	

Table 21(b). CASE HISTORIES WITH AGGRADATION COUNTERMEASURES

COUNTERMEASURES	RESULTS	CASE HISTORY NUMBERS
	SPUR DIKES	
Earth or Rock Embankment	S	57B*, 77A
	U	
	DROP STRUCTURES	
Drop Culverts	S	79, 77A
	U	
	BRIDGE OR CHANNEL MODIFICATIONS	
Addition to Spans To Bridge	S	
	U	77C
Dredging and/or Realignment of Approach Channel	S	77A
	U	266*, 78, 109

Table 21(b). CASE HISTORIES WITH AGGRADATION COUNTERMEASURES

COUNTERMEASURES	RESULTS	CASE HISTORY NUMBERS
COUNTERMEASURES BUILT INTO NEW OR REPLACEMENT BRIDGES		
Increased Bridge Length	S	57*, 105, 107B
	U	3A
Raising Bridge Superstructure	S	57*, 71*, 40, 105, 107B
	U	3A
Relocation of Bridge	S	105
	U	

Table 21(c). CASE HISTORIES WITH LATERAL EROSION COUNTERMEASURES

COUNTERMEASURES	RESULT	CASE HISTORY NUMBERS
FLEXIBLE REVETMENT		
Riprap (Rock)	S	37*, 74*, 92*, 115*, 120*, 126*, 151*, 153*, 157*, 169*, 171*, 180*, 192*, 193*, 211*, 213A*, 248*, 282*
	U	52*, 131*, 157C*, 173*, 174*, 176*, 177*, 223*, 236*, 243*, 258*, 281*, 14
Railbank	S	55*, 56*, 85*
	U	
Gabion	S	74B*
	U	
Car Bodies	S	216A*, 224*
	U	
Planted Vegetation	S	
	U	37*

Table 21(c). CASE HISTORIES WITH LATERAL EROSION COUNTERMEASURES (Continued)

COUNTERMEASURES	RESULTS	CASE HISTORY NUMBERS
RIGID REVETMENT		
Concrete Pavement	S	151A*, 152A*, 178*
	U	183C*
Grouted Riprap	S	75B*
	U	
Bulkheads	S	63B*, 173*, 243*, 247*
	U	224*
IMPERMEABLE SPURS		
Rock or Earth Embankment	S	92*, 151*, 243A*, 253*, 257*, 258L*
	U	181*
Sheet Pile	S	175*
	U	
PERMEABLE SPURS		
Timber Pile	S	63B*, 209C*, 220*, 225L*
	U	13*, 174*, 225C*
Sheet Pile, Pipe, or Rail	S	178*, 179*, 180L*, 207*, 209L*, 210A*, 241*, 283*
	U	180C*, 242*
IMPERMEABLE RETARDS		
Rock or Earth Embankment	S	226*, 227*
	U	
Timber Pile	S	192*, 211A*
	U	
Sheet Pile, Pipe, or Rail	S	75B*
	U	

Table 21(c). CASE HISTORIES WITH LATERAL EROSION COUNTERMEASURES (Continued)

COUNTERMEASURES	RESULT	CASE HISTORY NUMBERS
PERMEABLE RETARDS		
Timber Pile	S	63B*, 115*, 171M*, 208A*, 211A*, 216A*, 239M*, 14
	U	
Steel Pile, Pipe, or rail	S	180L*
	U	180C*
Jack or Tetrahedron	S	73*, 75B*, 120*, 133*, 175*, 222*
	U	281*
DIKES		
Rock or Earth Embankment	S	56L*, 74B*, 92*, 126*, 151*, 180*, 243A*
	U	56C*, 157*
SPUR DIKES		
Reveted Rock or Earth Embankment	S	115*, 151*, 152*, 192*, 257*
	U	193*
Rock or Earth Embankment Not Reveted	S	180*
	U	
BRIDGE OR CHANNEL MODIFICATIONS		
Addition of Spans to Bridge	S	
	U	148*, 193*
Dredging or Realignment of Approach Channel	S	208A*, 213A*, 248*
	U	192*, 226*
COUNTERMEASURES BUILT INTO NEW OR REPLACEMENT BRIDGES		
Realignment of Channel	S	157B*
	U	
Increasing Bridge Width	S	
	U	73*, 120*

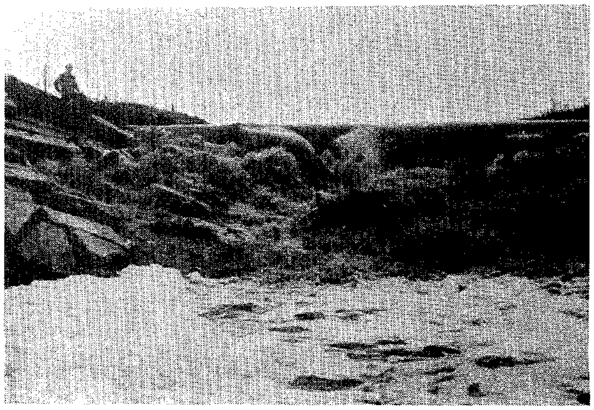


Figure 61. SHEET PILE CHECK DAM –
MOSQUITO CREEK, IA.

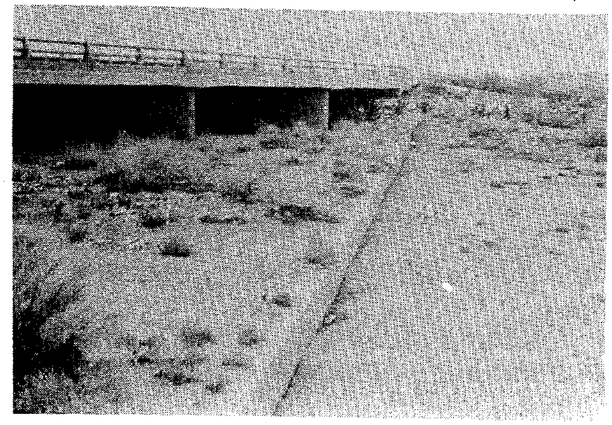


Figure 63. CONCRETE CHECK DAM –
RATTLESNAKE WASH, KINGMAN, ARIZ.

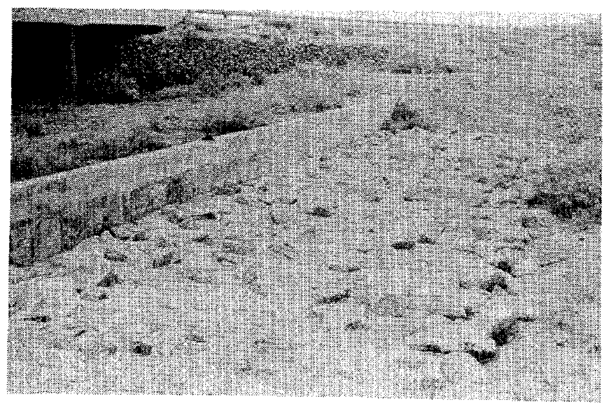


Figure 64. CONCRETE CHECK DAM –
FRIES WASH, KINGMAN, ARIZ.

Figure 62. SHEET PILE CHECK DAM –
PERRY CREEK, MISS.

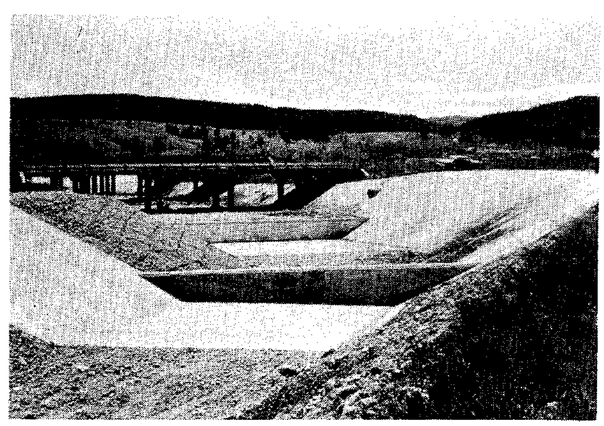


Figure 65. CONCRETE CHECK DAM –
BEAR BUTTE RIVER, S. DAK.

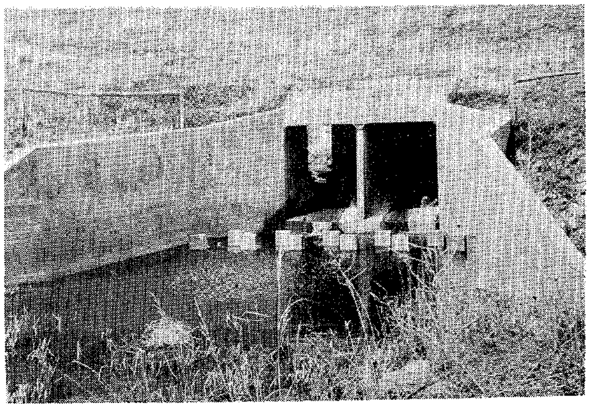


Figure 66. DROP CULVERT AT HIGHWAY 24 NEAR FORT YATES, N. DAK.

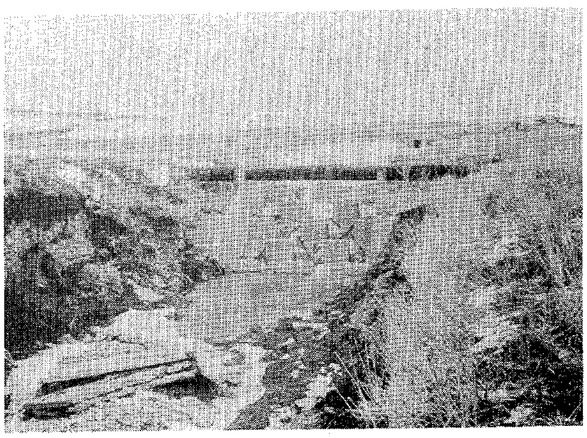


Figure 67. DROP FLUME – UNNAMED DRAW, KAYCEE, WY.

County, Iowa. Water flowing over this structure experiences a 3.5 m drop from the crest of the dam to the stilling pool below. To dissipate the energy produced by the drop, the channel banks and bed are lined with 1000 to 3000 kg of derrick stone. (At low flow it was observed that some of these stones have been dislodged.) Figure 62 shows a smaller sheet pile check dam using limestone riprap to provide energy dissipation downstream and protect the banks. Figures 63 and 64 document the use of concrete drop structures. Note that the drop in Figure 63 does not

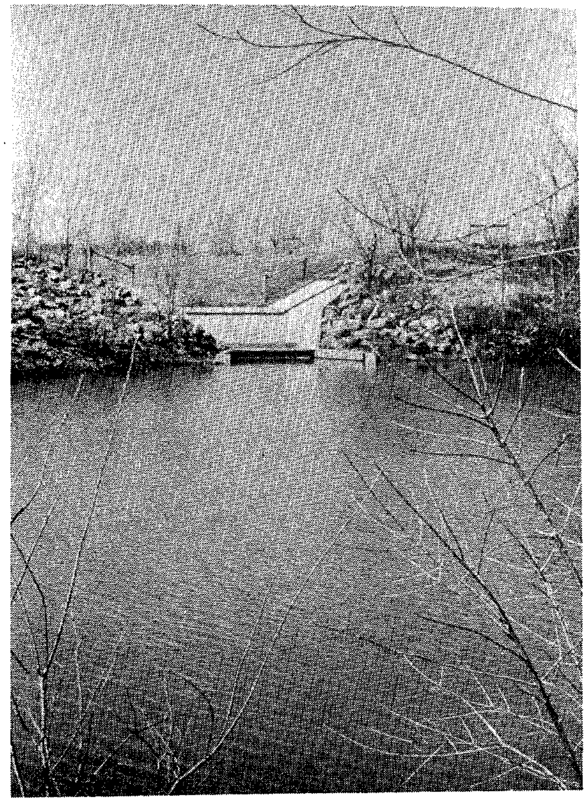


Figure 68. DROP FLUME – DELAWARE RIVER, KAN.

require an artificial means of energy dissipation just downstream of the drop while that shown in Figure 64 does. The two concrete drop structures shown in Figure 65 were constructed to protect the I-90 crossings over Bear Butte River in South Dakota. Two structures were used there to reduce the energy produced by a single drop. As of the date of this photograph, the structure has not been put to any significant test.

Structures similar in purpose to check dams include drop culverts and drop flumes. A typical drop culvert is shown in Figure 66. Figures 67 and 68 document the application of drop flumes.

The use of check dams was the most successful countermeasure. Of the 19 case histories documenting the use of check dams, 17 were successful; the two unsuccessful uses were underdesigned. Check dams have been extensively used in Iowa, Utah, and Arkansas, and cases were also documented in Idaho, Kentucky, and Tennessee. All cases documenting the successful use of check dams are on small or moderate streams having widths not greater than 100 m. However, check dams could conceivably be built on rivers of any size. Low head type drops (less than 1 m) (Figure 62) are the most common, primarily because of the cost involved in constructing a larger structure across a river. Check dams have been constructed on nonbraided streams as well as on streams with all degrees of braiding, with channel beds of sand, gravel, cobbles, and combinations of these.

In general, the type of construction material used to construct the structure depends on the availability of materials, the height of drop required, and the width of the channel. Rock riprap and timber pile construction were most successful on channels having small drops and widths less than 30 m. The use of sheet pile, gabion, and concrete structures was recorded on channels requiring larger drops and with widths ranging up to 100 m.

If not designed properly, check dams can initiate erosion problems on their own. These problems include lateral erosion of the channel banks and degradation of the channel bed downstream of the structure (Figure 69). Channel bed degradation is a result of excess energy dissipation evident in the turbulence at the drop. Depending on the erodibility of bed materials, the local scour produced has the potential to undermine the dam, causing its failure. Energy dissipators in connection with the drop structure can reduce the energy available to erode the channel bed. Another alternative is to construct several drops in series to reduce the energy available at each drop below that required to produce erosion (Figure 65). Experience in Utah with these low head drop structures has shown that they can be successful, and they reportedly allow greater fish movement.

Lateral erosion at the channel banks just downstream of drop structures is also a result of the turbulence produced by energy dissipation at the drop. Bank erosion comes from three sources: the dissipation of energy at the channel banks, bank slumping from local channel degradation, and eddy action at the banks. Besides causing structural problems at the check dams, this bank migration has led

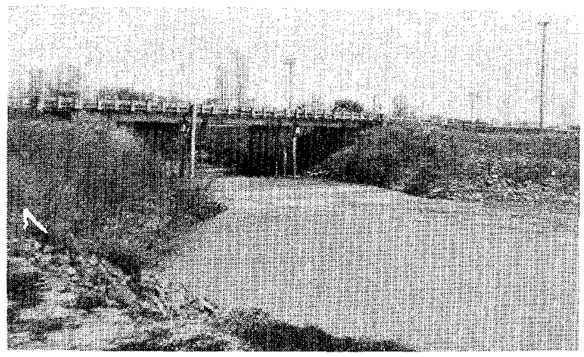


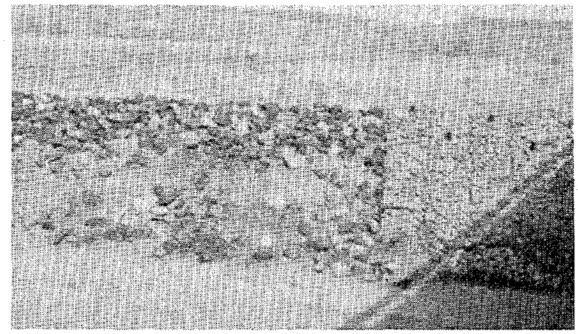
Figure 69. OVERFALL AT CHECK DAM AND RIPRAP ON ERODED BANKS BELOW DAM

to the erosion of approach embankments and abutment foundations in several cases. The logical solution to these problems is to increase the resistance of the bank to such erosion by lining it with properly sized riprap or some other revetment (analysis of various types of revetment bank protection is presented later.)

Another technique used in an attempt to halt channel degradation was to line the channel in the vicinity of the crossing. Channel lining was attempted with both concrete and riprap liners but proved unsuccessful in all cases. Its failure was a result of undermining of the protection. To protect the lining, a check dam would have to be placed at the downstream end to key it to the channel bed. Such a scheme would provide no more protection than would a check dam alone. The channel lining is unnecessary since it provides redundant protection (as documented in Case History No. 197).

Countermeasures To Minimize the Effects of Degradation

Countermeasures to minimize the effects of degradation at crossing sites are defined as those protection schemes that are designed, not to stop channel degradation, but to protect the crossing itself from the impacts of the degradation. This type countermeasure is the one most often used at degrading sites. In general, two changes occur with respect to channel geometry at crossing sites when a degradation profile passes through: first, the base level drops and then, because of the increased bank slope produced by the drop, the banks recede and the overall channel width increases. These changes impact bridges by undermining piers and abutments. The countermeasures used to protect the bridge abutments include various types of revetments and retards; to protect piers, riprap linings are most often used. Also included in this category are bridge modifications such as underpining and jacketing of piers and abutments, driving piles to greater depths, and constructing deeper footings. The success records of these countermeasures reflects mixed results with 18 successes and 26 failures. Here again, the care taken or the level of technology used in design of the countermeasure had more of an impact on its success then did the use of a particular countermeasure.

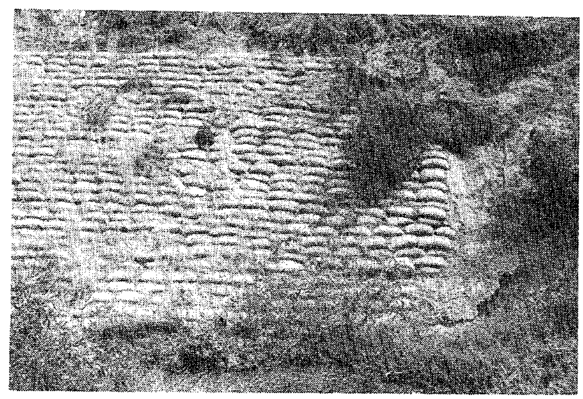


Source: FHWA/RD-78/162

Figure 70. DUMPED ROCK RIPRAP (LEFT); ROCK-AND-WIRE MATTRESS (RIGHT)

Revetment

The types of revetment used to protect abutments from the bank sloughing associated with degradation include riprap (Figure 70), sacked concrete (Figure 71), rock-and-wire mattresses (Figure 72), concrete pavement (Figure 73), and grouted riprap (Figure 74). The success records for these countermeasures were mixed. The most common cause of failure was undermining of the revetment base at the point at which it interfaces with the channel (Figure 73). As the channel bed drops, the revetment, if flexible, slumps into the channel and washes away; if the revetment is rigid, the undermining removes portions of the soil base beneath it causing it to crack, collapse, and break away from the abutment. These actions essentially leave the abutment slopes unprotected.



Source: FHWA/RD-78/162

Figure 71. SACKED CONCRETE REVETMENT SLIGHTLY UNDERCUT AT UPSTREAM (RIGHT) END, SR-85 AT STEVENS CREEK, CAL.

Rock riprap and sacked concrete have provided protection against bank sloughing only when (a) both the abutment and channel banks are on slopes less than the materials angle of repose and (b) the riprap is adequately keyed into both channel bed and banks. The key depth along the channel must be some minimum distance, m, below the expected total depth of scour as shown in Figure 75. This total depth of scour is made up of the sum total of degradation, general scour, dynamic scour, and local scour. The revetment should also be keyed into the banks, forming a wide arch around the abutment, for a distance sufficient to remain keyed into the bank after the degradation-induced bank sloughing has reached its maximum lateral extent. In addition, filter blankets should be used where needed as specified in HEC-11 (1970).

The use of rock-and-wire mattresses was documented in only two cases but was found to be quite effective in providing slope stability at abutments on small ($<$ 30 m) channels. The cases referred to were in arid regions at crossings having bank slopes too steep to hold available riprap. Wire enclosed rock mattresses were used to provide the required stability. In Case History No. 277, the mattress was

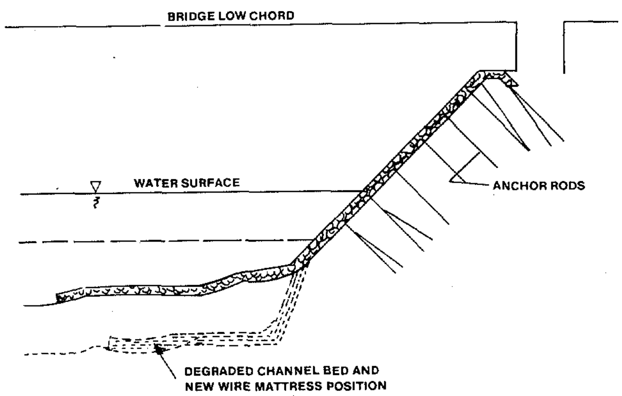
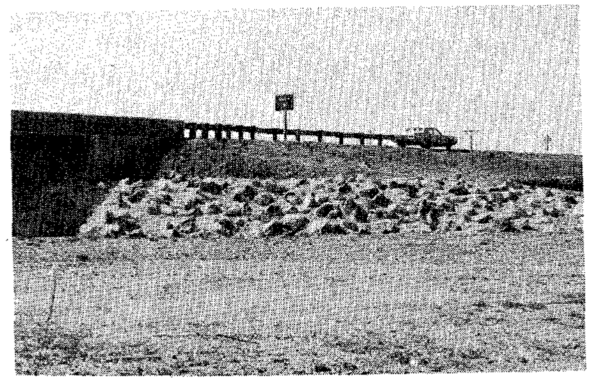
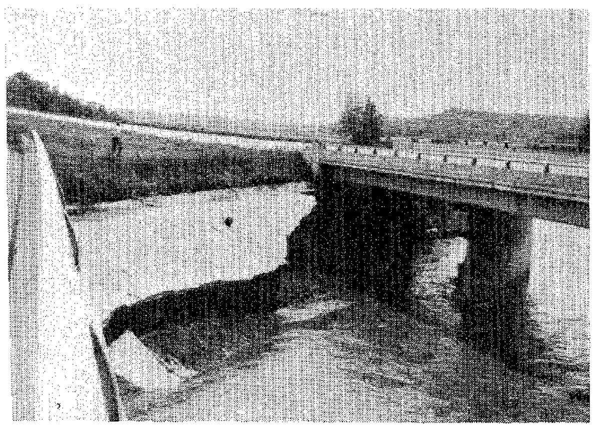


Figure 72. ROCK-AND-WIRE MATTRESS CONSTRUCTION



Source: FHWA/RD-78/162

Figure 74. CONCRETE GROUTED RIPRAP PROTECTING ABUTMENT, STONY CREEK, CAL.



Source: FHWA/RD-78/163

Figure 73. UNDERMINING OF CONCRETE PAVEMENT REVETMENT

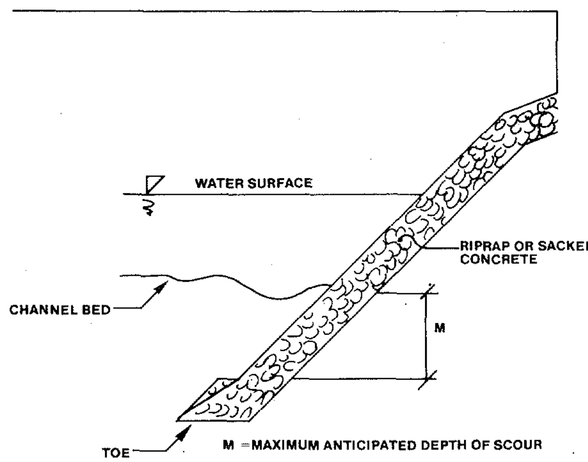


Figure 75. KEY DEPTH FOR REVETMENT CONSTRUCTION

used to protect against headcutting, and in Case History No. 278, it was used to protect against general channel lowering. (Both these case histories are documented in FHWA-RD-78-163.) To protect against the anticipated degradation, the mattress was extended along the channel bed for a distance slightly greater than the anticipated maximum degradation depth. As the degradation passed through the crossing, the mattress draped over the newly cut banks and provided the necessary resistance against bank sloughing and channel widening to adequately protect the streambank and abutments at the crossing (Figure 73). It is important that the mattress be well anchored at the top of the bank and down the slope to resist movement of the entire mattress as a unit.

Rock-and-wire mattresses are recommended for use on small channels experiencing small amounts of degradation but little or no lateral movement from meandering. They are particularly suited for use on spill through abutments where the fill slopes tend to encroach on the channel. Also, experience of the

Wyoming State Highway Department indicates that woven wire mesh should be used instead of welded wire because the welds have a tendency to break.

In one case, vegetation of the bank was used unsuccessfully as a countermeasure. Since revegetation provides no protection below the waterline it provided little protection against bank sloughing at the abutment.

Rigid revetments used include concrete pavement, grouted riprap, bulkheads, and concrete-filled fabric mats. One case of the use of filled fabric mats was documented as successful, but it has not been in place long enough to provide conclusive evidence of this success. Of the four cases reporting the use of concrete pavement, only one was successful. Again, the unsuccessful countermeasures were designed without keying or with inadequate keying. The failure mechanism in all cases was undermining either at the toe of the slope or at either end of the revetment. The one successful case was constructed with a cutoff wall at its toe to a depth sufficient to prevent undermining.

Several cases documented the use of bulkheads and bulkheads in combination with flexible revetment to protect abutments on piers outside the main low flow channel. An example of this later case is shown in Figure 76. That figure also shows the recommended arch type construction for proper keying to prevent outflanking. The arch should extend around the pier or abutment at its sides to provide protection against the maximum expected slumping. Both cases documenting the use of bulkheads were successful. No other revetment is likely to be as effective against the slumping associated with channel degradation as are bulkheads unless the embankment is graded to a low angle; in this latter case, keyed riprap revetment would be at least as effective. Combination of bulkheads and riprap revetment provide protection against lateral erosion as well as any surface erosion that might occur during discharges sufficient to inundate the bank above the level of the bulkhead. Where banks are on a grade sufficiently low to hold the riprap, this technique allows the construction of lower bulkheads which will provide a lower-cost solution (Figure 76). Such low-level bulkheads or cutoff walls can be used without slope revetment where vegetation of the bank provides sufficient resistance against surface erosion.

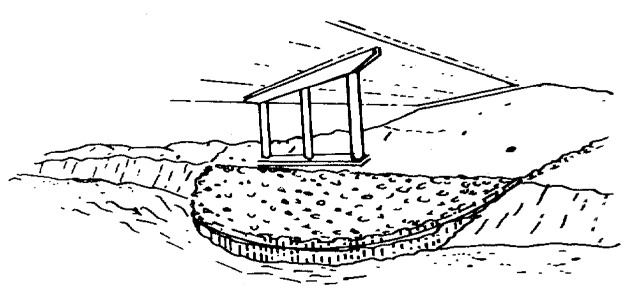


Figure 76. TYPICAL BULKHEAD CONSTRUCTION

As documented by the California Division of Highways (1970), failure of bulkheads is generally attributed to loss of foundation support. This loss can come by undermining or outflanking at the ends. As with revetments, bulkheads must be driven to depths that provide protection below the maximum depth of degradation as defined earlier.

Pier Protection

Another means for minimizing the impacts produced by general stream degradation is to provide additional pier protection. Countermeasures applied to existing bridges to increase pier stability include, underpinning or jacketing, increasing the depth of pier and abutments foundations, driving the piles deeper, and revetting foundations with riprap. These countermeasures have been used to protect existing bridge piers and have been incorporated into new bridge design as well.

The case history data base provided 12 cases documenting the application of a countermeasure to protect piers from channel degradation. A number of these concerned themselves only with the reduction of local scour problems, but enough cases were available documenting general channel degradation to establish guidelines for pier protection. It was found that while riprap (grouted or dumped) designed to currently recommended specifications provides a successful countermeasure against damage from local scour, it does not provide a successful degree of protection against general degradation. To successfully protect piers from being undermined as a channel degrades, the piers must be lengthened by providing a deeper foundation or by driving down additional, deeper piles. Jacketing piers having concrete foundations with steel casings or sheet piles has had limited success.

Countermeasures Built Into New or Replacement Bridges

Countermeasures built into new or replacement bridges to protect them against degradation include the use of drop structures (drop inlets at culverts), fewer or no piers in the channel, deeper foundations or piles, wider flow areas between abutments, and channel linings. In general, in the documented cases, all of these countermeasures have proven successful. The use of channel linings has met with mixed success and is recommended only in connection with check dams and when deemed necessary for additional channel bed protection.

The most economical solution to the degradation problem on small bridges is to widen the span between abutments, placing the abutments in an area that is not susceptible to undermining from bank sloughing and that requires no pier supports. On wider channels where placing the abutments outside susceptible areas necessitates the use of pier supports, a minimum number of these supports should be used and all foundations should be placed to depths below the maximum expected degradation depth.

Conclusions

The following is a condensed list of guidelines for the application of countermeasures at crossings experiencing degradation:

- The most successful technique for halting degradation at a site is the use of check dams or drop structures. Check dams can be economically designed for use on small to medium sized channels. When designing these structures, care must be taken not to promote degradation downstream.
- The use of channel linings has not proved a successful countermeasure to degradation problems unless combined with the use of a check dam, in which case it usually only provides redundant protection.
- Where abutment fill slopes are mild, properly keyed rock riprap provides sufficient protection aginst bank slumping. On steeper slopes, concrete paving has proved successful except where internal slope failures could occur. In such cases bulkheads provide the best protection.
- Rock-and-wire mattresses are recommended for use only on small ($<$ 30 m) channels experiencing little or no lateral movement from meandering or other forms of lateral instability.
- Combinations of bulkheads and riprap revetment have been used to successfully protect abutments where stream banks are characterized by steep cuts against mild abutment fill slopes.
- Riprap while providing a degree of protection against local scour problems at piers, does not provide an adequate degree of protection against general channel degradation.
- Successful pier protection involves providing deeper foundations at piers and pile.
- The most economical solution to degradation problems at new crossing sites on small to medium sized ($<$ 100 m) bridges is to minimize the number of piers in the flow channel and provide adequate foundation depths.

AGGRADATION

Aggradation problems at crossing sites occurred in 11 of the reported case histories. Most of these cases were documented on perennial streams where reservoir backwater induced deposition at highway crossings and in cases in which crossings were located on alluvial fans. Figures 6, 7, 21, and 27 document extensively clogged crossings. Current attempts to alleviate aggradation problems include channelization, bridge modifications, continued maintenance, or combinations of these. These countermeasures have met with mixed success. The success or failure of these projects has been more dependent on the degree of engineering analysis that went into the design of the project than on the type of countermeasures used.

Extensive channelization projects have been undertaken to alleviate aggradation conditions at crossings as documented in Case History No. 266 of Report No. FHWA/RD-78/163 and Case Histories 77, 78, and 109 contained in Appendix A of this report. These channelization projects have included dredging and clearing poorly defined drainage canals, constructing cutoffs to increase the local slope, constructing flow control structures to reduce and control the local channel width, and constructing relief channels to improve flow capacity at the crossing. All but the latter are designed to increase the sediment-carrying capacity of the channel, thus, reducing or eliminating the aggradation problem.

Bridge modifications are common countermeasures to aggradation problems. The most common changes documented are increasing the bridge length by the addition of spans and increasing the effective flow area beneath the structure by raising the bridge deck. In extreme cases, entire bridges have been relocated to reaches experiencing a lesser degree of aggradation (as in Case History No. 105 of Report FHWA/RD-78/163). Examples of bridge alterations are documented in Case Histories No. 57 and 71 of Report FHWA/RD-78/163 and 3, 90, 105, and 107 from Appendix A of this report.

Another countermeasure that has been successfully used to control aggradation problems at bridge crossing is a program of continued maintenance. In such a program, a monitoring system is set up to survey the affected crossing at regular intervals; when some preestablished deposition depth is reached, the bridge opening is dredged or cleared of the deposited material. In some cases this would require opening a clearing after every major storm. This solution is only a temporary one and is not recommended if analysis shows the problem to be permanent and continuous. An adequate evaluation of sediment supply and transport characteristics in the local reach as well as the entire system will provide the information necessary to estimate expected sediment load characteristics to determine whether a particular problem is temporary or permanent.

Maintenance programs prove to be very cost effective when compared, over the short term, with the high cost of channelization, bridge alterations, or relocations. In today's economy with the extreme pressure to hold down spending, it becomes tempting to use a maintenance program in place of other, more expensive countermeasures. In the long term, however, maintenance programs usually eventually cost more than some of the initially more-expensive measures. There are several reasons for this. First, a continued maintenance program is actually quite expensive when a cost analysis is run over the entire life of the structure. Second, information from several state highway engineers indicates that the reliability of maintenance programs is very low; such programs are often abandoned or forgotten after several years. With neglect, the aggradation problem can become bad enough to eventually require the construction of a new structure, thus adding the cost of construction to several years of continued maintenance.

However, if analysis of the transport characteristics and sediment supply in a river system reveals that the aggradation problem is only temporary (perhaps the excess sediment supply is coming from a construction site) or will have only minor effects over relatively long periods of time, a regular program of maintenance could prove the best, most cost efficient solution.

An alternative related to the maintenance program, which could be used on sand and gravel streams with persistent aggradation problems (such as those on alluvial fans), is the use of controlled sand/gravel mining from a debris basin constructed upstream of the bridge site. Use of such a system would require careful engineering analysis to ensure that the gravel mining did not upset the delicate balance of sediment and water discharges downstream of the debris basin. The rights to mine the sand and/or gravel could be leased out and become a source of income. As a precaution, a detailed analysis of the river system with respect to water and sediment would be necessary to ensure that the natural balance was maintained within the system; excess mining would produce a degradation profile downstream potentially impacting the bridge or other structures.

Aggradation problems are extremely difficult to control and the success or failure of the countermeasure is mainly dependent on the amount of analysis that goes into its design. This fact is exemplified in Case History No. 77 (in Appendix A) on the Arroyo Seco at US Highway 84 near Espanola, New Mexico (Figure 77). Initially this crossing consisted of three concrete box culverts. Between 1954 and 1967, there was a continual problem with aggradation at the site and the southmost section filled with sediment to 90 percent of its volume. Countermeasures were then undertaken to alleviate the associated flooding problems without adequately analyzing the sediment availability and transport characteristics within the system. Four additional concrete box culvert sections were added at the crossing to increase the available flow area, but that provided only a temporary solution to the problem; by 1972 the new culvert sections had also filled with sediment. At that time, an analysis of the system's sediment transport characteristics was undertaken and a new design scheme was implemented. The channel was reworked with a change in stream alignment to increase the local slope; rail bank and dikes were also added to confine the flow to a smaller channel (Figure 77). The result of these measures was to increase the transport capacity in local reach, keeping it clear of sediment. This new configuration has worked successfully to date. Its success indicates the importance of conducting an analysis of sediment transport characteristics within the reach to better define a solution to a grade change problem.

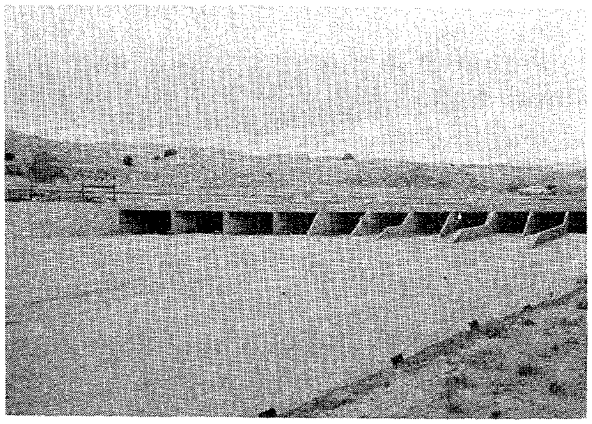


Figure 77. ARROYO SECO AT US-84 NEAR ESPANOLA, N.M.

The most successful countermeasures for controlling aggradation include continual maintenance, channelization, debris basins, and bridge alterations. Because several schemes are available for countering a given problem, a comparison of costs is necessary to produce the most cost-effective solution.

The cost of a specific control scheme varies because of the uniqueness of river systems, their sediment problems and the solutions to those problems, the availability of construction materials locally, etc. To aid the highway engineer in the choice of the most cost-effective countermeasure for a given problem, a comparison of the relative costs of the four most successful countermeasures was undertaken. Relative cost data were obtained from engineers familiar with the construction of such schemes and were used to construct the bar chart given in Figure 78.

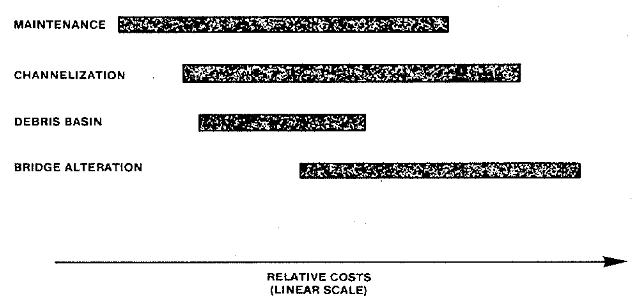


Figure 78. RELATIVE COST OF COUNTERMEASURES

Figure 78 compares the relative costs of the four recommended aggradation countermeasures. Considering the leftmost end of each of the bars, the cost ranking of the measures follows the order of maintenace, channelization, debris basins/gravel mining, bridge alterations. This order represents their cost-effectiveness on small channels with only minor aggradation problems. In that case maintenance would be the best solution. As aggradation conditions become more varied and severe on larger channels (to the right on the figure), any one of the countermeasures might prove to be more cost effective than any other and only adequate analysis of local conditions will provide the 'best' solution scheme.

The following is a list of guidelines regarding aggradation countermeasures:

- Extensive channelization projects have generally proven unsuccessful in alleviating general aggradation problems although some successful cases have been documented. These measures usually do not provide a sufficient increase in the sediment-carrying capacity at the crossing to eliminate or significantly reduce the problem. Channelization projects should only be used after analysis shows they will produce the desired results.
- Alterations to a bridge and its replacement are very expensive but are often required to provide sufficient area beneath a structure to accommodate maximum aggradation depths.
- Although maintenance programs have proved unreliable in the past, with proper supervision they would provide the most cost-effective solution where the aggradation is from a temporary source or on small channels where the problem is limited in magnitude.
- At aggrading sites on wide, shallow streams and rivers, the use of dikes lined with some form of flexible revetment to confine flow to a narrower, deeper section has proven successful in several cases.
- At crossings with severe problems, such as those on alluvial fans, the best solution might be the construction of a debris basin in combiantion with controlled sand and/or gravel mining.

LATERAL EROSION CONTROL

The case history data base provided 121 specific examples of countermeasures used to control lateral erosion problems resulting from aggradation and degradation. The case history numbers are listed in Table 21(c). The countermeasures documented include flexible and rigid revetments, impermeable and permeable spurs, impermeable and permeable retards, dikes, spur dikes, bridge or channel modifications, and countermeasures built into new or replacement bridges. Of the 121 cases reported, 88 provided successful channel control.

Revetments

Revetments provide erosion-resistant surfaces for banks or embankments. Revetments are also often used as linings at flow-control structures such as dikes, spur dikes, and spurs. The types of flexible revetments evaluated include rock riprap, railbank, gabion, car bodies, and planted vegetation.

Of the 38 case histories reporting the use of flexible revetment for the control of lateral erosion at highway crossings, 24 were successful and 14 were reported as unsuccessful. All cases documented were on perennial streams. Bed materials were predominently sand, sand/silt, and gravel with some cobble/boulder cases. Valley slopes ranged from 0.008 to 0.00012, with channel widths ranging from 10 m to 370 m. A few more than half the crossing sites studied were on braided streams, while the others were on sinuous to highly meandering streams.

A form of rigid revetment was used as the lateral erosion countermeasure in 10 of the cases studied. Of the three types of rigid revetment, concrete pavement and bulkheads were the most frequently used. In most cases these revetment types were used to form solid protective barriers around abutments. The high cost of rigid revetments prohibits their use in general bank protection although they have been used successfully to line flow control/alignment dikes in several cases reported.

Revetments, both rigid and flexible, provide an acceptable degree of protection against lateral instability as long as the protection schemes are adequately designed. The most common source of failure of a revetment system comes from undermining or lateral outflanking. When designing such a countermeasure, care must be taken to provide adequate coverage laterally as well as sufficient toe protection to prevent undermining. Another factor often neglected in revetment design is the influence produced by ice and debris loadings; lateral forces created when ice and debris impact an embankment often have been seen to dislodge sections of revetmented systems initiating failure. Rigid revetments have provided greater levels of safety in areas subjected to excessive debris and ice loading conditions.

The type of revetment to be used for a particular case will depend to a large part on its availability and cost at a particular site. If riprap of adequate size is not available, smaller stones can be used in woven wire mattresses or gabions. Plating of riprap increases its stability and is recommended where excess debris or ice loads are expected. Concrete paving also provides sufficient resistance to debris and ice loads but is not recommended in regions susceptible to frost heaving or bank subsidence since its poor tensile strength could lead to failure under those conditions. It is also not recommended for use in streams characterized by high flow velocities.

Some general conclusions about the use of revetments for control of lateral instability are:

- Flexible revetment, of which rock riprap is the most widely used type, has a better record of performance than does rigid revetment, of which concrete pavement is the most common type.
- Rock-and-wire mattresses (gabion) have proven successful where adequately sized riprap is not available and where streambanks are high and steep.
- Plating of riprap increases its stability and resistance to debris and ice loads.
- Concrete pavement provides a high degree of safety against debris and ice loads but is not recommended on channels characterized by high flow velocities.
- Rigid revetments are not recommended at sites where frost heaving and/or bank subsidence are common.
- Bulkheads are advantageous for control of internal slope failures where the slope cannot be graded to a low enough angle for the placement of other revetment.
- All revetment types must be designed to adequately protect against undermining and provide sufficient lateral coverage to avoid outflanking of the protection.

Retards

Both impermeable and permeable retards were documented in this study. Of the 22 cases in which retards were reportedly used to control lateral instability, 20 were classified as successful and 2 as unsuccessful. All case histories reported were on perennial, alluvial, sand bed channels with various degrees of sinuosity. Drainage areas ranged from 4 sq km to 7,290 sq km, with most being less than 200 sq km. Valley slopes ranged from 0.0003 to 0.0031, and channel widths from 25 m to 370 m although most channels were less than 100 m wide.

Permeable retards included timber pile, steel pile, pipe, or rail, and jack or tetrahedron field construction. Impermeable retards included rock or earth embankment, timber pile, and sheet pile or steel construction. Densities of impermeable retards varied. Although no research to date has investigated the appropriate degree of permeability for retards, review of the case histories reveals that the required permeability of a retard is inversely proportional to the radius of curvature of the channel bend being controlled; the sharper the bend, the greater the required density of retard needed to control the lateral erosion problem.

A significant factor affecting the success of retards is the availability or supply of sufficient bedload material to be deposited behind the retard. Such a supply would help reestablish some earlier bank line and promote the development of bank vegetation further aiding bank stability. This process has also been seen to enhance the structural stability of the retard itself.

The success of retards is a function of the care taken in their design and construction as with other countermeasures. Consideration must be given to local scour depths and potential impact loads from floating debris and ice when designing foundation depths. Construction practices must follow design specifications if the structure is to function properly. Rock or earth embankment and timber pile retards are the most commonly used and most esthetically acceptable.

In general, the three most important factors to be considered when contemplating the use of retards for the control of lateral erosion problems are the potential for channel-bank revegetation, the availability of bedload material for deposition, and the degree of retard permeability. If the basin under consideration is in an arid region and not conducive to natural or planned revegetation, permeable retards would provide only limited success; if there is not a significant supply of bedload material for deposition behind the retard, the chances of the successful use of permeable retards is minimal; also, the use of too permeable a retard for a given degree of bend curvature would produce an inadequate design. Retards have been used successfully for flow alignment and lateral erosion problems on small-to-moderate size channels (widths usually < 100 m) not frequently subjected to flow velocities in excess of 1.5 to 1.8 m/sec.

The general conclusions relating to the use of retards are:

- Retards have been used most successfully to provide flow alignment at crossings.
- Retards have proven to be most successful on channels having
  - small to moderate widths (< 100 m),
  - flow velocities not frequently exceeding 1.5 to 1.8 m/sec, and
  - sand bed channels with relatively large bed and suspended loads.
- Required retard permeability is inversely porportional to the radius of curvature of the bend.
- Retards by themselves preform best in humid regions where natural revegetation of banks aids the stabilization of bank lines.
- Retards, as other countermeasures, are susceptible to damage by ice and debris loads.
- Rock or earth embankment and timber pile retards provide the highest level of esthetic appeal of the retards analyzed.
- Retards in combination with spurs and/or revetment provide additional protection on channels experiencing high velocities and for abrupt flow alignment problems.

Spurs

In this report, a spur is defined as an impermeable or permeable linear structure that projects into a channel from the bank to alter flow direction, induce deposition, or reduce flow velocity along the bank. Alternative terms used by various highway agencies are jetty, groin, dike, deflector, and wing dam.

The use of spurs to control lateral erosion problems was documented at 25 of the case history sites. Nineteen of these cases were successful and six were not [Table 21(c)]. Two cases that were initially documented as being unsuccessful were later modified and have since proven successful. In Case History 63 (Report FHWA/RD-78/163), the use of spurs has proven successful to date, but the spur has not been in place long enough to adequately test its success. Seventeen cases report the use of permeable spurs, while eight document the use of impermeable spurs.

Spurs have been used predominantly for the control of flow around sharp bends or to provide proper alignment at bridge crossings. In both cases they are generally less expensive and more effective than riprap revetment. They have also been used effectively to constrict wide braided or unbraided rivers at crossings to reduce the required bridge opening width. In many respects spurs provide a function similar to that of retards and have often been used in combination with retards to provide the required control.

Spurs are generally more effective than retards where abrupt changes must be provided in flow alignment on large rivers (widths greater than 150 m), or where flow confinement is required, such as on wide braided streams. They also produce a higher level of safety on channels experiencing frequent high-velocity discharges such as on mountain streams or rivers.

Impermeable spurs are constructed of earth or rock embankment or sheet pile. Permeable spurs are of pipe and plank, H-pile with welded wire fabric facing, or timber or steel pile construction. Each of these was documented in the case histories.

Several trends have emerged with respect to the use of either permeable or impermeable spurs. In general, the use of permeable spurs has been confined to flow alignment on small, unbraided channels carrying relatively large bed and suspended sediment loads. They are frequently used in arid regions in combination with retards to provide additional stability at bends. In cases where they are used alone, their function parallels that of retards and their level of success is also quite similar. For flow alignment on small, low-velocity, high-sediment-load streams, spurs and retards are virtually interchangeable and the recommended use of one or the other will depend primarily on local cost and availability of materials as well as the preference of the design engineers. However, permeable spurs function more effectively than retards in high-velocity environments.

Permeable spurs are found to be less effective than impermeable spurs at controlling flow alignment on large channels. Heavy debris and/or ice loads have also been seen to produce greater hazards to permeable spurs than to impermeable ones.

Impermeable spurs have functioned in virtually all cases in which their design was adequate. However, they are best suited to handle cases that require abrupt changes in flow alignment such as at small radius meander bends and for the control of flow alignment on wide braided channels. In this latter case, impermeable spurs are usually combined with a dike system. Impermeable spurs are also more effective than permeable ones on large sand, gravel, and cobble bed channels (where the sediment load is almost entirely bed load).

A summary of general conclusions applying to the use of spurs is as follows:

- Spurs are predominantly used for flow alignment at bridges and bends where they may be less expensive and more effective than riprap revetments.
- Spurs are more effective than retards where abrupt changes are required in flow alignment on large rivers and rivers characterized by high velocity flows.
- Impermeable spurs are best suited to handle cases that require abrupt changes in flow alignment on wide rivers or that require substantial flow constriction such as is the case on wide braided rivers.
- Permeable spurs are best suited to providing flow alignment at bridges and meander bends on small to medium (< 100 m) unbraided channels carrying large bed and suspended sediment loads.
- Impermeable spurs provide a higher level of safety against ice and debris hazards than do permeable spurs.

Dikes and Spur Dikes

Dikes and spur dikes were used at 14 sites, and at all but three, they were successful. Although the primary purpose of spur dikes, is to prevent erosion at abutments from flood flow concentration along the upstream face of approach embankments, they also serve as conventional dikes by confining the flow to a constricted channel at a crossing. In all the cases studied, these dikes were used to prevent flood waters from bypassing a bridge or to confine channel width and maintain channel alignment at bridges. All documented cases were on braided or anabranched systems. Flow control dikes are usually covered with some form of revetment to provide erosion resistance. Of the revetment used, plated riprap was the most successful.

Dikes were used both by themselves and in combination with spurs. For moderated flow constriction on braided streams, a revetted dike provides sufficient channel control. However, in cases in which it is necessary to drastically constrict the flow or on channels exhibiting anabranching or extensive flip-flopping of individual braids, the combination of dikes and spurs provides the best solution.

Again, proper design and construction of all countermeasures is necessary to their proper functioning.

Bridge and Channel Modifications

Bridge and channel modifications undertaken in attempts to protect the structure from damage caused by lateral erosion include the addition of spans to the structure and the dredging or realignment of approach channels. These measures were taken at existing bridges, replacement bridges, and new bridges. Of the ten case histories included in this category, six documented channel realignment and four documented increasing the bridge width to accommodate increased lateral movement.

Channel realignment showed a 67 percent chance of success. However, of the four successful cases, one countermeasure had not been in place long enough to adequately document its success and two others were used in combination with other countermeasures, making it difficult to document the degree of success attributable to the channel changes by themselves. In general, channel realignment is a viable solution only when combined with some other flow control scheme because it does nothing on its own to alter the natural instability of the system.

Increasing the width of bridges as a countermeasure to lateral erosion was documented in four cases: two at new or replacement bridges and two at existing bridges. Again, because increasing the width of such a structure does nothing to alter the general channel instability, all four cases were unsuccessful.

Providing channel realignments or adding spans to bridges are among the most expensive countermeasures documented, and they provide the least successful results. In general, these measures should be avoided unless combined wth additional flow control measures to stabilize the channel.

CONCLUSIONS

Countermeasures for the control of degradation and aggradation and the associated lateral erosion problems at highway crossings were evaluated. The most significant factor found to be influencing the effectiveness of the various control schemes is the level of engineering expertise and analysis that goes into crossing design. The most common cause of failure of countermeasures is undermining and lateral outflanking. It is extremely important that the dynamic nature of river systems be considered in the design. Too often countermeasures are designed with inadequate knowledge of river system response and the basic hydraulic principles and concepts involved in sediment transport. The choice of an appropriate control scheme for a given site should be based on an understanding of the factors influencing the river system under investigation, the results of an adequate stability analysis, and good engineering judgment.

Chapter IX

SUMMARY AND CONCLUSIONS

GENERAL DESCRIPTION OF STUDY

The Sutron Corporation has prepared an assessment of the causes and consequences of aggradation and degradation at highway stream crossings. This study was divided into two phases. The first phase consisted of an analysis of 110 case histories of aggradation and/or degradation at highway crossings to determine the geographic extent, nature, and causes of such problems. Information from this first phase is contained in Report No. FHWA/RD-80/038. The second phase provides highway engineers with techniques and procedures for considering gradation changes in the design of crossings. It also includes means for recognizing and selecting remedial measures for gradation problems at existing bridge sites.

HIGHWAY PROBLEMS RELATED TO GRADATION CHANGES

Gradation problems at highway crossings include aggradation and degradation; lateral erosion problems often occur as a consequence of these changes. The highway problem most associated with *aggradation* is reduction of flow area, which increases bridge backwater. The problem associated with *degradation* is undermining; footings, pile bents, abutments, cutoff walls, and other flow-control or crossing structures can be undermined. Degradation has also been found to undermine bank protection resulting in the instability of channel banks and increasing debris problems. The most common problem associated with lateral erosion is bank slumping, which undermines abutments and piers located near the bank line.

ANALYSIS OF THE CASE HISTORY DATA BASE

Bridge sites suitable for inclusion in the case history data base were obtained from state highway departments. Most of the sites were visited and photographed. Some sites were obtained from reports of other investigators such as Brice, Blodgett, et al. (1978).

Two hundred and seventy-five sites were identified as suitable for this study. One hundred and ten were documented for reasons of economy.

Of the 110 case histories analyzed, aggradation problems were found in 29 cases and degradation in 82 (Case History 22 exhibited both aggradation and degradation); in 15 cases some form of lateral erosion was also found. The causes of the gradation problems were most often found to be such human activities as channel alterations, land use changes, streambed mining/excavation, damming, and reservoir regulation, and construction activities. In some cases aggradation and degradation problems were seen to result from natural causes, the most significant of which was alluvial fan buildup; other natural causes documented included natural armoring, braiding, debris, meandering/migrating, natural cutoffs, recurrent flooding/high stream velocities, and delta growth. Fewer than one case history in five had natural causes contributing to the gradation changes.

Since human activities were found to have the most serious impact on channel stability, it is extremely important that the highway engineer evaluate both the short- and long-term impacts of human activities on each highway crossing for design and maintenance considerations.

REGIONALIZATION OF GRADATION PROBLEMS

Regionalization of gradation problems – recognition that certain aggradation or degradation problems occur most often in specific regions of the country – is one method that provides insight into the detection of such problems or potential problems.

Virtually every U.S. river that flows in an alluvial bed has a potential for gradation change. The prevalance of human activities as the chief cause of gradation problems means that most rivers suffer to some degree. Engineers working in areas with high sediment yield should consider the possibility of gradation problems. Figure 79 shows a correlation between the locations of case histories having gradation problems and the concentration of sediment in

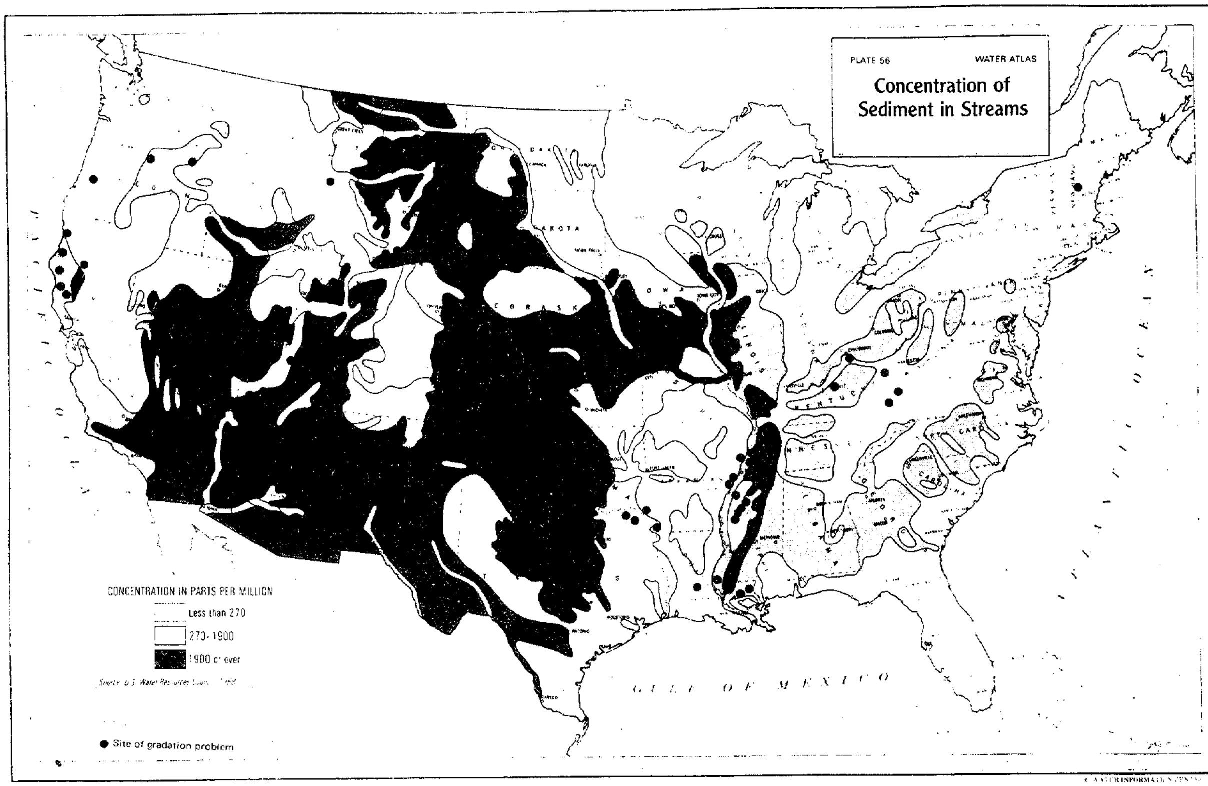


Figure 79. LOCATION OF CASE HISTORY SITES AND SEDIMENT CONCENTRATIONS IN STREAMS

major U.S. streams. Figure 80 locates the case histories with respect ot major river drainage basins within the U.S. The western and central portions of the country from a line along the western borders of Arizona and Montana to the Mississippi River seem to be particularly prone to gradation problems. Significant gradation problems have also been found to be characteristic of the lower reaches of the Missouri River and the Red River.

RELATIONSHIPS BETWEEN GRADATION PROBLEMS AND HIGHWAY CROSSING DESIGN

A review of current design practices was undertaken to evaluate crossing design procedures and the effect of grade changes on these procedures. The parameters most inflenced by grade changes are those used as input to the hydraulic design procedures currently in use. These input parameters include

- design discharge,
- channel roughness,
- energy slope,
- bed slope,
- velocities,
- shear stresses,
- cross-sectional geometry,
- base level,
- flow depth, and
- flow alignment.

Other components of crossing design affected by grade changes include foundation depth, bridge deck clearance, and flow opening size.

Problems encountered at bridge crossings include bridge capacity, backwater, pier and abutment alignment, footing depth at piers and abutments, and construction depth for flow-control and debris-control structures. With respect to bridge capacity and backwater, aggradation produces the most severe problems. However, debris problems associated with degradation can also have a significant impact. Foundation depths for piers, abutments, and

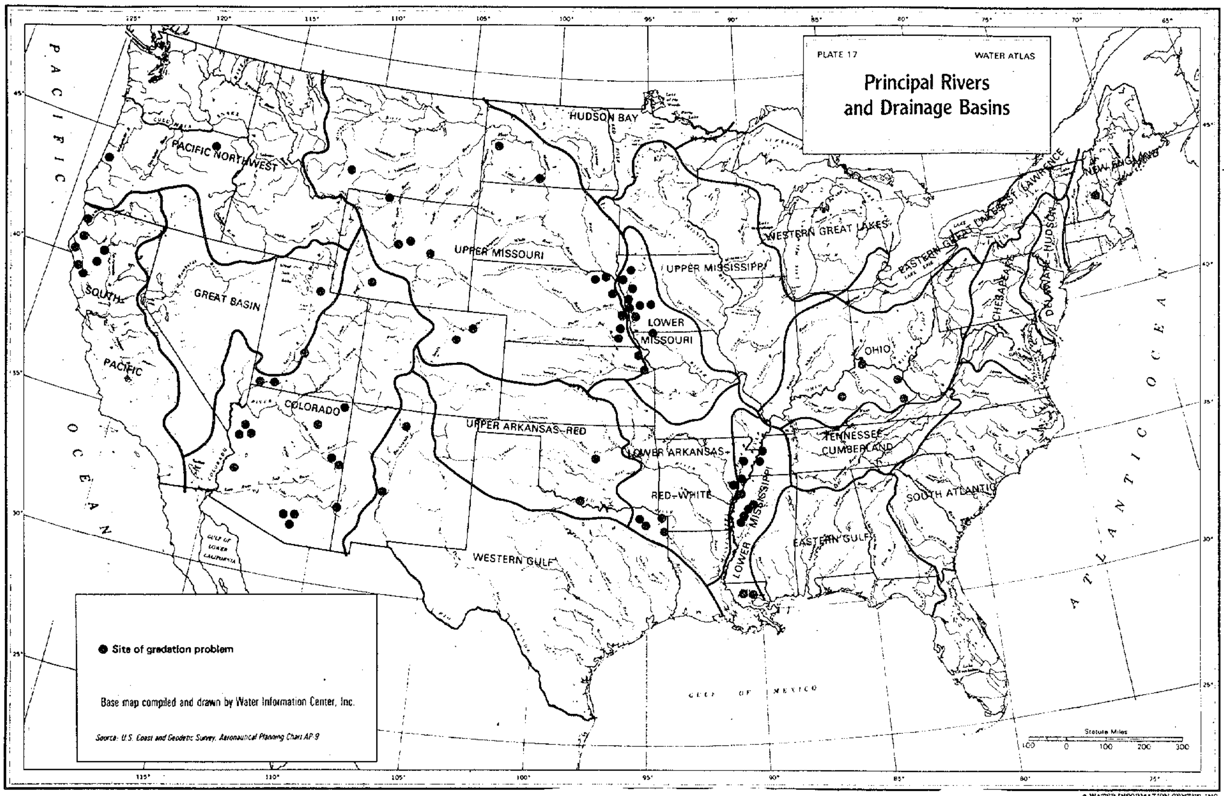


Figure 80. LOCATION OF CASE HISTORY SITES NEAR PRINCIPAL RIVERS AND DRAINAGE BASINS

flow-control structures can be influenced in two ways by grade changes: the normal streambed base level will be altered and the "normal" hydraulic conditions at a site used as input to local scour computations will be changed. Foundation depth problems were encountered in almost all the case histories reviewed for this study in which degradation problems were encountered. The important components of bank protection design adversely affected by grade changes are key depths and the vertical extent of bank protection.

Problems encountered at culvert crossings can be the result of general grade changes produced by long-term changes in stream morphology or inadequate design and/or construction of culvert systems. The design components most often influenced are culvert capacity and structural stability. The greatest danger produced by aggradation is partial plugging of the culvert opening resulting in a damming effect and increasing the magnitude and frequency of flooding upstream of the structure. Degrading stream reaches affect culvert systems by reducing their structural stability. General streambed degradation has been seen to undermine the foundations of culvert systems resulting in their complete failure.

RIVER SYSTEM RESPONSE

The river system, composed of a watershed, channels, and those entities that act on and react to them, is a highly nonlinear, complex system. An alteration to the river environment may bring about numerous subsequent system responses. However, because all rivers are governed by the same basic forces, certain system responses can be categorized. Table 22 gives a listing of man-induced and natural causes of gradation problems and the typical response as documented by the case histories. Note that some activities cause either an aggradation or degradation response.

METHODS FOR DETERMINING GRADE CHANGES

Grade changes in river systems can be predicted

Table 22. RIVER SYSTEM RESPONSE TO CHANGE

PROBLEM CAUSE	RESPONSE	
	AGGRADATION	DEGRADATION
Human Activities		
Channelization/Straightening		X
Clearing/Snagging	X	X
Streambed Mining		X
Daming and Reservoir Regulation		
Upstream	X	
Downstream		X
Land Use Change		
Urbanization and Community Development		X
Agriculture	X	
Construction Activities	X	
Natural Processes		
Cutoffs		X
Alluvial Fan Development	X	
Tectonic Activity	X	X
Climatic Changes	X	X

through the use of various levels of analysis, ranging from simple to extremely complex. The simplest techniques are qualitative in nature, involving the application of geomorphic concepts. The most complex techniques involve the analysis of entire drainage systems using detailed dynamic computer modeling of water and sediment transport processes. Between these two extremes are numerous quantitative techniques involving geomorphic and basic engineering relationships as well as some simpler modeling techniques.

Qualitative geomorphic techniques are primarily based on a well-founded understanding of the watershed and river response. Interpretation of aerial photographs provides valuable information on river hydraulics and channel geometry problems. Evidence of bank cutting, shifting of the thalweg, lateral/meander tendencies, vegetation changes, and sediment deposition can be documented by studying photographs for different years. Other relationships, such as the one presented by Lane (1955), shown in Equation 97, can be used to describe the changes in river morphology caused by modifications of water and sediment discharge.

$$Q S \sim Q_s D_{50} \tag{97}$$

where

Q = water discharge,

S = channel slope,

Q_s = sediment discharge, and

D_{50} = median sediment size.

While qualitative techniques do not provide quantitative answers with regard to aggradation and degradation, they indicate the direction of a grade change and the potential severity of a given problem.

Geomorphic principles can be applied to available data to evaluate gradation problems more accurately. Quantitative geomorphic analysis requires collection and analysis of data over several years.

Analysis of stream gaging station trends provides useful information on long-term trends, such as information on suspended sediment load and shifts in rating curves. Another method for verifying gradation changes is stream profile evaluation. The general properties of a river, such as geometry and alignment, can be estimated using geomorphic channel geometry relationships. Incipient motion considerations can also be used to establish bed and bank stability.

The quantitative magnitude of aggradation or degradation to be expected in the vicinity of the highway crossing can be estimated rather quickly by analyzing the potential bed-material volume changes in the local reach. This analysis requires that sediment transport rates into and out of the local reach be estimated. To make that estimate requires describing the channel geometry, selecting the hydrologic condition of interest, determining the velocities within the reach using a backwater or flow routing model, and then applying one of the recommented sediment transport equations. Knowing the rate at which sediment is transported in and out of the reach will provide the information needed to calculate the change in streambed elevation.

Quantitative techniques are also documented for application to specific situations. These situations include degradation downstream of dams, aggradation upstream of dams, and aggradation in streams caused by overloading. Because each of these techniques was developed for use under a given set of conditions, care must be taken in their applications.

Hydraulic modeling techniques have been developed that reproduce the behavior of rivers and river systems. These models range from methods employing hand calculation procedures aided by small computers or desktop calculators to full-blown dynamic computer models capable of unsteady flow and sediment routing.

COST AND LEVEL OF EFFORT

The cost of any engineering analysis is a function of the level of effort, which, in turn, depends on the accuracy required, the time available, and the analysis techniques applied. The more complicated the analysis, the more accurate the results but the higher the cost. An estimate of the time requirements for the various levels of effort has been made assuming an experienced engineer conducts the analysis. A qualitative analysis based on geomorphic concepts would require 1 to 2 weeks. A quantitative solution based on geomorphic concepts and basic engineering relationships would require 4 to 10 weeks. The various levels of mathematical modeling techniques would require 6 to 20 weeks for completion.

APPLICATION OF METHODS

Because each river system is unique and because the factors that produce gradation changes in these systems are also unique, a "cookbook" approach to the solution of gradation problems is not possible. However, a general outline documenting a recommended approach to the solution of gradation problems is presented.

The first step in the analysis consists of the following three stages:

- obtain background information/data,
- analyze information/data, and
- draw conclusion with respect to stability.

After the potential for or the existence of a grade change problem is confirmed, the bridge engineer must determine the extent or magntiude of the problem. This determination constitutes the second phase in the analysis and consists of the following steps:

- determine the level of analysis,
- acquire additional data,
- assess further stability considerations,
- analyze grade change, and
- analyze other impacting activities.

Once the magnitude of the grade change has been established, the impacts of the change on bridge and countermeasure design can be evaluated. The process involves the following steps:

- evaluate the impacts on crossing design,
- select an appropriate countermeasure, and

- evaluate other impacting factors at the crossing.

Several case histories have been selected for an analysis in sufficient detail to assist individuals with similar problems. These case history analyses provide models or examples of

- data requirements,
- grade change analysis, and
- countermeasures.

CLASSIFICATION AND ANALYSIS OF COUNTERMEASURES

Countermeasures have been evaluated for the control of degradation and aggradation and the associated lateral erosion problems at highway crossings. The most significant factor found to influence the effectiveness of the various control schemes is the level of engineering expertise and analysis that goes into the initial bridge design. The choice of an appropriate control scheme for a given site should be based on an understanding of the factors influencing the river system under investigation, the results of an adequate stability analysis, and good engineering judgment.

The following is a condensed list of recommendations and guidelines for the application of countermeasures at crossings experiencing degradation:

- The most successful technique for halting degradation on small to medium streams is the use of check dams or drop structures.
- The use of channel linings alone has not proved a successful countermeasure against degradation problems.
- On mild abutment fill slopes, properly keyed rock riprap provides sufficient protection against bank slumping. On steeper slopes, concrete paving has proved successful except where internal slope failures could occur.
- Combinations of bulkheads and riprap revetment have been used to successfully protect abutments where streambanks are characterized by steep cuts against mild abutment fill slopes.
- Riprap does not provide an adequate degree of protection against general channel degradation.
- Successful pier protection involves providing deeper foundations at piers and pile bents.
- Jacketing piers with steel casings or sheet piles has also proved successful where expected degradation extends only to the top of the orginal foundation.
- The most economical solution to degradation problems at new crossing sites on small to medium size rivers is to minimize the number of piers in the flow channel and provide adequate foundation depths.

The following is a list of conclusions regarding aggradation countermeasures:

- Extensive channelization projects have generally proven unsuccessful at alleviating general aggradation problems at crossings.
- Maintenance programs can provide the most cost effective solution where the aggradation is from a temporary source or on small channels where the problem is limited in magnitude.
- At crossings with severe problems, such as location on alluvial fans, the best solution might be the construction of a debris basin in combination with controlled sand and/or gravel mining.

Some general conclusions about the use of countermeasures to control lateral stability problems associated with degradation and aggradation are:

- Flexible revetment, of which rock riprap is the most widely used type, has a better record of performance than does rigid revetment.
- Rock and wire mattresses (gabion) have proven successful where adequately sized riprap is not available and where streambanks are high and steep.
- Plating (keying) of riprap increases its stability and resistance to debris and ice loads.

- Concrete pavement provides a high degree of safety against debris and ice loads.
- Rigid revetments are not recommended at sites where frost heaving and/or bank subsidence are common.
- Bulkheads are advantageous for control of internal slope failures where the slope cannot be graded to a low enough angle for the placement of other revetment.
- All revetment types must be designed to adequately protect against undermining at toe and ends.
- Retards have been used most successfully to provide flow alignment at crossings.
- Retards have proven to be most successful on channels having
  - small to moderate widths ($<$100m),
  - flow velocities not frequently exceeding 1.5-1.8 m/sec, and
  - sand bed channels with relatively large bed and suspended loads.
- Required retard permeability is inversely proportional to the radius of curvature of the bend.
- Retards by themselves perform best in humid regions where natural revegetation of banks aids the stabilization of bank lines.
- Retards in combination with spurs and/or bank revetment provide additional protection on channels experiencing high velocities and for abrupt flow alignment problems.
- Earth or rock embankment and timber pile retards provide the highest level of esthetic appeal of the retards analyzed.
- Spurs are predominantly used for flow alignment at bridges and bends where they may be less expensive and more effective than riprap revetments.
- Spurs are more effective than retards where abrupt changes are required in flow alignment on large rivers and rivers characterized by high velocity flows.
- Impermeable spurs are best suited to handle cases that require abrupt changes in flow alignment on wide rivers or that require substantial flow constriction such as is the case on wide braided rivers.
- Impermeable spurs provide a higher level of safety against ice and debris hazards than do permeable spurs.
- Permeable spurs are best suited to providing flow alignment at bridges and meander bends on small ($<$ 30) unbraided channels carrying large bed and suspended loads.

CHAPTER X

RECOMMENDATIONS

Analysis of a large number of case histories of gradation problems in this report indicates that highway engineers can benefit from

- a greater understanding of the natural and human causes of such problems,
- a greater knowledge of the methods for identifying grade changes and calculating their extent, and
- more complete information on the countermeasures that are available for use in controlling aggradation, degradation, and the associated lateral erosion at highway stream crossings.

This report has presented information on the causes of gradation problems, the methods and techniques for recognizing them and calculating their severity, and the measures available to control them.

The following recommendations are made:

- *Information from the case history data base should be used to study appropriate changes in design procedure to account for short- and long-term gradation changes. The impact should be determined for water surface profiles, flood limits, hydraulic properties such as width and depth, scour, debris, and lateral movement of the river.*
- *The streambed elevation at bridges in problem areas should be inspected annually because of the prevalence of gradation problems.*
- Countermeasures to control aggradation and degradation have been documented in a number of case histories. *The selection of an appropriate control scheme for a given site should be based on an understanding of the factors influencing the river system, the results of an adequate stability analysis, and good engineering judgment based on the recommendations and guidelines outlined in Chapter VIII.*
- The magnitude of aggradation or degradation to be expected in the vicinity of a highway crossing can be estimated by analyzing the potential bed-material volume change in the local reach. Three levels of analysis or procedures for predicting grade changes are given in Chapter VI. *If the problem includes complicated flow situations and numerous highway crossings, a fully dynamic modeling approach should be used after a preliminary investigation using less complex techniques documents the potential for a grade change problem. If the staff of the local highway department does not have the experience required to conduct such an analysis, help should be sought from other highway offices or other competent engineering groups.*
- *Aggradation and degradation should be considered not only in the design of highway crossings but also in the installation of any countermeasures, including*
  - *flexible revetment or bed armoring;*
  - *rigid revetment or bed armoring;*
  - *flow control structures;*
  - *special devices for protection of piers;*
  - *modifications of bridge, approach roadway, or channel; and*
  - *measure incorporation into design of a replacement bridge.*
- *Because so many gradation changes are caused by human activities, quantitative estimates of aggradation and degradation should be made at a bridge crossing to evaluate the impact of any proposed changes in the river system.*

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GLOSSARY OF TERMS

aggradation — general and progressive upbuilding of the longitudinal profile of a channel by deposition of sediment.

alluvium: — unconsolidated material deposited by a stream in a channel, floodplain, alluvial fan, or delta.

alluvial channel: — channel wholly in alluvium; no bedrock is exposed in channel at low flow or likely to be exposed by erosion.

alluvial fan: — a landform shaped like a fan in plan view and deposited where a stream issues from a narrow valley of high slope onto a plain or broad valley of low slope.

anabranch: — individual channel of an anabranched stream.

anabranched stream: — a stream whose flow is divided at normal and lower stages by large islands or, more rarely, by large bars; the width of individual islands or bars is greater than about three times water width; the channels are more widely and distinctly separated than those of a braided stream.

armor: — surfacing of channel bed, banks, or embankment slope to resist erosion.

armoring of the bed: — development of a thin layer of coarser particles at the bed surface.

average velocity: — velocity at a given cross section determined by dividing discharge by cross-sectional area.

avulsion: — a sudden change in course of a channel, usually by breaching of the banks during flood.

backwater: — the increase in water surface elevation relative to the elevation occurring under natural channel and floodplain conditions, induced upstream from a bridge or other structure that obstructs or constricts a channel.

bank (of channel or stream): — lateral boundaries of a channel or stream, as indicated by a scarp or, on the inside of bends, by the streamward edge of permanent vegetal growth.

bank, left (right): — the side of a channel as viewed in a downstream direction.

bankfull discharge: — discharge that fills a channel to the height of its banks where the banks stand at floodplain elevation; for many streams, bankfull discharge has a recurrence interval of about 1.5 years.

bar: — an elongated deposit of alluvium, not permanently vegetated, within or along the side of a channel.

bed (of channel or stream): the part of a stream, bounded by banks over which water flows.

bed layer: a flow layer, several grain diameters thick (usually two) immediately above the bed.

bed load: sediment that is transported in a stream by rolling, sliding, or skipping along the bed or very close to it; considered to be within the bed layer.

bed load discharge (or bed load): the quantity of bed load passing a cross section of a stream in a unit of time.

bed material: sediment consisting of particle sizes large enough to be found in appreciable quantities at the surface of a streambed.

bed-material discharge (or bed-material load): the part of the total sediment discharge that is composed of grain sizes found in the bed and is equal to the transport capability of the flow.

bed shear (tractive force): the force per unit area exerted by a fluid flowing past a stationary boundary.

boulder: a rounded or angular fragment of rock the diameter of which is in the size range of 250 to 4,000 mm.

braid: a subordinate channel of a braided stream.

braided stream: a stream whose flow is divided at normal stage by small mid-channel bars or small islands; the individual width of bars and islands is less than about three times water width; a braided stream has the aspect of a single large channel within which are subordinate channels.

bridge: a structure, including supports, erected over a depression or an obstruction such as a body of water, a road, or a railway having a track or passageway for carrying traffic and a length of opening greater than 6 m.

bridge opening: the cross-sectional area beneath a bridge that is available for conveyance of water when the water surface approaches but does not touch the bottom of the superstructure.

bridge waterway: the area of a bridge opening available for flow, as measured below a specified stage and normal to the principal direction of flow.

bulkhead: a steep or vertical wall that supports a natural or artificial embankment and may also serve as a protective measure against erosion.

channel: the bed and banks that confine the surface flow of a natural or man-made stream; braided streams have multiple subordinate channels that are within the main stream channel; anabranched streams have more than one channel.

channelization: straightening or deepening of a natural channel by artificial cutoffs, grading, flow-control measures, or diversion of flow into a parallel artificial channel.

channel pattern: the aspect of a stream channel in plan view, with particular reference to the degree of sinuosity, braiding, or anabranching.

check dam: a low dam or weir across a channel, for the control of water stage or velocity or for preventing channel degradation.

cobble: a fragment of rock the diameter of which is in the size range of 64 to 250 mm.

countermeasure: a measure, either incorporated into the design of a bridge or installed separately at or near the bridge, that serves to prevent or control hydraulic problems.

concrete paving: plain or reinforced concrete slabs poured or placed on the surface to be protected.

constriction: a control section, such as a bridge crossing, channel reach or dam, with limited flow capacity in which the discharge is related to the upstream water surface elevation; a constriction may be either natural or artificial.

contact load: sediment particles that roll or slide along in almost continuous contact with the streambed.

contraction: the effect of channel constriction on flow.

clay: particles, usually of clay minerals, the diameter of which is less than 0.004 mm.

crib: an open-frame structure filled with rocks, intended as protection for a bank or embankment.

cross section (of channel): a section perpendicular to the trend of a channel, bounded by the bed banks and water surface; in geomorphology, the term "cross profile" is applied if either the water surface or the channel perimeter, but not both, are shown.

cutoff: a natural or artificial channel that shortens the length of a stream; natural cutoffs may occur either across the neck of a meander loop (neck cutoffs) or across a point bar (chute cutoffs).

cutoff wall: a wall, usually of sheet piling or concrete, that extends from the toe of revetment down to scour-resistant material or to below the expected scour depth.

debris: material transported by the stream, either floating or submerged, such as logs or brush that may lodge against the bridge.

deflector: alternative term for "spur."

degradation: general and progressive lowering of the longitudinal profile of a channel by erosion.

density of water-sediment mixture: bulk density (mass per unit volume) including both water and sediment.

design high-water level:	the maximum water level that a bridge opening is designed to accommodate without contravention of the adopted design constraints.
dike:	an impermeable linear structure for the control or containment of overbank flow; a dike trending parallel with a river bank differs from a levee only in that the dike extends for a much shorter distance along the bank.
discharge:	the volume of flow of a stream per unit time, usually expressed in m^3/s.
discharge weighted concentration:	dry weight of sediment in a unit volume of stream discharge, or the ratio of discharge of dry weight of sediment to discharge by weight of water-sediment mixture.
drift:	alternative term for "debris."
eddy current:	a current of water usually moving in a circular pattern within or at the flow boundaries of the main current; it is associated with turbulence of flow.
ephemeral stream:	a stream or reach of a stream that does not flow for parts of the year. As used here, the term includes intermittent streams whose flow is less than perennial but more then ephemeral.
erosion:	the general process by which solid materials at the Earth's surface are loosened or worn away and simultaneously transported.
fill-slope:	side or end slope of an earth-fill embankment. Where a fill-slope forms the streamward face of a spillthrough abutment, it is regarded as part of the abutment.
filter blanket:	one or more layers of gravel or layers of intermediate sized sand and of gravel placed between bank material and riprap to prevent erosion of the embankment fill behind the riprap.
filter cloth, plastic:	cloth of woven plastic strands that serves the same purpose as a granular filter blanket.
fine material (or wash load):	that part of the total sediment load that is composed of particle sizes finer than those represented in the bed. Normally the fine-sediment load is finer than 0.062 mm for a sand-bed channel. Silts, clays, and sand could be considered as wash load in coarse gravel and cobble bed channels.
flashy stream:	stream characterized by rapidly rising and falling stages, as indicated by a sharply peaked hydrograph. Most flashy streams are ephemeral but some are perennial.
floodplain:	a nearly flat, alluvial lowland bordering a stream, formed by stream processes, that is subject to inundation by floods.
flow-control structure:	a structure either within or outside a channel that acts as a countermeasure by controlling the direction, depth, or velocity of flowing water.

flow hazard: flow characteristics (discharge, stage, velocity, or duration) that are associated with a hydraulic problem or that can reasonably be considered of sufficient magnitude to cause a hydraulic problem or to test the effectiveness of a countermeasure.

foundation: the supporting material upon which the substructure portion of a bridge is placed.

freeboard: the vertical distance above a design stage that is allowed for waves, surges, drift, and other contingencies.

Froude number: a dimensionless number that represents the ratio of gravitational to inertial forces. High Froude numbers are indicative of high flow velocity and the potential for scour.

gabion: a basket or compartmented rectangular container made of steel wire mesh. When filled with cobbles or other rock of suitable size, the gabion becomes a flexible and permeable block with which flow-control structures can be built.

general scour: scour in a channel or on a floodplain that is not localized at a pier, abutment, or other obstruction to flow. In a channel, general scour usually affects all or most of the channel width.

gravel: particles, usually of rock, whose diameter is between 2 and 64 mm. The term gravel is also applied to a mixture of sizes (gravel with sand or gravel with cobbles) in which the dominant or modal fraction is in the gravel size range.

groin: alternative term for "spur."

guide bank: alternative term for "spur dike."

headcutting: channel degradation associated with abrupt changes in the bed elevation (headcut), that migrates in an upstream direction.

historical flood: a past major flood event of known magnitude that may be a significant factor in the hydraulic design of a bridge.

hydraulic problem (at a bridge): an effect of stream flow, tidal flow, or wave action on a crossing such that traffic is immediately or potentially disrupted.

incised stream: a stream that flows in a well-defined channel; the banks that stand more than 5 m above the water surface at normal stage are regarded as high.

icing: masses or sheets of ice formed on the frozen surface of a river or floodplain. When shoals in the river are frozen to the bottom or otherwise dammed, water under hydrostatic pressure is forced to the surface where it freezes.

instantaneous discharge: discharge at a given moment.

invert: lowest point in the channel cross section or at flow control devices such as weirs or dams.

island: a permanently vegetated area, emergent at normal stage, that divides the flow of a stream. Some islands originate by establishment of vegetation on a bar, and others originate by channel avulsion or at the junction of minor tributaries with a stream.

jack: a device for flow control and protection of banks against lateral erosion; it has six mutually perpendicular arms rigidly fixed at the center and strung with wire. Kellner jacks are made of three steel struts; concrete jacks are made of three reinforced concrete beams bolted together at the midpoints.

jack field: rows of jacks tied together with cables, some rows generally parallel with the banks and some perpendicular thereto or at an angle. Jack fields may be placed outside or within a channel.

jetty: alternative term for a "spur."

local scour: scour in a channel or on a flood plain that is localized at a pier, abutment, or other obstruction to flow.

lateral erosion: erosion in which the removal of material has a dominantly lateral component, as contrasted with scour in which the component is dominantly vertical.

levee: a linear embankment outside a channel for containment of flood water.

load (or sediment load): sediment that is being moved by a stream.

longitudinal profile: the profile of a stream or channel drawn along the length of its centerline. In drawing the profile, elevations of the water surface or the thalweg are plotted against distance as measured from the mouth or from an arbitrary initial point.

meander loop: an individual loop of a meandering or sinuous stream lying between inflection points with adjoining loops.

meander scrolls: low concentric ridges and swales on a floodplain, marking the successive positions of former meander loops.

meandering stream: a stream having a sinuosity greater than some arbitrary value, herein placed at 1.25. The term also implies a moderate degree of pattern symmetry, imparted by regularity of size and repetition of meander loops.

measured scour: the measured depth to which a surface is lowered by scour below a reference elevation.

median diameter: the particle diameter at the 50-percentile point on a size distribution curve such that half of the particles (by weight for samples of sand, silt, or clay and by number for samples of gravel) are larger and half are smaller.

mid-channel bar: a bar lacking permanent vegetal cover that divides the flow in a channel at normal stage.

migration:	change in position of a channel by lateral erosion of one bank and simultaneous accretion of the opposite bank.
natural levee:	a low ridge along a stream channel, formed by deposition during floods and that slopes gently away from the channel.
nonalluvial channel:	a channel whose boundary is wholly in bedrock.
normal stage:	the water stage prevailing during the greater part of the year.
perennial stream:	a stream or reach of a stream that flows continuously for all or most of the year.
pile bent (or pile pier):	a pier composed of piles capped or decked with a timber grillage or with a reinforced-concrete slab forming the bridge foundation.
point bar:	an alluvial deposit of sand or gravel lacking permanent vegetal cover occurring in a channel at the inside of a meander loop usually somewhat downstream from the apex of the loop.
railbank protection:	a type of countermeasure composed of rock-filled wire fabric and supported by steel rails or posts driven into the streambed.
reach:	a segment of stream length that is arbitrarily bounded for purposes of study.
recurrence interval (R.I.); return period; exceedance interval:	the reciprocal of the annual probability of exceedence of a hydrologic event.
relief bridge:	an opening in an embankment on a floodplain to permit passage of overbank flow.
retard:	a permeable or impermeable linear structure in a channel, parallel with the bank and usually at the toe of the bank, intended to reduce flow velocity, induce deposition, or deflect flow from the bank.
revetment:	rigid or flexible armor placed on a bank or embankment as protection against scour and lateral erosion.
riffle:	a natural shallow extending across a streambed at which the surface of flowing water is broken by waves or ripples. Typically, riffles alternate with pools along the length of a stream channel.
riprap:	in the restricted sense, layer or facing of broken rock or concrete dumped or placed to protect a structure or embankment from erosion; also the broken rock or concrete suitable for such use. Riprap has also been applied to almost all kinds or amor, including wire-enclosed riprap, grouted riprap, sacked concrete, and concrete slabs. Usage in the restricted sense is following herein.
saltation load:	sediment bounced along the stream bed by energy and turbulence of flow and by other moving particles.

sediment (or fluvial sediment): fragmental material transported by, suspended in, or deposited by water.

sediment concentration (by weight or by volume): weight or volume of sediment relative to quantity of transporting or suspending fluid or fluid-sediment mixture.

sediment discharge (or sediment load): the quantity of sediment that is carried past any cross section of a stream in a unit of time. Discharge may be limited to certain sizes of sediment or to a specific part of the cross section.

sediment yield: the total sediment outflow from a watershed or a drainage area at a point of reference and in a specified time period. This outflow is equal to the sediment discharge from the drainage area.

spatial concentration: the dry weight of sediment per unit volume of water-sediment mixture in place or the ratio of dry weight of sediment to total weight of water-sediment mixture in a sample or unit volume of the mixture.

spillthrough abutment: a bridge abutment having a fill slope on the streamward side. The term originally referred to the "spillthrough" of fill at an open abutment but is now applied to any abutment having such a slope.

spread footing: a pier or abutment footing that transfers load directly to the earth.

spur: a permeable or impermeable linear structure that projects into a channel from the bank to alter flow direction, induce deposition, or reduce flow velocity along the bank.

spur dike: a dike extending upstream from the approach embankment at either or both sides of the bridge opening to direct the flow through the opening. Some spur dikes extend downstream from the bridge.

stage: the height of water surface above a specified datum.

stream: a body of water that may range in size from a large river to a small rill flowing in a channel. By extension, the term is sometimes applied to a natural channel or drainage course formed by flowing water whether it is occupied by water or not.

suspended load (or suspended sediment): sediment that is supported by upward components of turbulent currents and stays in suspension for an appreciable length of time.

suspended-sediment discharge (or suspended load): the quantity of suspended sediment passing through a stream cross section above the bed layer in a unit of time.

thalweg: a line connecting the lowest points along the bed of a channel.

total sediment discharge: the sum of suspended-sediment discharge and bedload discharge or the sum of bed material discharge and wash load discharge of a stream.

total sediment load (or total load): the sum of suspended load and bedload or the sum of bed-material load and wash load of a stream.

uniform flow: flow of constant cross section and average velocity through a reach of channel during an interval of time.

unit shear force (shear stress): the force or drag developed at the channel bed by flowing water. For uniform flow, this force is equal to a component of the gravity force acting in a direction parallel to the channel bed on a unit wetted area. Usually expressed in units of stress, lb/ft^2 or kg/m^2.

unsteady flow: flow of variable cross section and average velocity through a reach of channel during an interval of time.

velocity: the rate of motion of a stream or of the objects or particles transported therein; usually expressed in ft/s or m/s.

velocity-weighted sediment concentration: the dry weight of sediment discharged through a cross section during unit time.

vertical (full-height) abutment: an abutment, usually with wingwalls, that has no fill slope on its streamward side.

wandering thalweg: a thalweg whose position in the channel shifts during floods and typically serves as an inset channel that transmits all or most of the streamflow at normal or lower stages.

wash-load discharge (wash-load): that part of total sediment discharge that is composed of particle sizes finer than those represented in the bed and is determined by available bank and upslope supply rate.

weephole: a hole in an impermeable wall or revetment to relieve the neutral stress or pore-water pressure.

wire mesh: wire woven or welded to form a mesh, the openings of which are of suitable size and shape to enclose rock or broken concrete or to function on fence-like spurs and retards.

APPENDIX A

CASE HISTORIES

This appendix contains 110 detailed case histories documenting aggradation and degradation problems and their consequences at highway crossings. It forms the basis of information used for this report and provides a useful source of information for designers or researchers involves in this area of river mechanics.

Copies of Appendix A are available through Stephen A. Gilje (HRS-42), the contract manager.

APPENDIX B

ANNOTATED BIBLIOGRAPHY

This appendix contains an annotated bibliography of the literature, information, and other studies reviewed, and provides annotations of several hundred references on various aspects of aggradation and degradation. Areas covered include case histories, control methods, prediction techniques and other theoretical concepts, and mathematical and laboratory models of aggradation and degradation processes. This bibliography is intended for use by researchers and not as a general reference.

Copies of Appendix B are available through Stephen A. Gilje (HRS-42), the contract manager.

Appendix C

APPLICATION OF METHODS TO CASE HISTORIES

CASE STUDY I: SALT RIVER BRIDGE, PHOENIX, ARIZONA

PROBLEM STATEMENT

The Salt River Bridge study illustrates an actual analysis that was used at an existing bridge to establish protection measures.

The Salt River has been subjected to repeated floods over the past few years. Local scour and general bed degradation during successive flows have altered the river bed around piers of the Salt River Bridge. The Arizona Department of Transportation (ADOT) concluded the bridge was susceptible to further damage. Analysis of the susceptibility of the pier foundations to failure from general channel bed degradation combined with local scour during future floods was required. Structural and nonstructural means of controlling the expected degradation were studied. The structural methods included (a) channelization using guide banks, (b) a downstream grade control structure, and (c) control of side drainage flows. Nonstructural measures included (a) controlled gravel mining and (b) operation of upstream reservoirs to regulate the flow.

PHASE I: QUALITATIVE SYSTEM STABILITY

The qualitative system stability analysis is based upon site inspection, examination of maps and photographs, and a review of the historic data available for the site.

River/Watershed Classification and Background Information

The Salt River Bridge is located just south of Phoenix, Arizona, on I-10 (Figure C-1). The Salt River is an alluvial sand/gravel/cobble channel located in a valley setting of moderate relief. The channel is sinuous with a high degree of braiding and anabranching. It experiences random variation in overall width ranging from 600-1500 m with individual braids averaging about 150 m. The average channel slope is 0.002 but falls to 0.0003 in a 2 km reach in the vicinity of the bridge. Vegetation is sparse along the channel banks, being mostly desert shrubs. Upland soils are moderately erodible, gravelly sand loam with the sand sizes predominating.

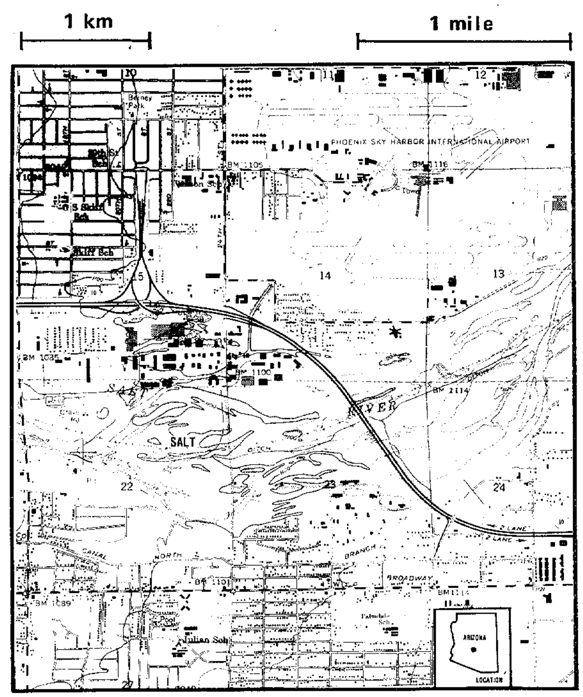


Figure C-1. LOCATION OF I-10 (SALT RIVER) BRIDGE

The flow habit of the Salt River is ephemeral with the channel experiencing flow less than 20 percent of the time. The Salt River is extremely flashy and has a history of flooding. A description of floods in the Salt River from November 1965 to January 1976 has been developed by the U.S. Geological Survey (USGS). The USGS report substantiated the possibility of major flood events such as those of 1979 and 1980. Specifically, six floods have recently occurred in the Salt River.

Man's influence within the Salt River Basin has been extensive. Urbanization in the Phoenix-Tempe area has been widespread. The urbanized area has increased from about 50 to more than 150 km^2 since 1953. This expansion is documented in the sequential aerial photographs shown in Figure C-2. The expansion has encroached on the flow channel in numerous locations. As can be seen in Figure C-2, the Phoenix airport runways were built into the floodplain and significantly encroach on the Salt River channel. In addition, gravel mining activities are common in the river channel and floodplain both upstream and downstream of the bridge as is evident in the aerial photographs. More recent visits to the site (1979) indicate that these human activities have caused the low-flow channel to shift to the south endangering some of the piers.

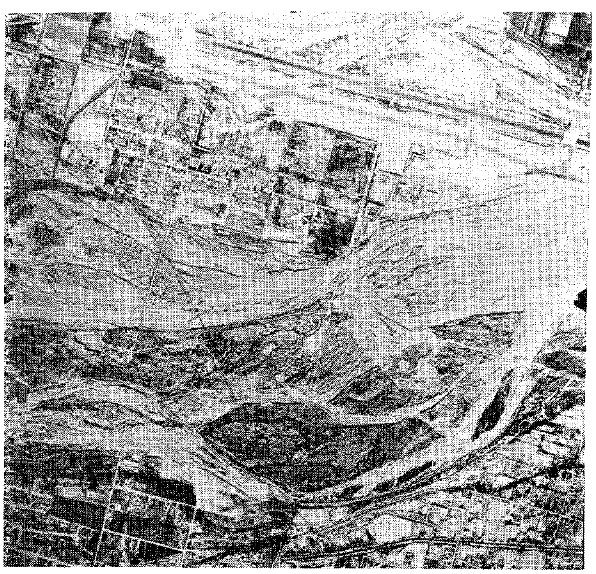
Stability Consideration

River responses to human influences are discussed in detail in Chapter IV. Gravel mining operations are known to produce degradation and headcutting. The increased runoff and larger peak discharges resulting from urbanization will tend to reshape the channel system making the river generally wider, deeper, and more sinuous. Also, encroachments within the flow channel will increase the local energy slope, increasing the potential for degradation at and downstream of the encroachment. These river system responses are verified by applying Lane's Principle as discussed in Chapter VI of the main text.

This preliminary investigation reveals a significant potential for vertical as well as lateral instability within the reach of the Salt River under investigation. A detailed quantitative investigation is called for at this point to investigate further the potential impact of the anticipated future channel degradation on the I-10 crossing.

PHASE II: ANALYSIS OF POTENTIAL GRADE CHANGES

The complexity of the grade change problem at the I-10 crossing is reflected in the number and magnitude of the influencing activities. It is further complicated by the existence of seven other crossing structures within this local reach of the Salt River. To

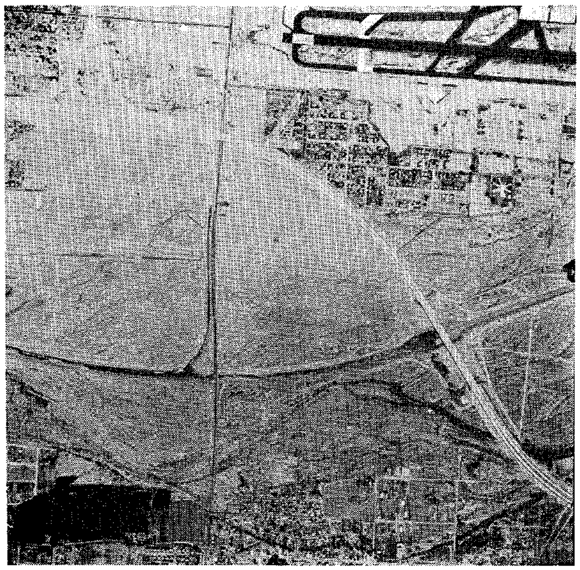


(1)

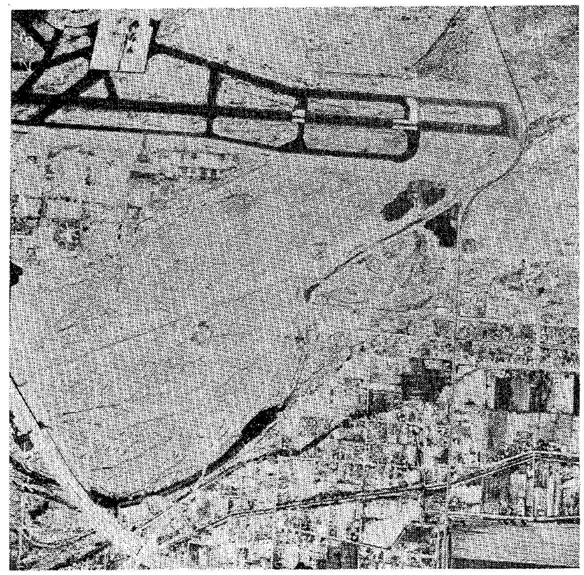
(2)

(a) 1960

Figure C-2. SALT RIVER SEQUENTIAL AERIAL PHOTOGRAPHS



(1)



(2)

(b) 1965

(1)

(2)

(c) 1972

Figure C-2. SALT RIVER SEQUENTIAL AERIAL PHOTOGRAPHS (Continued)

analyze accurately the I-10 crossing, all bridges within this reach must be included in the analysis. The effects of gravel mining, channel encroachments, and increased runoff must also be considered. The factors just mentioned as well as the importance of maintaining a safe interstate highway crossing here reflect the need to apply a water and sediment modeling procedure to adequately quantify the potential grade changes and their impacts within this reach. This analysis required personnel with an advanced understanding of river hydraulics, sediment transport, and river and watershed modeling techniques.

Data Requirements for Qualitative Analysis

Data requirements for the analysis of the I-10 Salt River Bridge include detailed channel geometry, bridge geometry data at all eight crossings, hydrologic data, bed material size distributions, and flow resistance information.

Channel Geometry

The index map showing the location of cross sections and reaches used in the analysis is given in Table C-1. Twenty-two cross sections were utilized in the analysis of the existing condition. The total length of the reach of river analyzed was approximately 8.9 km. The study area extended about 4.4 km upstream of the Salt River (I-10 Bridge) and 4.5 km downstream of the bridge. A downstream control is assumed to exist at the Seventh Street bridge crossing. The cross-sectional data were digitized from a 1980 survey. The floodplain cross-sectional data were augmented with aerial photographs. The chan-

Table C-1. INDEX MAP FOR THE SALT RIVER IN THE VICINITY OF THE I-10 BRIDGE (EXISTING CROSS SECTION)

RIVER DISTANCE (m)	CROSS SECTION NUMBER	LOCATION	REACH DEFINITION
8,680	22	← Upstream Boundary	
7,890	21		8
7,375	20		
7,175	19		7
6,580	18		
6,130	17		6
5,500	16		
5,190	15		
4,875	14		5
4,725	13		
4,570	12		
4,540	11	← I-10 Bridge	4
4,500	10		
4,375	9		
4,250	8		3
3,900	7		
3,400	6		
2,880	5		2
2,250	4		
1,600	3		
790	2		1
0	1	← Downstream Boundary (Seventh Street)	

nelization condition required three additional cross sections to apply the proposed design. To simplify the analysis, it was necessary to define specific channel reaches. Hence, eight reaches were defined in Table C-1; the I-10 Bridge is in Reach No. 4. The average channel gradient is about 0.002, which is fairly steep for such a large river.

Bridge Data

Nineteen piers support the I-10 Bridge, and the elevations of the pier foundations are given in Table C-2. As shown in Table C-2, the low-flow channel was originally located between Piers No. 1 and 9. Other data required included bed elevations at and between piers, bridge deck and low chord elevations, pier type descriptions, pier spacing, and abutment geometry and type. This information was required at each of the bridges and was obtained from bridge plans for modeling purposes.

Table C-2. ELEVATION OF PIER FOUNDATIONS

PIER NO.	FOUNDATION ELEVATION (m)
1	326.77
2	326.87
3	326.92
4	327.02
5	327.08
6	325.13
7	325.16
8	325.19
9	325.22
10	327.20
11	328.27
12	328.70
13	328.67
14	328.64
15	328.60
16	328.54
17	328.48
18	328.39
19	328.30

Hydrology

The flow habit of the Salt River is highly ephemeral. As a result, the channel bed responds to individual flow events, and therefore, the analysis was conducted to find the impact of the design flood.

The peak discharge of the design flood for the bridge adopted by the ADOT is 4956 m^3/sec, and the estimated peak discharge for the February 1980 flood is 5239 m^3/sec. However, the actual hydrograph was not available. With this constraint, the best method for predicting the flood hydrographs for the study area is to employ an advanced theoretical approach involving rainfall-runoff relationships and numerical routing techniques. However, because of time and money constraints, a more practical means was used for estimating the shape of the hydrograph for the design flood. The method involved the development of a typical hydrograph normalized with respect to the flood peak. The resulting design hydrograph is shown in Figure C-3. Flood hydrographs for several other return periods were also generated. The development of flow hydrographs is beyond the scope of this report.

Bed Material Size Distributions

Both surface and subsurface samples of the bed material were collected and analyzed. The size dis-

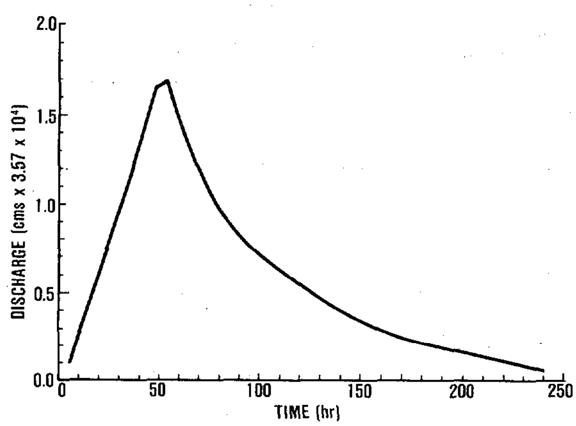


Figure C-3. ONE-HUNDRED YEAR FLOOD HYDROGRAPH

tribution for the composite surface sample is shown in Figure C-4. The size distribution for the composite subsurface sample is shown in Figure C-5. A comparison of these figures verifies the existence of an armor layer. Field observations verify that it is difficult to determine representative sizes that accurately describe the bed material in the Salt River. For simplicity, the following characteristics of the bed material were adopted for the scour analysis: the surface layer has a D_{50} (median diameter) of 237 mm and a σ (gradation coefficient) of 1.6; the subsurface layer has a D_{50} of 123 mm and a σ of 7.0.

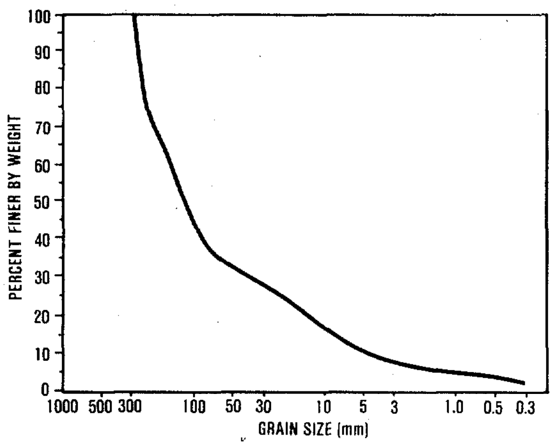


Figure C-4. GRADATION CURVE FOR COMPOSITE SUBSURFACE SAMPLE, SALT RIVER

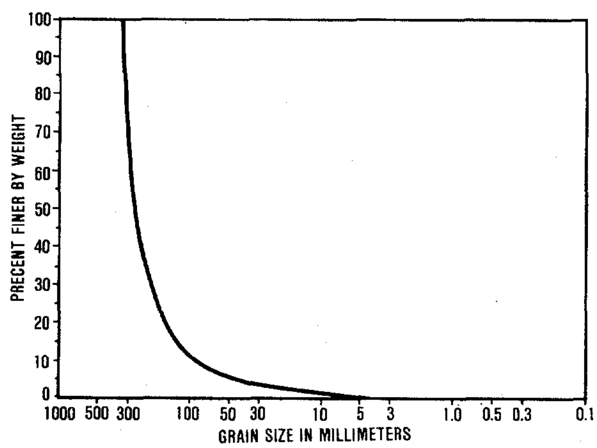


Figure C-5. GRADATION CURVE FOR COMPOSITE SURFACE SAMPLE, SALT RIVER

Flow Resistance

It is difficult to estimate accurately resistance to flow in a gravel bed channel. The relative roughness can change greatly during a flood. According to Chow (1959), the estimated value of Manning's roughness coefficient, n, is about 0.04 for a small mountain stream with no vegetation in the channel, banks usually steep, trees and brush along banks submerged at high stages, and a channel bottom consisting of gravel, cobbles, and a few boulders. This description fits the Salt River, except that the Salt River is not small. It is a larger intermittent river, so the Manning's roughness for the Salt River bed will be less than 0.04. The minimum value for this type of stream is about 0.03.

Strickler's formula [Simons and Senturk (1977)] is often used to estimate the grain resistance. Assuming that the D_{50} is 237 mm, Manning's n is 0.03 according to Strickler's formula. A recent study at Colorado State Unviersity by Simons, Al-Shaikh-Ali, and Li (1979) indicated that the passage of a sediment wave during the flood can significantly reduce the resistance to flow. In addition, the stream bed is likely to be in upper regime during flood periods [Simons and Senturk (1977)]. For a conservative estimate of potential scour, a Manning's n value of 0.03 is utilized to analyze the hydraulics of the main channel. Manning's n for the portion of the floodplain comprised of gravel, boulders, and sand is assumed to be 0.05 and for the portion of the floodplain occupied by buildings, the n value is assumed to be 0.1.

Sediment Transport Considerations

The Meyer-Peter and Muller equation [Equation (34)] is the best suited transport equation for steep gravel and cobble bed streams. For practical application of this relationship, the coefficients (A" and B") should be adjusted using field data; however, since no such data were available and time would not allow for the collection of the required data, the original coeffients of A" = 0.047 and B" = 0.25 were used. The

Meyer-Peter and Muller type equation accounts for the bed load only, so the suspended portion of the bed material load is computed by the Einstein procedure given in Equation (46).

The Meyer-Peter and Muller equation was used to predict the transport of individual size fractions within the surface and subsurface bed materials. The armor layer was analyzed first to see if it would remain stable or be broken up. If flow conditions were sufficient to break down the armor layer, the transport of subsurface and surface materials was considered. By summing the transport of each size fraction with the Meyer-Peter and Muller equation, the total volume of transported material was analyzed. Material not transported for given hydraulic conditions is assumed to remain behind, and this information is used to adjust the gradation curves to reflect the loss of the transported material. This procedure properly accounts for the armoring effect of the coarser particle sizes. Past experience verifies that the Meyer-Peter and Muller equation used in this fashion combined with the Einstein suspended load procedure provides the best estimate of sediment transport for field situations in sand/gravel/cobble bed channels carrying significant amounts of suspended load.

Sediment Yield

Because no detailed information is available on the upland watersheds and upstream channels, a realistic and conservative assumption was utilized to determine the sediment supply from the upstream channels. The upstream supplies of sediment have been reduced by the construction of three dams on the Salt River and two dams on the Verde River. As a result, the primary source of sediment to the Salt River is from tributaries below Bartlett Dam on the Verde and below Stewart Mountain Dam on the Salt River. The continuous mining of sand and gravel upstream of the I-10 Bridge can further reduce the supply of sediment to the study site. For a conservative estimate of river response, the assumed sediment yield to the study reach is largerly controlled by the ability of the flows to transport bed materials to the site from upstream. This assumption neglects the potential supply of fine sediment from upland watersheds. For a more comprehensive analysis, evaluation of degradation and aggradation should include effects of the upstream channel and watershed systems. However, for this estimate, it is assumed that the upstream channel and flow conditions dictate the sediment supply.

The estimated sediment yield during the passage of the design flood hydrograph utilizing the modified Meyer-Peter and Muller equation is approximately 46,000 tonnes. Assuming a value of 1.3 for the sediment bulking factor, this weight of sediment has a volume of 20,644 m^3. It is likely that the actual sediment yield from the system is higher than this estimated value. Therefore, the subsequent analysis will yield a high estimate of degradation.

Grade Change Analysis

The I-10 Bridge is a very important bridge over the Salt River. Because of the excessive cost of replacing the structure and the severe traffic disruption that would be caused by the loss of the bridge, a detailed mathematical analysis was used.

The scour potential for the existing bridge site condition was analyzed considering the passage of the design flood as documented in Figure C-2. This single storm event analysis was considered to be the most critical condition for the bridge since, in this case, it will produce the maximum anticipated degradation. This alternative is essentially a "no action" plan that considers the bridge to be expendable and plans for rebuilding and/or significant maintenance after major floods. This existing condition can be regarded as the "basic" condition for comparison and design of alternatives.

The analysis of the existing condition included the determination of the following quantities: (a) sediment yield from the upstream channel reach, (b) degradation and aggradation within reaches and by cross section, (c) local scour around the pier foundations, (d) susceptibility of the pier foundations to erosion, and (e) water surface elevation.

Degradation and Aggradation

The mechanics of degradation and aggradation are complicated to the extent that simplifying assumptions are needed to obtain practical solutions. The dominant physical processes including water runoff, sediment transport, sediment routing, degradation, aggradation, breaking and forming of the armor layer, etc., are unsteady in nature. To simplify the solution and make the results of the analysis compatible with the HEC-2 flood level analysis, a known-discharge assumption is adopted. The known-discharge solution is appropriate in this study because of the short distances involved in the analysis. In addition, to save computer time, the degradation and aggradation analysis is conducted on a reach basis utilizing the average hydraulic parameters from the HEC-2 analysis. The amount of predicted aggradation and degradation is distributed to the verticals of a cross section according to the channel conveyance to yield a set of new cross sections.

The steps in the analysis procedure are as follows:

Step 1. Divide the design flood hydrograph into steady flow increments or steps,

Step 2. Apply HEC-2 or similar backwater program over a steady time increment to evaluate hydraulic conditions,

Step 3. Route sediment through reaches based on hydraulic conditions from HEC-2,

Step 4. Distribute aggradation and degradation in each reach to the verticles of each section,

Step 5. Using these newly developed channel sections, return to Step 2 for second time step.

To simulate the degradation process, the flood hydrograph was divided into 25 steady time increments of 6, 12, and 24 hours depending on the rate of rise and fall of the hydrograph. Using the first steady flow step, the HEC-2 water surface profile was run to determine the hydraulic conditions necessary for input into the Meyer-Peter and Muller transport equation for sediment routing. The Meyer-Peter and Muller equation was then used to route sediment through the system based on the continuity concept expressed in Equation (84). The volume of aggradation or degradation computed for each reach is then accounted for by distributing it to the verticals of each cross-section based on channel conveyence. This creates a new set of cross-sections that represent geometric conditions at the end of the first time step. Then the new geometric conditions are input to HEC-2 to reevaluate hydraulic conditions during the second time step. This process is continued until the entire hydrograph has been evaluated and a final bed profile is reached.

The spatial resolution of the study area is given in Table C-1; it shows the channel reaches and river cross sections considered in the analysis of the I-10 Bridge site. The temporal resolution considered in the analysis is the design hydrograph (Figure C-3). A comparison of original and final bed profile after passing the design flood is given in Figure C-6, which illustrates the significant scour that will occur in the vicinity of the bridge. This result is similar to that observed after the 1980 flood. In order to provide better information about potential general scour, the time-lapse change of general scour depth at the I-10 Bridge crossing during the design flood for the existing condition is given in Figure C-7. That figure shows that the degradation follows approximately a stepwise pattern that results from the forming and breaking of armoring layers, temporal and spatial distribution of flow, and variation of upstream sediment supply. Large erosion rates do not necessarily coincide with large flow rates, and aggradation takes place during the recession limb of the hydrograph. This evaluation is physically sound, considering the mechanics of erosion and sedimentation.

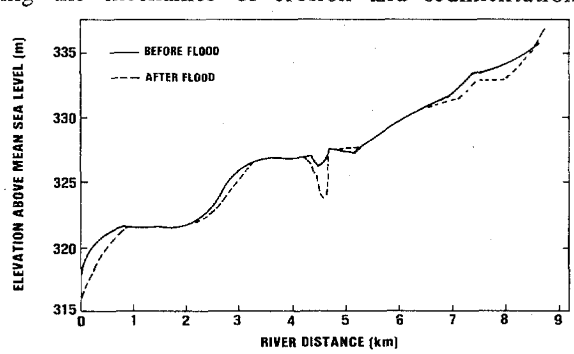


Figure C-6. CHANGE OF THALWEG PROFILE CAUSED BY DESIGN FLOOD

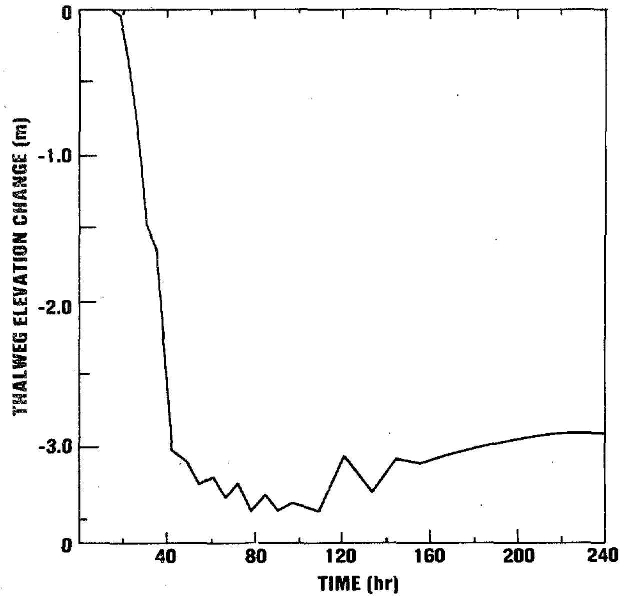


Figure C-7. **DEGRADATION – AGGRADATION AT BRIDGE DURING DESIGN FLOOD**

Local Scour

Local scour must be considered in the analysis of bridge safety. The flow distribution and the quantity of sediment and debris were found to offset local scour in the vicinity of the piers supporting the bridge. The local scour was estimated by the following four methods: (a) Shen's Formula, utilizing the pier Reynolds number [Transportation Research Board (1970)], (b) Neill's Formula [Simons and Senturk (1977)], and (c) a modified method based on the Shields' criteria of incipient motion [Simons and Senturk (1977)] assuming an armoring size of 270 mm. The determined depth of local scour for the existing condition ranges from approximately 1.5 to 2.1 m. If a 15-degree angle of attack is assumed for flow approaching the pier, the scour depth would range from 3.8 to 5.3 m. Assuming the total scour is a sum of the general scour and the local scour, the total depth of potential scour can be determined. The total scour depth ranges from 5 to 9 m depending on the angle of attack. The flood experience of the past 3 years indicates that the channel has a tendency to move toward the south. Hence, low-flow channelization requires protection of the south bank.

PHASE III: IMPACTS ON CROSSING AND COUNTERMEASURE DESIGN MEASURES

The shifting of the river thalweg is a major problem in the Salt River. Many existing bridges have been designed and constructed without adequately considering lateral channel migration. An appropriate measure to mitigate the problem is implementation of a channelization scheme that controls the location and direction of the flow.

Guide banks have often been used effectively on both sand and gravel bed streams to guide the flow of water through a bridge opening and to control the position of scour and protect the abutments. The principal factors that must be included in the design of guide banks include controlled convergence of the flow normal to the opening, plan shape, upstream and downstream lengths, cross section, crest elevation, and scour and riprap protection.

In the United States, the practice is to give the guide banks an elliptical form convergent to the opening; in Pakistan and India, the banks are straight and parallel to the opening with a curved section at the upstream and downstream ends. The form of the short, elliptical guide banks was illustrated by Karaki (1959). Straight guide banks probably do a better job of straightening the flow and minimizing the attack on the abutments. Elliptical guide banks move the scour hole further upstream and downstream of the bridge opening. Generally, the straight guide banks are more effective than the short elliptical guide banks. The suggested upstream length for straight guide banks ranges from 0.75 to 1.25 times the opening width, and the downstream length ranges from 0.1 to 0.25 times the opening width [Neill (1973); Control Board of Irrigation and Power, India (1956)]. Longer guide bank lengths are recommended for use to control the high potential for lateral migration of the low flow channel of the I-10 Bridge site. An example of the proposed layout is given in Figure C-8. The major concept of the proposed channelization plan is to redirect the flow to the old low-flow channel between Piers No. 1 and 9, which are the deepest. This procedure limits the maximum opening width to 215 m. Since the available width of the right-of-way is limited, it is recommended that the channelization be integrated with a

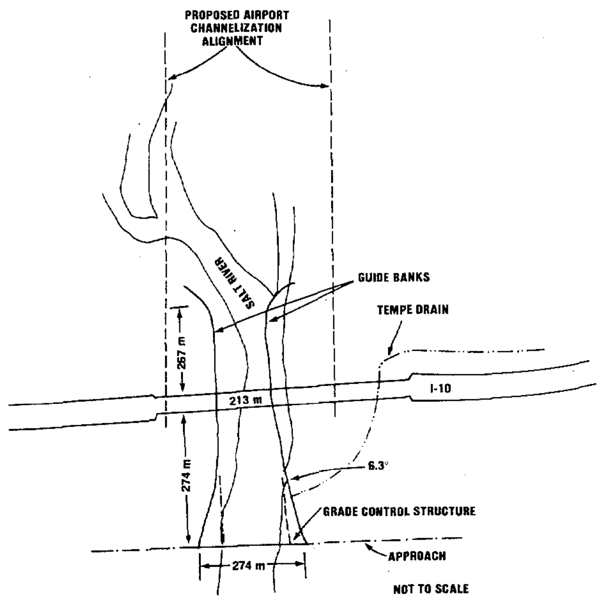


Figure C-8. LAYOUT OF PROTECTION PLAN

downstream grade control structure as indicated in Figure C-8. Three channelization plans were investigated, and it was concluded that the 213.4 m opening with a 274.3 m long grade control structure normal to the flow will function the best hydraulically considering all constraints. The design channel slope for the channelization plan is 0.001. The elevation of the channel invert near the bridge is 329.2 meters.

A side drainage channel, currently flows to the north. Before the recent flood, the old channel was located approximately 61 m north and paralleled the interstate until it reached the south end of the I-10 Bridge. The channel then turned and passed under the bridge between the south abutment and Pier No. 19, continuing in a westerly direction to the low flow channel. The alignment of this drainage channel can remain the same as the old alignment. However, a better and more stable channel cross section should be established. A preliminary estimate indicates that a wider and shallower cross section, yet one that fits within two piers, should suffice.

A grade control structure is usually an effective means for controlling degradation. Such structures prevent headcuts if the gravel pit initiating the headcut is shallow. To adequately protect the bridge, both abutments should be protected with riprap that can withstand the forces exerted by the flow around them. The approaches should also be designed to provide additional structural stability.

Two types of control structures are feasible for limiting erosion through bridge openings: (a) a structure formed of rock riprap reinforced with steel rods will require minimum maintenance and (b) a conventional reinforced concrete drop structure that can more effectively accommodate larger differences in head. However, the latter is much more expensive to construct and maintain. The riprap control structure can be effectively utilized if the potential drop in head across the structure is on the order of 0.5 to 1.0 m. It is usually impractical to use a dumped riprap drop structure for a potential head drop larger than 1.2 m. Riprap revetment structures used to protect the guide banks should be placed on a slope flatter than 1:2 (vertical to horizontal).

A comparison of the original and final bed profiles after passing a 100-year flood for the channelization and grade control plan shows that the grade change for the proposed channelization plan would be insignificant. In fact, the reach at the bridge site will experience slight aggradation. Utilizing guide banks, it is expected that the angle of attack of the flow on the piers will be negligible and computed local scour depth ranges from 1.2 to 1.8 m depending on the equation utilized. The leading edge of the guide banks will experience approximately 2 m of local scour, as estimated by Liu's Equation [Simons and Senturk (1977)]. If a safety factor of 1.5 is required, the bed will potentially degrade to 326 m. According to Table C-2, Piers 2, 3, 4, 5, and 10 are potentially in danger. Riprap with a median diameter of 45.7 cm should be placed in the channelized portion. Additional dumped riprap protection must be provided nar Piers 2, 3, 4, 5, and 10.

CASE STUDY II: DRY CREEK NEAR HEALDSBURG, CALIFORNIA

Case Study II illustrates the application of qualitative and quantitative techniques to a large watershed experiencing a variety of gradation problems. The complexity of this case dictates a very detailed analysis. The preliminary qualitative analysis will be covered in detail.

PROBLEM STATEMENT

Dry Creek is a major tributary to the Russian River located near Healdsburg, California (Figure C-9). The confluence of these two rivers is just south of Healdsburg. The Russian River Basin drains approximately 235 km^2 of watershed in Sonoma and Mendocino Counties, California. Dry Creek has a drainage area of 34 km^2.

In recent years the Russian River Basin has been subjected to man-made changes that have altered the characteristics of the entire watershed. Coyote Dam on the Russian River has altered the flow pattern and the transport of sediment through the system. Furthermore, Healdsburg Dam on the Russian River serves as a control that prevents upstream headcut migration. Similarly, Dry Creek will be even more significnatly altered when the Warm Springs Dam is completed and put in operation. Another major factor that has proved to be significant is the impact of gravel mining activities on these channel systems in relation to their stability. Gravel mining in the Russian River and other tributaries such as Dry Creek has been an important industry in Sonoma County since the early 1900s. In recent years, such activities have continued in Dry Creek and in the Russian River near the mouth of Dry Creek. These activities have been alleged to cause increased bank erosion and degradation in the Dry Creek watershed.

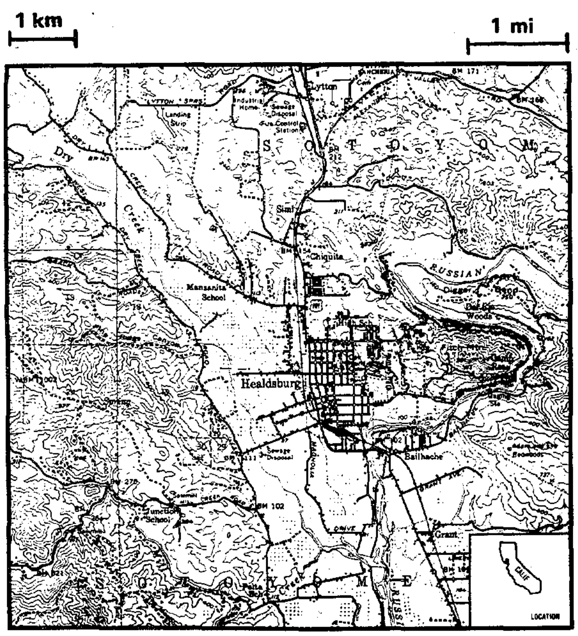


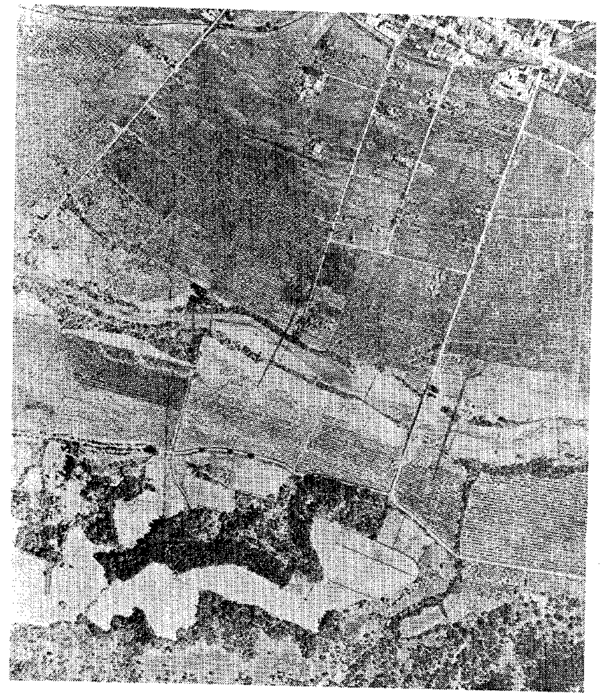
Figure C-9. DRY CREEK CONFLUENCE WITH RUSSIAN RIVER

QUALITATIVE ANALYSIS

The qualitative analysis of gradation problems on Dry Creek was conducted through a study of maps and aerial photographs, hydrology, hydraulics, and natural and man-produced impacts in the watershed.

Dry Creek is located in a valley of high relief ($>$ 300 m) on a well-defined wide floodplain averaging 1.2 km in width (Figure C-9). The low-flow channel is incised within a wider flow channel bordered by low scarps. The low-flow channel has a sinuosity of approximately 1.20, while the wider flood flow channel is straight (sinuosity approximately 1.05). Channel boundaries are alluvial, being composed of sand and gravel. The low flow channel is locally braided and locally anabranched. Both the low-flow and flood-flow channels are equiwidth with the development of wide point bars prevalent along the low-flow channel. Tree cover is generally less than 50 percent of the bankline, and cut banks are evident. Bank material is generally noncoherent silt, sand, gravel, and cobbles. These characteristics of Dry Creek are evident in the sequential aerial photographs shown in Figure C-10.

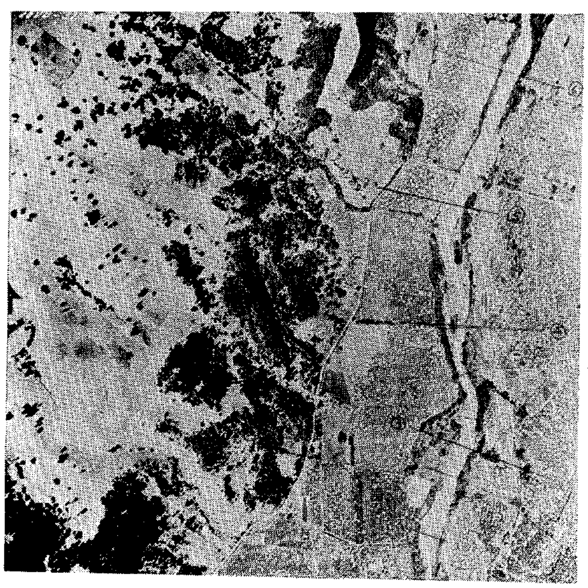
Aerial photographs were available for the first 17.7 km of Dry Creek for the years 1940, 1945,

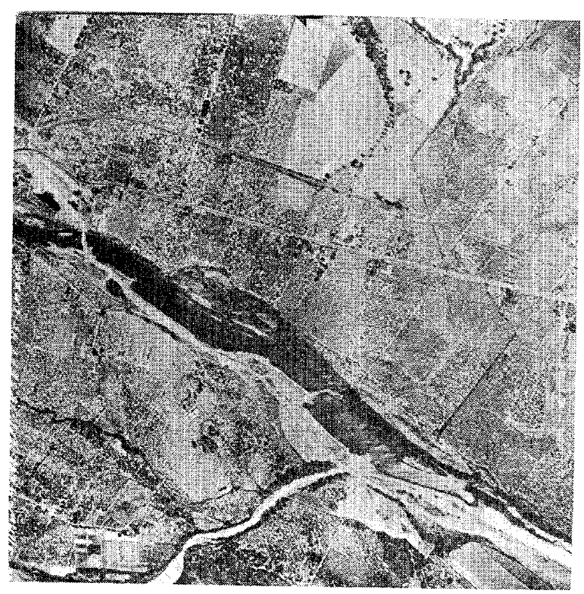


(1)

(2)

(a) 1940



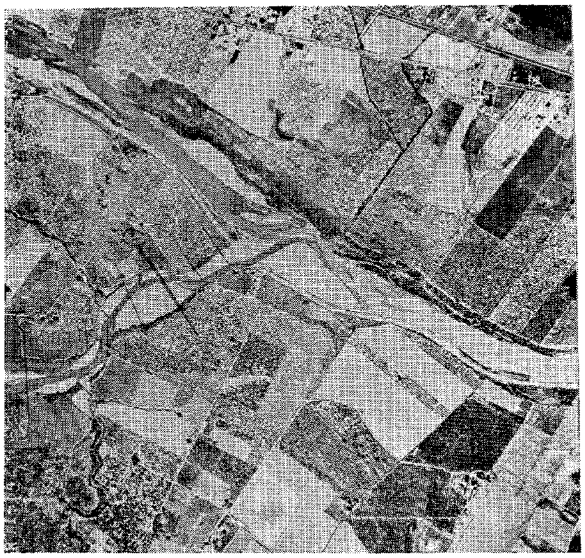


(1)

(2)

(b) 1945

Figure C-10. DRY CREEK SEQUENTIAL AERIAL PHOTOGRAPHS



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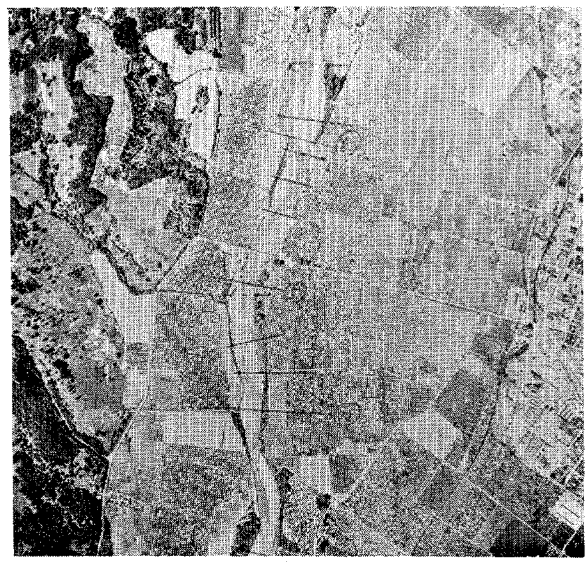
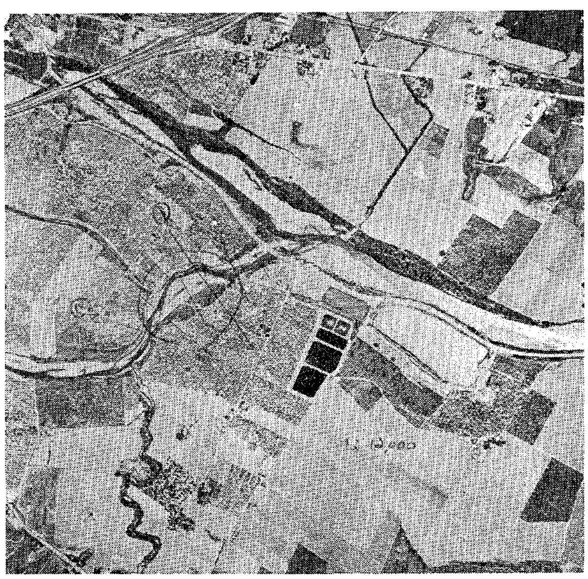


(2)

(c) 1958



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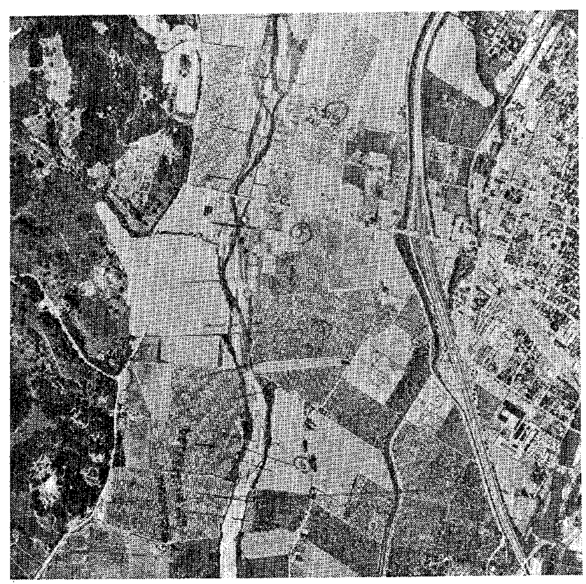


(2)

(d) 1973

Figure C-10. DRY CREEK SEQUENTIAL AERIAL PHOTOGRAPHS (Continued)

(1) (2)

(e) 1979

Figure C-10. DRY CREEK SEQUENTIAL AERIAL PHOTOGRAPHS (Continued)

1956, 1958, 1959, 1967, 1969, 1973, and 1979. These photographs were used for the analysis. The two downstream most photographs show the confluence with the Russian River and a 4.5 km reach of Dry Creek upstream from the confluence for the years 1940, 1945, 1958, 1973, and 1979 (Figure C-10a through C-10e, respectively). Several observations can be made from these photographs. The first observation is that the river is extremely active. Although the overall channel sinuosity remains about the same from year to year, a comparison of the location of meander bends and crossovers reveals that their positions change with time. This is exemplified by studying the bridge crossing in Figure C-10. In 1940 the main channel is against the right abutment; the 1958 and 1973 photographs show the channel against the right abutment; finally, in 1979 the main flow channel has shifted back against the left abutment. There has also been a significant amount of activity just upstream of the confluence with Russian River [Figure C-10 a(2) through e(2)]. Specifically, by comparing the 1945 and 1958 photographs a major channel shift to the north can be observed. This is indicative of the general bank cutting that is prevalent all along Dry Creek although, usually, it is to a lesser degree.

The influences of man's activities can also be observed on the aerial photographs of Figure C-10. Gravel mining activities are evident between sections four and five in Figure C-10 (b) (1). There is also evidence of gravel mining near the confluence of Dry Creek and the Russian River in Figures C-10 d(2) and e(2). Close inspection of Figures C-10 c(2), d(2), and e(2) reveals a general deepening of the Russian River. This is due to the closure of Healdsburg Dam in 1952 and Coyote Dam in 1958. The lowering of the base level in the Russian River has induced a general lowering of the base of Dry Creek.

Significant land uses and land use changes are also evident from the aerial photographs. The Dry Creek valley is agricultural, with citrus fruits as the major crop. The upland areas are rugged, consisting primarily of hilly and mountainous terrain. Urbanization has impacted the Dry Creek Basin to some extent. The town of Healdsburg, located at the downstream end of the Dry Creek Basin, has experienced some growth over the past two decades [compare Figure C-10 c(1) and d(1)]. Other land use changes include the conversion of forest lands to agricultural and grazing lands.

A review of the hydrology and hydraulics within the basin and a history of human activities in the area provide insight into the gradation problems on Dry Creek. Dry Creek is a braided stream that is overloaded with sands and gravel. The overloaded condition exists as a result of the steep upper watershed conditions, the pattern of dry summers and wet winters, and adverse land conversion that has occurred in recent decades. Recently, this condition has been aggravated by record storms and wildfire, both contributing to abnormal runoff volumes. Major fires in the Dry Creek watershed occurred in 1959, 1964, 1970, and 1973, while major floods occurred during water years 1956, 1962, 1963, 1965, 1969, and 1974. (Note that in the years following the major fires in 1964 and 1973 there was a major flood). The deposition of sediments from the overloaded condition creates gravel bar formation and aggradation of the channel bed. The presence of the gravel bars enhances lateral migration of the channel. These conditions create severe bank erosion problems because of the highly erodible bank. Bank erosion problems were detected more than 30 km upstream of the confluence with Russian River and have been documented from records and photographs as early as 1940. In addition, the similarity of degradation and lateral migration tendencies between the areas upstream and downstream of the Lambert Bridge (control) indicates the significance of natural channel instability.

The stability of the channel banks in the lower reach of Dry Creek has been complicated by the deep incision of the channel. The channel of Dry Creek downgraded significantly in the 1950s and 1960s because of the drop in the base level of the Russian River. This drop was the result of in-stream gravel mining and the construction and operation of Healdsburg and Coyote Dams. A total streambed elevation drop of 7 m has been recorded in a 0.6 km length near the Healdsburg Dam since 1940. The current Healdsburg Dam structure, completed in 1952, has maintained the past upstream bed elevation and is acting as a control that prevents upstream headcuts in Russian River. However, the base level drop in the Russian River initiated a headcut in Dry Creek that has propagated a total distance of 13 km upstream from the mouth. The headcut was controlled when rock just upstream of the Lambert Bridge was exposed in 1972-1977. Calculations indicate the headcut traveled at an estimated rate of 0.6 km/year, requiring 23 years to travel the entire 13 km. The time of completion of the headcut to the region near Lambert Bridge coincides with the time of bedrock exposure, as determined from the analysis of aerial photographs. The headcut created a deeply incised channel downstream of Lambert Bridge that has reduced the probability of flooding adjacent farmland; however, depths of flow, resistance to flow, and vegetation cover were changed such that flows with high energy passed through the channel. With the deeper channel system and with controlled flooding, higher banks became exposed to attack by the flowing water. Analysis based on the concepth of a critical slip circle indicates the bank stability will decrease with increasing bank height and instability is significantly accelerated when bank heights exceed approximately 5.2 m.

During the past three decades, the abnormal flooding and fire sequence produced record runoff and sediment that has caused the deeply incised lower ends of Dry Creek and Mill Creek (a tributary to Dry Creek) channels to begin to widen again from extensive bank erosion. Analysis of aerial photographs indicates significant increases in channel width have occurred. The lateral migration effects have been largely limited to the lower 1.6 km, in comparison with the degradation from the headcut that extended 13 km upstream.

Qualitative analysis reveals that the erosion problems along Dry Creek are complicated. The acceleration of bank erosion has been caused by (a) lowering of the base level at the confluence of the Russian River as a result of the construction of Coyote Dam and Healdsburg Dam and subsequent degradation; (b) runoff resulting from record storms, complicated by land use changes that produce a higher runoff per unit of rainfall and also by the wildfire and fire-flood sequences; (c) sand and gravel, which overload the stream, causing lateral migration; (d) removal of riparian vegetation and other agricultural practices that weaken bank stability; and (e) gravel mining activity on the Russian River and Dry Creek. The erosion problems can further be divided into degradation (vertical downcutting of the river bed) and lateral migration (bank line shifting). These two processes are mutually dependent. An erodible bed usually leads to reduced erosion of the banks. However, excessive bed degradation has also caused bank sloughing in this case where both bed and banks are highly erodible.

PRELIMINARY QUANTITATIVE ANALYSIS

Quantitative techniques were conducted to clearly define the causes of the gradation problems in Dry Creek. Calculations were made to determine the relative importance of gravel mining to the overall grade change problem. The average annual volume of sediment transported by both Dry Creek and Russian River was calculated by applying the Meyer-Peter and Muller/Einstein combined transport equations (as outlined in Case Study I). The bed material size distributions for the Russian River and Dry Creek are given in Figure C-11. Other input requirements for this technique included channel geometry (cross-section data), channel slopes, channel bed roughness, upland sediment yields, and typical flow duration curves. The actual procedure consisted of the following three stages:

- the annual flow duration curve was divided into incremental time steps defining steady flow steps;

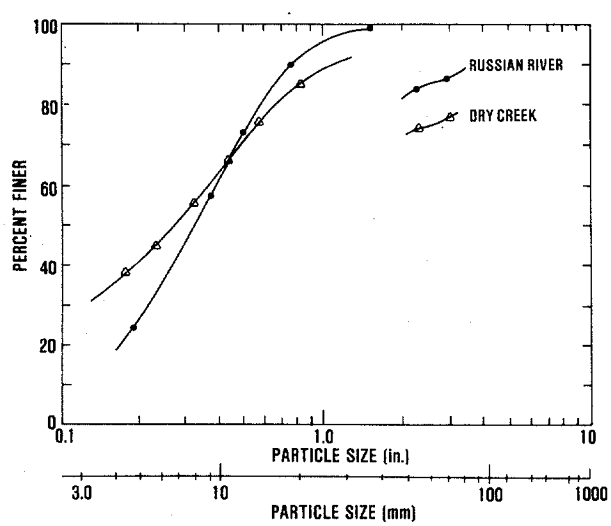


Figure C-11. MEASURE PARTICLE SIZE DISTRIBUTION FOR THE RUSSIAN RIVER AND DRY CREEK

- with the channel geometry and bed material size gradations as input, the quantity of bed material transported through the system was computed for each time period; and
- the total annual bed material sediment yield was computed by summing yields for each time increment.

The results indicated the bed material sediment yield for Dry Creek and the Russian River is large compared to that from other U.S. rivers. Dry Creek carries approximately four times more sediment than the Russian River per unit of water discharge. The Russian River supplies 380,000 to 760,000 tonnes of sand and gravel to the middle reach below the Healdsburg Dam. The estimated annual bed material sediment discharge rate for Dry Creek is 370,000-536,000 tonnes/year.

Estimates of sand and gravel extraction were available from representatives of various gravel companies and other individuals. The estimated cumulative amount of sand and gravel produced in Dry Creek (calculations discussed above) is significantly greater than the total extraction since 1961. However, a similar comparison for the Russian River indicates greater extraction than production from 1961 to 1965, resulting in net degradation in the Russian River. Data previous to 1961 indicate a large amount of material has been extracted from the Russian River, particularly during the period from 1951 to 1959. Furthermore, the construction of Coyote Dam and the presence of Healdsburg Dam limited the supply of the bed material delivered from upstream. The excess extraction rate combined with the limited supply has caused the degradation in the Russian River which has increased the local water surface slope in the mouth of Dry Creek and initiated headcutting and degradation that has significantly altered the lower end of the Dry Creek system.

The examination of precipitation records indicated that there was no abnormal rainfall input to the river system during the period of analysis. However, the evaluation of the yearly runoff-rainfall ratio verifies that the runoff per unit of rainfall had increased about 30-40 percent. This abnormal runoff resulted from accelerated land use changes, wildfire, and fire-flood sequences that increased the rate of conversion of rainfall to runoff. The removal of riparian vegetation along the bank reduced the Manning roughness coefficient about 5 to 10 percent (a change from 0.035 to 0.032).

CONCLUSIONS

The analysis documents that significant changes have occurred in this river system as a consequence of dams, reservoirs, gravel mining, land use changes, and widely varying climatic conditions. To completely analyze the future impacts of these changes, analysis procedures documented in Chapter VI should be applied. Of primary importance is lateral erosion. This type of an analysis is fundamental to the adequate design of bridges, and control structures.

CASE STUDY III: CANADA DEL ORO WASH, TUCSON, ARIZONA

Hydraulic design of bridges on braided rivers is a challenging task. Braided rivers are capable of transporting large amounts of sediment during short, dramatic flood events. The Canada del Oro Wash near Tucson, Arizona, illustrates an approach to the analysis of a river system for which a series of bridges must be considered.

PROBLEM STATEMENT

The steady growth of Tucson in recent years has extended the city into the lower potion of the Canada del Oro watershed. The old transportation system of dirt roads and road crossings is now inadequate, and the need for more reliable crossings of the Canada del Oro must be met. Urbanization is also changing the hydraulic and hydrologic characteristics of the basin. Urban landscape practice and the increase in impermeable area has decreased sediment supply during storm events. Urban land use adjacent to the main channel has increased. The channel has been narrowed and attempts have been made to control the erratic nature of the river. The river has been controlled by structural means such as channelization and grade control structures.

For this analysis, the design of four crossings was considered. The schematic diagram showing the location of the bridges is given in Figure C-12.

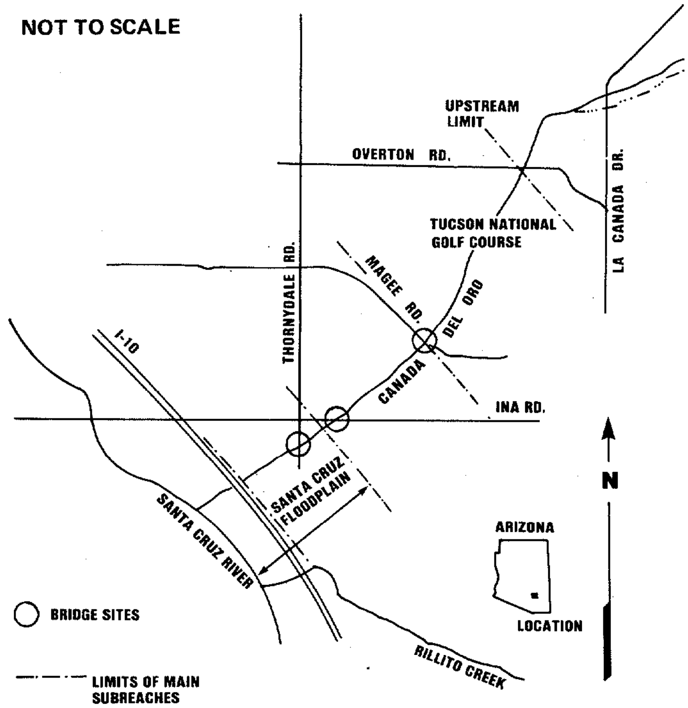


Figure C-12. CANADA DEL ORO SYSTEM

PHASE I: QUALITATIVE ANALYSIS

A qualitative analysis of the watershed system is an essential first step. By assessing the influence of geomorphic characteristics of the river and characteristics of the watersheds, the limits of the system are determined.

The Canada del Oro Wash is located in the south-central portion of Arizona just north of Tucson. The channel is generally braided and has alluvial boundaries of medium to coarse sand set in a valley of low relief. The channel is steep, dropping at a rate of 3.5 m per km, but is essentially straight, having a sinuosity of 1.1. Its flow habit is ephemeral. The channel is equiwidth and experiences only local anabranching. The channel banks are vegetated with tree and shrub cover along less than 40 percent of its bankline.

A review of human activities within the watershed provides insight into the stability of the river reach. Prior to any activity in the watershed, the river channel maintained a relatively stable graded condition. Sediment transported by upstream tributaries was transported by the main channel. The dimensions of the main channel were, on the average, fairly constant for a given reach. Individual locations would have substantial changes in shape after major runoff events, however. Depth, width, and thalweg location were seen to change during periods of high flow.

On the Canada del Oro, aerial photographs dating from 1941 were important in evaluating the river system. The 1941 photographs (unavailable for this report) predate any urban development, and no intensive land use, such as grazing, was evident. The area was covered by grasses and small shrubs and bushes typical of the arid climate. The photographs show that the channel exhibited a tendency to meander in the lower reach as it entered the Santa Cruz River floodplain. The reach has since been straightened to accommodate urban development in the area. Figure C-13 shows a reach of Canada del Oro Wash between Ina Road and Magee Road. This photograph documents extensive urbanization along the wash. A flood control channel has also diverted a number of small tributaries to this area. Further upstream, a grade control structure has been installed below Magee Road to enhance channel stability through the Tucson National Golf Course. The presence of this grade control structure has severely limited the capacity of the Magee Road Bridge. Finally, plans are being made to remove Golden Dam, located 32.2 km upstream, because of safety problems. If the dam is removed a large sediment accumulation will be available for transport.

Figure C-13. CANADA DEL ORO WASH BETWEEN INA ROAD AND MAGEE ROAD

The qualitative assessment of the Canada del Oro system indicates the following:

- because of the straightening of the lower reach and the addition of urban runoff, the response of the reach will be to degrade;
- along with the degradation, the lower reaches of the channel can be expected to meander in attempts to further reduce its slope;
- upstream of the lower reach, the channel is well within the braided range and can be expected to remain stable within its bankline.
- the grade control structure at Magee Road acts as an important control in the system. A change in the elevation of this control will have significant effects throughout the system; and
- the removal of the Golden Dam will supply excessive sediment to the system. The impact of this sediment must be considered.

PHASE II: QUANTITATIVE ANALYSIS

The next phase is a quantitative analysis.

In a dynamic river system actions taken at one location may affect conditions at other locations. The watershed-river system is bed modeled as one system. Conceptually, the system is divided into two sections: watershed and the main river channel and floodplain. The watershed areas are the major source of wash load and bed material discharge. The main river is the transporting portion of the system; channel aggradation and degradation are calculated within the river. the main river channel is further subdivided into computational reaches, which are portions of the river where sediment transport characteristics are similar. These reaches are defined in Figure C-12.

Data Requirements for Quantitative Analysis

Data requirements for the system are given in the following paragraphs.

The following hydrologic data are required:

- flood hydrographs for the 2-, 10-, 25-, 50-, and 100-year storms for the main channel and important tributaries;
- flood history of major recorded floods, including visual observations or reports that aid in documenting the response of the system; and
- a drainage basin description covering the factors that influence runoff and sediment yield (soil type, vegetal cover, hydraulic structures, and development).

The following hydraulic data are required:

- channel cross sections;
- typical channel cross sections for the tributaries; and
- plans of existing and proposed hydraulic structures in the system, including bridges, grade structures, and bank protection measures.

The following sedimentation data are required:

- sediment samples of bed and bank material;
- information on the history of base level changes;
- information on geologic conditions that control sediment yield or transport; and
- information on river management activities; these activities can be unregulated, such as gravel mining, or regulated, such as maintenance procedures to increase conveyance.

Backwater Analysis

The next phase of the analysis is a rigid boundary backwater profile analysis using the HEC-2 model to identify areas with similar hydraulic and sediment transport characteristics. A computer backwater model was chosen here because of the complexity of the system (number of bridges and control structures, and the variation in hydraulic conditions). On the Canada del Oro system, hydraulic conditions were analyzed from the I-10 bridges to Overton Road (Figure C-12) for both subcritical and supercritical flow conditions. The 100-year peak discharge of 935 m^3/sec and a discharge of 425 m^3/sec were used to bracket typical flood discharges in the system. Results of these runs showed that below Magee Road, hydraulic conditions are predominantly subcritical, while upstream conditions are predominantly supercritical. Therefore, it was necessary to divide the sediment routing process into two parts in order to correctly model hydraulic conditions throughout the system. The lower reach was linked to the upper reach by sediment and water hydrographs.

For the Canada del Oro system, hydraulic conditions dictated that the original HEC-2 data set contain cross sections at intervals of approximately 90 meters. The results of the HEC-2 runs indicated that hydraulic conditions were uniform enough that the cross-section interval could be increased to approximately 180 meters for the sediment routing model. To adequately describe channel geometry it was found that less than 20 points per section were required. This allowed for proper definition of the channel thalweg, banks, and floodplains.

Dynamic Sediment Modeling

The next step in the analysis of the Canada del Oro system is to evaluate the potential magnitude of the expected grade change. Considering the complexity of flow conditions (both subcritical and supercritical), the number of impacted crossings (four, with two additional private crossings proposed), the existence of a grade control structure at Magee Road, the potential impact the removal of Golden Dam will have on the system, and the dynamic nature of the alluvial channel boundary, a dynamic model for routing water and sediment to adequately predict the potential grade change was required. The application of a dynamic water and sediment routing model is complex and requires a broad knowledge of river mechanics as well as sufficient experience in applying computer routing techniques to water and sediment. Highway agencies may not have personnel with the required background on staff. If this is the case, the required modeling should be carried out by a qualified consultant.

For this case study, a transport similar to the one documented by Simons and Li (1979) for routing sediment by size fraction would be appropriate. It could be used to estimate the degradation and deposition in the study reaches as a function of time and discharge. This type model utilizes the hydraulic conditions determined by the HEC-2 program, and hence the results are comparable to the conventional method for determining flood levels but much more useful. The analysis is totally different from the conventional HEC-2 analysis. It recognizes that the channel bed is movable and adjustable, responding to scour and deposition. This degradation and aggradation analysis provides the necessary information to

- evaluate the stability of each bridge structure,
- determine the lateral migration tendencies of the channels,
- estimate the extent of expected general channel degradation,
- determine the potential local scour around bridge piers and abutments, and
- estimate the long-term effects of sediment degradation and aggradation on the channel bed and water surface profiles.

System hydrology is extremely important to the application of any degradation/aggradation analysis. Considering the ephemeral nature of the Canada del Oro, it is appropriate to analyze the passage of storm events as opposed to an annual flow duration curve. There is no flow in the system for the major portion of the year. However, for large events, such as a 100-year flood, the sediment capacity of the stream and the upland sediment supply will be large. For these events, sediment movement can significantly alter channel geometry by the processes of erosion and sedimentation.

The hydrologic approach for estimating a 50- or 60-year response to degradation and aggradation under these conditions is to evaluate the 2-, 5-, 10-, 25-, 50-, and 100-year floods singly and in selected combinations. These hydraologic conditions are then the input to the dynamic routing model.

Based on an input hydrologic condition, the potential degradation at each crossing can be evaluated. Using the dynamic model, the transport capacity at each cross-section is determined using the hydraulic conditions provided from a HEC-2 type flow model, and a sediment transport equation (in this case the Meyer-Peter and Muller equation for steep, sand and gravel bed channels). Sediment routing is accomplished by applying the sediment continuity equation [Equation (72)], considering the upstream sediment supply by size fraction and the bed material size distribution. Predicted amounts of aggradation and degradation are distributed according to channel conveyance values. Local scour is then evaluated using appropriate equations based on hydraulic conditions at the crossing after degradation.

CASE STUDY IV: POLO CREEK AT I-90 NEAR WHITEWOOD, SOUTH DAKOTA

This case study documents the use of quantitative geomorphic and engineering relationships to evaluate the potential for a grade change at the I-90 crossing over Polo Creek near Whitewood, South Dakota. A general background to the problems at this site can be found in Case History 109 (Appendix A). The purpose of this example is to demonstrate a technique for analyzing grade change problems at sites where time, money, or other constraints will not permit the application of modeling techniques. Because of project limitations, some of the information used in this example might not reflect site conditions. This case is only meant to be used as an example and should not be used for any design work at the I-90 site.

PHASE I: QUALITATIVE ANALYSIS

Channel and Watershed Classification

Polo Creek is located near Whitewood, South Dakota. Polo Creek originates in the Black Hills of South Dakota and is tributary to False Bottom Creek about 1.24 km north of the I-90 crossing. Just upstream of the crossing, a tributary, Miller Creek, has its confluence with Polo Creek. At this site, the channel is on an alluvial fan with its boundaries poorly defined. The channel has alluvial boundaries composed of gravel and cobble size particles; channel banks are composed of sand, gravel, and cobbles as well as small amounts of clay. Observations from aerial photography, topographic maps, and field observations permitted adequate classification of the site and drainage systems. Channel and watershed classifications are presented in Tables C-3 and C-4, respectively. Figure C-14 shows a topographic map of the study site.

Table C-3. CHANNEL CLASSIFICATION – POLO CREEK

KEY COMPONENT	DESCRIPTOR
Channel Width	Small 3 - 6 m
Channel Boundaries	Alluvial
Bed Material	Gravel/Cobble Some Sand Minor Amounts of Silt and Clay
Banks	Sand/Gravel/Cobble Small Amounts of Clay Most Cohesion from Root Systems (Trees/Grass)
Valley Setting	Site Located on Alluvial Fan (No Valley) U/S and D/S Valley Ranges to 150 m and Is Moderate
Floodplain	Wide At Site. Narrow in U/S and D/S Valleys
Degree of Sinousity	Sinuous (1.12)
Degree of Braiding	Generally
Variability of Width	Equiwidth
Bar Development	None
Cut Banks	Local
Tree Cover on Banks	$<$5% Generally
Vegetation	Mostly Grass with Few Trees

Table C-4. WATERSHED CLASSIFICATION – POLO CREEK

KEY COMPONENT	DESCRIPTOR
Geometry	Drainage Area = 5.2 km^2 Channel Slope = 0.082 → 0.0083 Valley Slope = 0.1 → 0.01 Channel Length = 16.25 km Average Channel Width = 3 m Average Bankfull Depth = 0.6 - 0.9 m Sinuousity = 1.12 Channel Characterized as Narrow, Steep, Deep As It Flows Through the Black Hills. Approaching I-90 Site, System Flattens Out onto an Alluvial Fan. Basin Fan Shaped and Steep; Channel Poorly Defined on Fan.
Soil	Mostly Sand, Gravel, and Cobbles to a Depth of 3.7 m. 48% of Material (by weight) Larger Than Sand Size Particles (Will Not Pass #10 Sieve); 34% Sand Size Particles; 18% Smaller Than Sands. Erodible
Geology	Crystalline Rocks (Including Quartzite, Granite) and Sandstone. Some Deposits of Gypsum and Anhydrite.
Vegetation	Predominently Pine Timberland Small Amounts of Understory 60% Canopy Cover 40% Ground Cover
Climate	Yearly Average Precipitation: ~50 cm Average January Temperature: -7 °C Average July Maximum Temperature: ~22 °C
Hydrology	Mostly Surfacewater Flow with a Little Groundwater Flow Torrential Type Stream (Flashy) Ephemeral Quick Response Times Due to Steepness of Topography
Sedimentation	Erosion Mainly Due to Channel Degradation in Steep Upstream Reaches Some Surface Erosion in Upland Areas Bed Load Is Predominant Mode of Transport
Man's Influence	Construction of Routes 85 and 90 Only Development Activities Some Logging Activities in Area Minimal Impacts Some Debris and Log Stockpiles in Floodplain

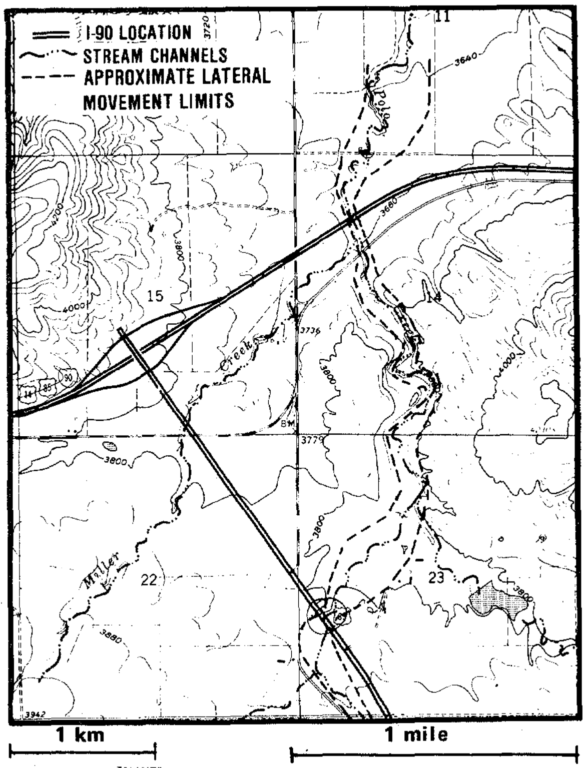


Figure C-14. LOCATIONS OF POLO CREEK AND MILLER CREEK

Channel slopes and drainage areas were measured from topographic maps of the entire watershed (not shown).

Observations from a field survey of the creek in the area of the crossing revealed additional information not included in the general classifications. First, it was observed that a significant number of trees are located in the channel. Besides increasing the channel's roughness the trees act to deflect flow contributing to the channels lateral instability. The presence of trees also indicates the tendency of the channel for lateral movement. In fact, remnants of a previous channel are apparent to the west. Sections of the channel are poorly defined. During periods of no flow it becomes difficult to distinguish the low flow channel. Numerous stockpiles of logs and debris were observed both upstream and downstream of the bridge crossing. This indicates the potential for debris problems at a crossing in this area.

Stability Considerations

The aerial photography and maps reviewed (both historic and current) as well as information from field surveys indicate Polo Creek to be relatively stable in the mountainous region in the Black Hills. However, upon exiting the steep walled valleys of the Black Hills, the creek fans out onto an alluvial plain where the channel experiences significant lateral movement. A similar situation exists on Miller Creek, which has its confluence with Polo Creek just upstream of the I-90 crossing site.

An analysis of the valley slopes for Polo Creek indicates that the alluvial fan in the region has been building over a long period of time. This is evident from the shape of the channel profile shown in Figure C-15 (plotted from contours on a current topographic map), which is concave upward and does not indicate an abrupt change of slope as the channel exits the steep Black Hills region. A young alluvial fan would show a more distinct slope break where it reaches the valley floor and presents a convex (concave downward) profile typical of aggrading reaches. Figure C-15 also shows a profile for Miller Creek.

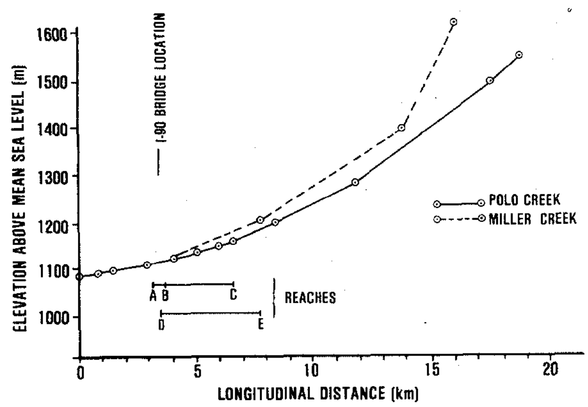


Figure C-15. CHANNEL PROFILES, POLO CREEK AND MILLER CREEK

Observations also indicate aggradation problems in the vicinity of the proposed crossing. Channel depths get extremely shallow just downstream of the site, indicating aggradation. Also, several reaches of the channel are poorly defined, which is characteristic of aggrading channels. It is also evident that the channel has shifted from an apparently abandoned channel near the western edge of the floodplain to its present location. Aggrading channels often shift their positions on a floodplain as the channel fills and loses capacity.

To assist in the understanding of the processes taking place in the Polo Creek watershed, Lane's relationship as given in Equation C-1, was evaluated. Figure C-16 depicts the mechanics of alluvial fan development and is used to describe the changes predicted from Lane's equation.

$$Q\,S \sim Q_s\,D_{50} \qquad \text{(C-1)}$$

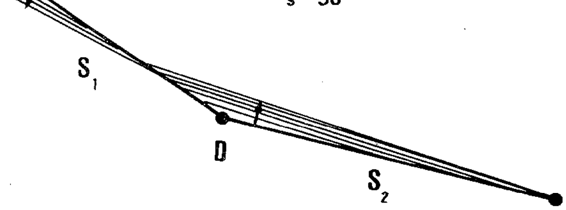


Figure C-16. DEVELOPMENT OF AN ALLUVIAL FAN

Initially the section of Polo Creek in the Black Hills has some steep slope, S_1, and the valley some milder slope, S_2. The steep slope, S_1, produces the potential for some sediment carrying capacity, Q_{S1}. From Lane's relationship, as the slope decreases from S_1 to S_2 the sediment load, Q_{S1} must also drop to some lower value, Q_{S2}. To satisfy the proportionality in Equation 72, the excess sediment load (Q_{S1} - Q_{S2}) carried from the Black Hills will be deposited on the valley floor starting at the slope break point indicated as Point D in Figure C-16. The deposition process will continue until the slope on the valley floor is sufficient to carry the excess sediment load carried by S_2. In time the sediment load transported from the Black Hills will decrease as degradation reduces S_1 and/or as the available upland sediment supply is depleted; in the same time, the transport capacity on the valley floor will increase as deposition increases S_2. An equilibrium condition will be reached when the magntiude of S_1 and S_2 are such that the materials eroded from the upland areas can be transported through both reaches. This condition will be reached when both slopes, S_1 and S_2, become equal or when armoring effects reduce the availability of material in transported size ranges below that of the waters available transport capacity in the upland areas so that some lower slope on the alluvial fan will be able to transport all the material supplied. This last condition is the case most often encountered.

Note that there is more involved here than just a change in slope. Discharges, channel sections, and other hydraulic parameters will also change. However, the explanation just given does indicate the existence of an aggradation problem at the proposed bridge site.

PHASE II: QUANTITATIVE ANALYSIS

The above explanation centered on a qualitative analysis of channel stability at the I-90 crossing site. The potential for continued aggradation in the reach is apparent. The magnitude of the aggradation problem will next be evaluated using quantitative engineering techniques.

The approach for the quantitative analysis will be to apply geomorphic and engineering relationships to examine the potential bed volume change in the vicinity of the I-90 crossing. This approach will apply the sediment continuity equation presented in Equation (72) to evaluate the magnitude of the anticipated aggradation problem at the crossing. This requires the analysis of the incoming sediment load as well as the transport capacity within the local reach. The procedure requires the analysis of sediment transport within three reaches; transport within the local 450-meter reach at the crossing must be considered as well as transport in the next upstream reach. The upstream reach defines the sediment load supplied to the crossing reach. The sediment yield from Miller Creek must also be considered. The analysis of the transport capacity of a reach of Miller Creek just upstream of the confluence with Polo Creek will be used to define this sediment yield. Implied in this discussion is the assumption that available upstream sediment supply is equal to the capacity of the channel to transport sediment.

Data Requirements

Before a quantitative analysis can be conducted, it is necessary to obtain the required field data and conduct some preliminary analysis. Data requirements include channel geometry, channel bed profiles, sediment samples, channel roughness estimates, and discharge data. The required preliminary analysis includes evaluation of channel geometric properties, evaluation of flow resistance, analysis of local hydrology, and determination of flow type.

Channel Geometry

The reaches of interest are documented in Figure C-15. As shown, Reach $\overline{AB}$ is the 450-meter reach centered on the I-90 crossing; Reach $\overline{BC}$ is the reach just upstream on Polo Creek that is used to define the influent sediment yield from Polo Creek: Reach $\overline{DE}$ is the reach on Miller Creek just upstream of the confluence that defines its sediment yield. Approximate channel sections for these reaches are presented in Figure C-17. These sections were approximated from the survey data. Trapezoidal sections were used to simplify computations. Figure C-18 presents the geometric properties of these sections in terms of depth vs. $AR^{(2/3)}$ curves. Channel slopes for these reaches are given in Table C-5.

Sediment Size Distributions

Several bed material samples were taken in each of the three reaches under consideration. These samples were averaged and composite samples developed for each reach. Reaches $\overline{AB}$ and $\overline{DE}$ have very similar size distributions, while Reach $\overline{BC}$ is more coarse. From the average size distributions, D_{50}, D_{90}, and D_m were determined. These values are given in Table C-5.

Channel Roughness

Channel roughness estimates were made using Manning's roughness coefficient. Four such coefficients are important to the analysis of sediment transport in sand/gravel bed rivers: particle roughness (n_p), Channel bed roughtness (n_b), channel sidewall roughness (n_w), and average flow roughness for the entire channel (n_m). Particle roughness estimates were made based on the Strickler and Lane/Carlson equations presented as Equations 2 and 3, respectively. Channel bed roughness was based on considerations of particle roughness and bedform roughness (dunes form during most significant flow events based on Figure 32 in the main text). Sidewall roughness is based on particle roughness, degree of bank vegetation, and the presence of obstructions near the bank. The average channel roughness is a composite of the other roughness values. The channel roughness coefficients just described are listed in Table C-5.

Hydrologic Analysis

On ephemeral streams in arid regions grade changes often come as a result of major runoff events. However, because of the importance of snowmelt runoff from the Black Hills, it is also important to

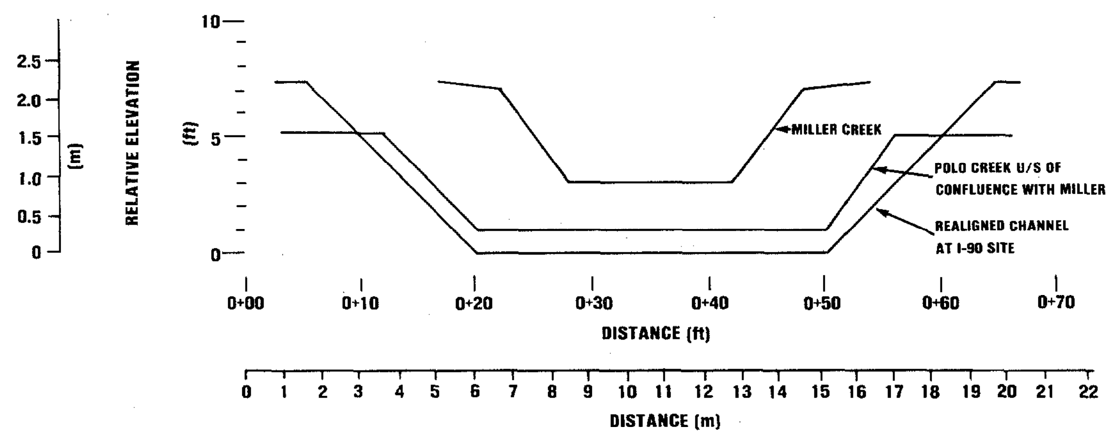


Figure C-17. CHANNEL SECTIONS, POLO CREEK AND MILLER CREEK

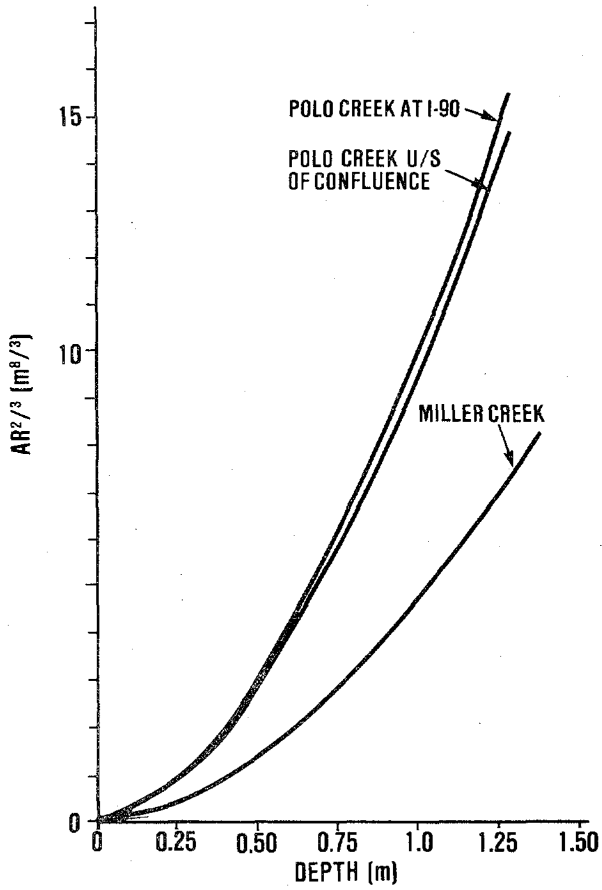


Figure C-18. GEOMETRIC PROPERTIES, POLO CREEK AND MILLER CREEK

consider grade changes brought about by longer-term runoff events. Therefore, both flow duration curves and flow frequency curves are required for the analysis.

Flow duration curves were developed for each of the three reaches based on gaged flow data. Since there are no streamflow gages on Polo Creek, gage data from Coolidge Creek near Custer, South Dakota, were used. Coolidge Creek is located in the same geographic region as Polo Creek and is assumed to exhibit the same runoff characteristics. Flow duration data from Coolidge Creek were adjusted using the ratio

$$\left(\frac{Q}{Q_T}\right) = \left(\frac{A}{A_T}\right)^n$$

where

Q = gaged flow rate,

Q_T = flow rate from ungaged area,

A = tributary area for gaged flow rate,

A_T = tributary area for ungaged flow rate, and

n = a regional exponent = 0.54 in this case.

The flow duration curves for Polo Creek are shown in Figure C-19.

Flow frequency data for Polo Creek were based on "A Method for Estimating Magnitude and Frequency of Floods in South Dakota" published by the U.S. Geological Survey. The flow frequency curves developed using this method are presented in Figure C-20. Based on this figure, the 50-year design discharge is 62.3 m^3/sec, 51.0 m^3/sec, and 11.3 m^3/sec for reaches $\overline{AB}$, $\overline{BC}$, and $\overline{DE}$, respectively. Based on the flow frequency analysis and unit hydrograph considerations, design hydrographs were developed based on a 50-year 24-hour rainfall of approximately 8.9 cm and also include the influence of snowmelt runoff. These hydrographs are shown in Figure C-21.

Quantitative Stability Considerations

Although the qualitative analysis concluded that the I-90 crossing site was susceptible to aggradation problems, a more quantitative anlaysis of channel stability is required to verify this conclusion. Incipient motion concepts are reviewed in Chapter V. For sand/gravel bed channels such as Polo Creek, Shields' critical shear stress criterion is used. The evaluation of bed shear stresses is based on Equation (23) and Figures 34 and 35, assuming flow to be fully turbulent. For the banks, the critical shear is computed considering the gravitational forces as well as those produced by the flowing water. The appropriate relationship for such a consideration is

$$\frac{\tau_c^{\,w}}{\tau_c^{\,b}} = \cos\theta \left[1 - \frac{\tan^2\theta}{\tan^2\phi}\right]^{1/2}$$

Table C-5. POLO CREEK DATA
(Related to Figure C-15)

PARAMETER	REACH $\overline{AB}$	REACH $\overline{BC}$	REACH $\overline{DE}$
Slope	0.0085	0.0104	0.0206
Channel Roughness			
n_p	0.023	0.025	0.023
n_b	0.030	0.033	0.033
n_w	0.040	0.050	0.050
n_m	0.032	0.035	0.035
Sediment Size (mm			
D_{50}	12.2	13.5	12.2
D_{65}	17.0	26.0	17.0
D_{90}	44.5	46.6	44.5
D_m	18.6	23.0	18.6

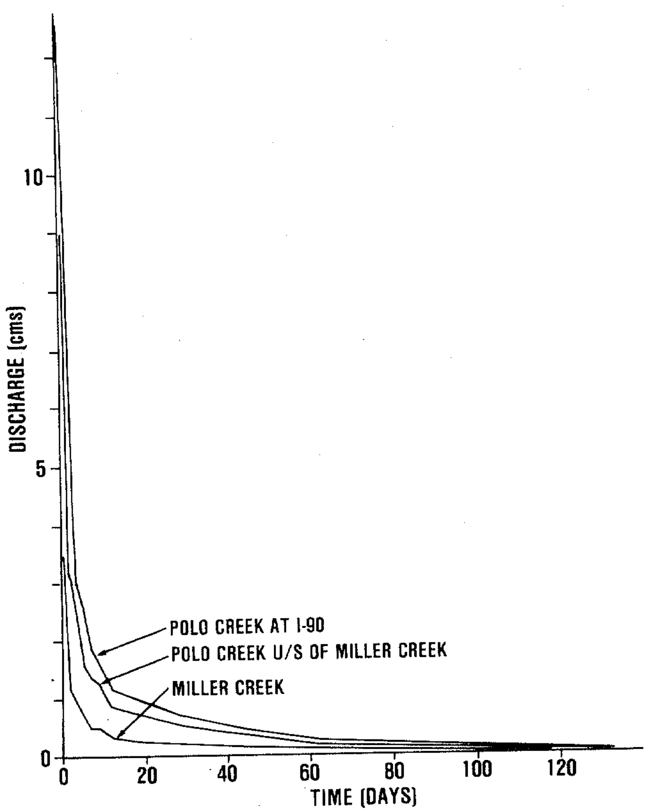


Figure C-19. FLOW DURATION CURVES, POLO CREEK AND MILLER CREEK

where

τ_c^w = critical shear stress at the bank,

τ_c^b = critical shear stress on the bed,

θ = angle of bank to horizontal, and

ϕ = angle of repose of bank material.

Based upon the channel geometry in the vicinity of the crossing, both bed and bank material are susceptible to transport if the discharge exceeds 1.4 m^3/sec. The flow duration curve presented in Figure C-19 indicates that flows exceed this value on an average of 10 days out of every year. During this period both the bed and banks are susceptible to vertical and lateral movement.

Analysis of Bed Material Volume Change

The analysis procedure for determining the amount of aggradation or degradation at a crossing site is discussed in Chapter VI. The procedure is here applied to both the flow duration curve and the design hydrograph.

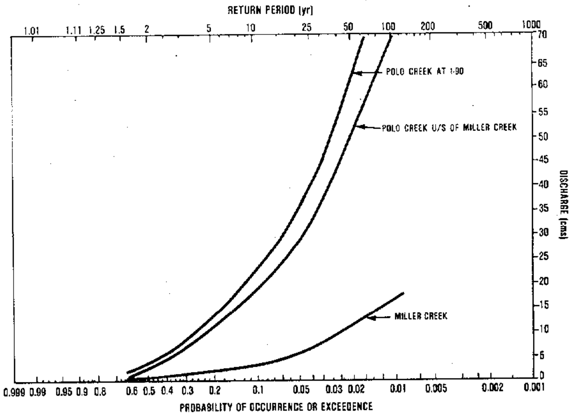


Figure C-20. FLOW FREQUENCY DIAGRAMS, POLO CREEK AND MILLER CREEK

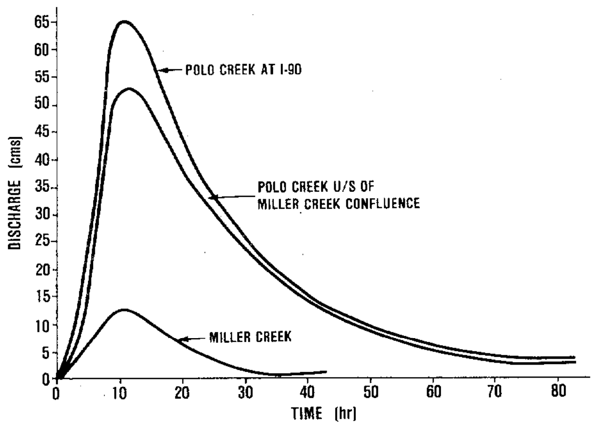


Figure C-21. FIFTY-YEAR FLOOD HYDROGRAPHS, POLO CREEK AND MILLER CREEK

The first step in the analysis is to divide the flow duration curve and design flow hydrographs into incremental time steps of assumed constant discharge. The time steps and discharges for the flow duration curve and design flow curve are given in Tables C-6 and C-7 for each of the three reaches under consideration. The flow depths given for each of the discharges in Tables C-6 and C-7 are computed using Manning's resistance formula [Equation (4)]. Based on Froude number considerations, it was found that for the range of flows being considered, reaches $\overline{AB}$ and $\overline{BC}$ are subcritical while reach $\overline{DE}$ (Miller Creek) is supercritical.

With the channel geometry and bed material size data as input, the second step of the analysis is to compute the transport volume within each reach for each incremental discharge. The equation used is the Meyer-Peter and Muller equation presented a Equation (34). In this case, the only change made to the equation was the substitution of $1/n_s$ for K_s; thus, the term

$$\left(\frac{K_s}{K_r}\right)^{3/2}$$

became

$$\left(\frac{D_{90}^{\ 1/3}}{26\ n_s}\right)^{3/2}$$

because

$$K_r = 26/(D_{90})^{1/3}$$

and

$$K_s \sim 1/n_s$$

With A" = 0.047, B" = 0.25, $g = 9.82\ m/sec^2$, γw = 1 $tonne/m^3$, and γ_s = 2165 $tonnes/m^3$, Equation (34) was reduced to

$$q_{bw} = 25\ B\left[\frac{Q_s}{Q}\left(\frac{D_{90}^{\ 1/3}}{26\ n_s}\right)^{3/2} Y\ S - 0.0776\ D_m\right]^{3/2}$$

where

B = channel width (meters) and

q_{bw}= transport in tonnes/sec.

Also, Richardson et al. (1974) reports that for trapezoidal channels

Table C-6. INCREMENTAL FLOW DISCHARGES AND DEPTHS FOR THE AVERAGE ANNUAL FLOW DURATION CURVE (Related to Figure C-15)

Δ TIME (days)	REACH $\overline{AB}$		REACH $\overline{BC}$		REACH $\overline{DE}$	
	Q (m^3/sec)	Y (m)	Q (m^3/sec)	Y (m)	Q (m^3/sec)	Y (m)
1	12.74	0.64	9.42	0.53	3.31	0.37
1	8.49	0.50	6.23	0.41	2.26	0.29
1	4.1	0.32	3.06	0.26	1.05	0.16
1	3.64	0.30	2.60	0.23	0.91	0.15
1	2.83	0.29	2.04	0.21	0.79	0.13
2	2.18	0.20	1.58	0.18	0.59	0.12
5	1.50	0.18	1.10	0.13	0.40	0.08
16	0.91	0.09	0.68	0.08	0.23	0.06
15	0.57	0.06	0.42	0.05	0.14	0.05

Table C-7. INCREMENTAL FLOW DISCHARGE AND DEPTHS FOR THE 50-YEAR DESIGN DISCHARGE CURVE (Related to Figure C-15)

Δ TIME (hours)	REACH $\overline{AB}$		REACH $\overline{BC}$		REACH $\overline{DE}$	
	Q (m^3/sec)	Y (m)	Q (m^3/sec)	Y (m)	Q (m^3/sec)	Y (m)
5	8.49	0.50	5.66	0.40	2.83	0.35
2.5	34.53	1.10	26.60	0.98	7.92	0.62
2.5	60.85	1.40	48.96	1.33	11.89	0.75
2.5	64.81	1.45	52.36	1.37	12.45	0.78
2.5	61.69	1.41	50.94	1.35	10.75	0.73
5	49.81	1.30	42.45	1.25	7.36	0.58
5	37.64	1.15	33.39	1.11	4.25	0.43
5	28.87	1.01	26.89	0.99	1.98	0.26
10	18.68	0.79	18.11	0.79	0.57	0.12
10	11.32	0.59	10.75	0.59	0.57	0.12
10	7.08	0.45	6.51	0.43	0.57	0.12
10	4.53	0.34	3.96	0.30	0.57	0.12

$$\frac{Q_s}{Q} = 1 \Big/ \left[1 + \frac{2Y(1+H^2)^{1/2}}{B}\left(\frac{n_w}{n_s}\right)^{3/2}\right]$$

where

H = horizontal distance for one unit of vertical distance.

The total volume of bed load transport during each time step within each reach for both the flow duration curve analysis and 50-year frequency flood analysis are given in Tables C-8 and Table C-9, respectively.

In Step 3 the sediment continuity equation is applied at each time step to compute the amount of material eroded or deposited during each time interval. This was accomplished by summing the transport from Reach $\overline{BC}$ and $\overline{DE}$ and subtracting the material transported through Reach $\overline{AB}$. The resulting values are tabulated in Column 5 of Tables C-8 and C-9.

In Step 4, the values in Column 5 of Tables C-8 and C-9 are summed to find the total volume of material deposited in reach $\overline{AB}$ during an average year and during a 50-year flow event. These values are 660 and 1940 tonnes, respectively.

The last step is to estimate the depth of deposition based on channel geometry and typical bed material in-place density. An analysis of sediment samples from the bed of Dry Creek reveals that its in-place density is approximately 1.5 tonnes per

Table C-8. BED LOAD TRANSPORT FOR AVERAGE ANNUAL FLOW DURATION CURVE (Related to Figure C-15)

TIME (days)	TRANSPORT (tonnes)			DEPOSITION REACH $\overline{AB}$ (tonnes)
	$\overline{AB}$	$\overline{BC}$	$\overline{DE}$	
1	885	698	687	500
2	507	270	421	185
3	46	0	30	- 16
4	15	0	10	- 4
5	5	0	0	- 5
7	0	0	0	0
12	0	0	0	0
28	0	0	0	0
43	0	0	0	0

Table C-9. BED LOAD TRANSPORT FOR 50-YEAR FLOOD HYDROGRAPH (Related to Figure C-15)

TIME (hours)	TRANSPORT (Tonnes)			DEPOSITION IN REACH $\overline{AB}$ (tonnes)
	REACH $\overline{AB}$	REACH $\overline{BC}$	REACH $\overline{DE}$	
5.0	106	46	132	73
7.5	166	205	144	183
10.0	168	234	165	232
12.5	166	234	168	236
15.0	168	234	162	229
20.0	340	465	272	397
25.0	337	446	185	295
30.0	318	413	66	161
40.0	512	644	0	132
50.0	318	369	0	51
60.0	154	133	0	- 20
70.0	31	3	0	- 27

m^3. Deposition depths were computed using this value and the following relation describing the variation of volume with depth:

$$V = 2LY^2 + BLY,$$

where

V = reach deposition volume,

L = reach length,

Y = deposition depth, and

B = bottom width of section.

Based on the average flow duration curve and assuming deposition over the entire 460 m reach in the vicinity of the crossing, the expected deposition depth is 0.10 m. If deposition is assumed over only half of the reach, the depth is 0.29 m. Considering the 50-year flow event, the total expected deposition depth over the entire reach is 0.29 m. If deposition is limited to half the reach, its depth will be 0.55 m. Referring to Table C-9 it is noted that during the last two time steps, reach $\overline{AB}$ degraded slightly. Therefore, the total amount of deposition computed was not the maximum deposition. The maximum amount of deposition was 1986 tonnes. This would result in an aggradation depth of 0.3 m over the entire reach and 0.56 m over half the reach. This analysis necessarily assumes a uniform deposition depth.

Lateral Migration

As discussed earlier, the channel of Polo Creek is susceptible to lateral migration and has been observed to jump its channel occasionally. Lateral migration on alluvial fans is a common occurrence. As the channel capacity is filled, the stream will attempt to move to some new location. Evidence of this action was revealed during the preliminary qualitative analysis.

The lateral extent of the movement of Polo Creek in the vicinity of the I-90 crossing will be controlled by the ridges on each side of the channel just upstream of the crossing. Flow in this area is also controlled by the I-90 and Route 114 highway embankments. In this case the extent of lateral movement is confined by physical controls. Estimates of maximum lateral movement are shown on Figure C-14.

Conclusions

The analysis presented above reveals that for an average year, aggradation at the I-90 crossing site will not cause major problems. However, the amount of aggradation produced by one or a series of flood events will impact the crossing. The problems here do not warrant structural control measures. A well managed maintenance program designed to monitor the site and excavate deposited material after major runoff events would be sufficient.

CASE STUDY V: MOSQUITO CREEK

Cohesive, clay-type soils are encountered in many regions of the country, and this case study documents the use of geomorphic and engineering techniques to evaluate the potential magnitude of grade changes in cohesive soils. This study is based on background information from Case History 34 (contained in Appendix A), which describes degradation problems at Iowa Highway 191 crossings over Mosquito Creek and its tributaries. The purpose of this study is to outline a procedure for approaching the analysis of degradation problems in cohesive materials. This is only meant as an example, and at times the information and data presented might not reflect actual conditions at the site. Therefore, the data used as a part of this example should not be used for design.

PROBLEM STATEMENT

Mosquito Creek was involved in extensive channel straightening in the late 1930s and channel degradation began shortly thereafter. Iowa Highway 191 parallels Mosquito Creek for more than 80 km, crossing it and its tributaries periodically. These crossings were designed and constructed during the 1930s and early 1940s without knowledge or anticipation of future grade changes within the system. By the mid-1960s degradation problems were evident at almost all of the I-191 stream crossings. This implies that an analysis of the entire Mosquito Creek system should be undertaken to evaluate the potential for further degradation and its impacts on each crossing. However, to simplify this example, only two crossings will be considered. The approach taken for these crossing could easily be extended to the entire system.

The two I-191 crossings being considered are located between Panama and Earling, Iowa. Figure C-22 shows their relation to each other. The bridges will be referred to as downstream and upstream. Degradation at these crossings between 1940 and 1961 is shown in Figure C-23. In that period, the upstream bridge experienced 3.35 m of degradation and the downstream bridge 4.33 m. Note the exposure of pier foundations in both cases.

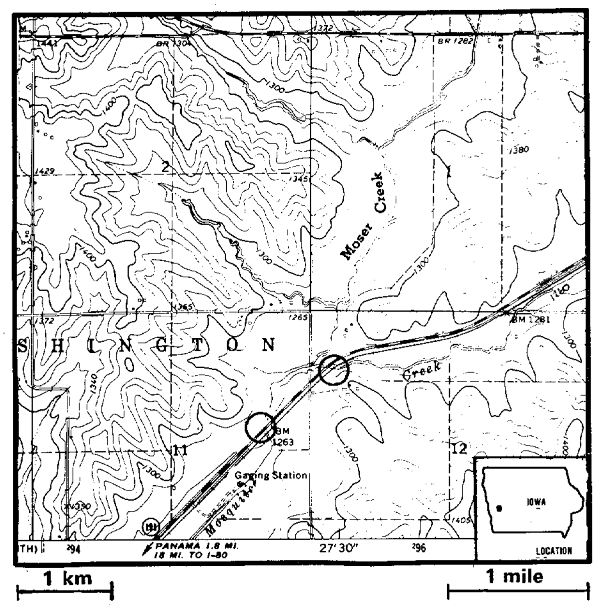


Figure C-22. LOCATION OF I-191 CROSSINGS OVER MOSQUITO CREEK

PHASE I: QUALITATIVE ANAYSIS

Channel and Watershed Classification

Mosquito Creek flows through Shelby, Harrison, and Pottawattamie Counties, Iowa, on its way to join the Missouri River at Council Bluffs, Iowa. The topography of the area is characterized by rolling hills and bluffs composed of loess type soils (D_{99} = #200). The area is farmed extensively with only small areas of homes and trees that are not under cultivation. As mentioned above, Mosquito Creek was straightened dramatically in the 1930s to provide additional farm land along its banks and reduce flooding problems. This straightening shortened the overall length of Mosquito Creek by 20 percent, from 110 km to about 88 km.

The two bridges of interest here are located at a point where Highway 171 and the railroad that parallels it cut across a bend in Mosquito Creek. In all, five bridges cross Mosquito Creek in the space of 0.5 km at the site: the two I-191 bridges, two railroad bridges, and a county road bridge (Figure 58 of the main text). The channel here, as well as along the entire length of Mosquito Creek is cut into alluvium composed of water- and wind-deposited loess-type soils. Channel banks are nearly vertical

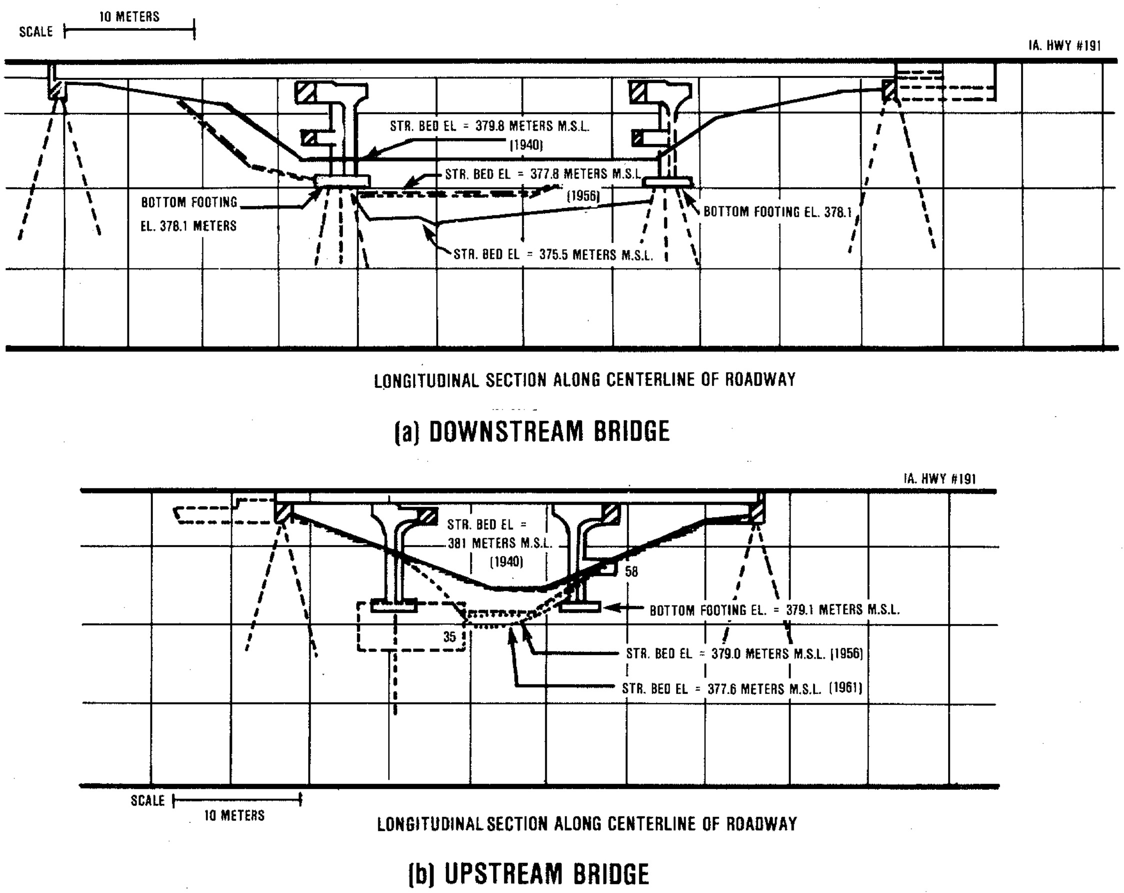


Figure C-23. DEGRADATION AT IOWA HIGHWAY I-191 BRIDGES BETWEEN PANAMA AND EARLING

except where they have been undercut by the degradation and are slumping. Channel banks in loess-type soils also have a tendency to slump when they become water-soaked. Vegetation along the channel banks consists of a line of trees and brush at the top of banks and grass cover on some slumped banks.

Channel and watershed classifications are presented in Tables C-10 and C-11. Channel and valley slopes were obtained from topographic maps of the watershed. A composite typical channel cross-section is given in Figure C-24.

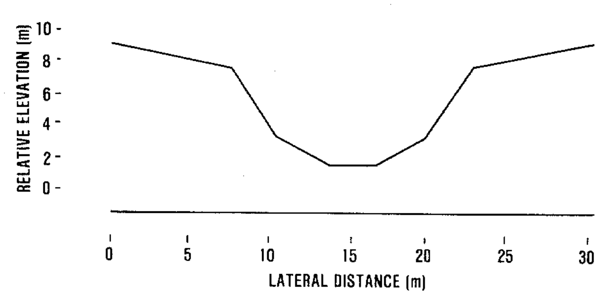


Figure C-24. TYPICAL CROSS SECTION OF MOSQUITO CREEK

Application of Lane's Relationship

Although the channel's instability is evident from field observations, Lane's relationship, presented in Equation (C-1), is applied to verify the cause of degradation. Lane's relationship predicts the type of problems that have occurred on Mosquito Creek as a result of the channel straightening.

Before the channelization and straightening were done on Mosquito Creek, it was stable in that the

Table C-10. MOSQUITO CREEK CHANNEL CLASSIFICATION

KEY COMPONENT	DESCRIPTION
Flow Habit	Perennial But Flashy
Channel Boundaries	Alluvial – loess
Bed Material	Silt - Clay
Valley Setting	Moderate Relief 60-90 m
Floodplain	Wide
Sinuosity	Artifically Straightened Historic – Sinuous, ~1.25
Braiding	Not Braided
Anabranching	Not Anabranched
Width Variability	Equiwidth
Incision	Deeply Incised
Bank Height	6 - 15 m
Cut Banks	General
Bank Material	Losses/Alluvium (Cohesive)
Vegetal Patterns	Narrow Belt of Vegetation along Both Banks

channel bed was not degrading or aggrading. The major source of sediment was runoff from the adjacent land, most of which is farmed. After straightening, the slope became steeper because of the reduced channel length. Other factors in Equation (C-1), median particle size D_{50} and discharge Q, can not change because they are related to the location, land use, and climate. Thus, in order to balance the relationship in Equation C-1, the amount of sediment (Q_s) carried by the creek must increase.

Because the land use in the watershed did not change after the straightening, the average sediment contribution from runoff into the creek also remained fairly constant. Another source of sediment to balance Lane's relationship had to be found and, as a result, the channel bed began eroding to supply the required sediment. This erosion will continue until the streambed slope hs been reduced to a point where the variables in Equation (C-1) are again in balance.

The discussion just given considered changes over the entire watershed. By looking more specifically at the local reach, the problems at the two crossings under consideration can be better defined. It is important to note that the reach of Mosquito Creek between the two bridges was not extensively straightened as were the reaches upstream and downstream of the crossings. Therefore, the channel straightening created steep sloped channels upstream and downstream as shown by the heavy line in Figure C-25.

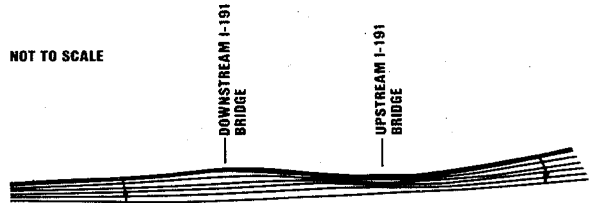


Figure C-25. DEGRADATION PROCESS WITHIN LOCAL REACH, MOSQUITO CREEK

The two steeper reaches upstream and downstream of the crossings will tend to degrade as the discussion of Equation (C-1) (Lane's relationship) revealed. The middle reach between the bridges will initially remain stable. As the downstream reach degrades, the reach between the bridges will steepen. This process will continue until all three reaches attain some stable slope configuration (Figure C-25).

Other Observations

Periodic site inspections at both bridges revealed the degradation problems. Channel cross-section surveys were made at both bridges in 1940, 1956, and 1961. These channel sections reveal that the slope of the reach between the two bridges changed from 0.0018 in 1940 to 0.0020 in 1956 to 0.0035 in 1961. This gradually increasing slope verifies the degradation process documented in Figure C-25.

It is also known that the slope of the straightened reaches was about 0.0018 before straightening and 0.0033 after being straightened.

Field observations also revealed that there are now significant vertical channel controls on Mosquito Creek to control its base level besides the resistance of the bed material itself. Therefore, an analysis of

Table C-11. MOSQUITO CREEK WATERSHED CLASSIFICATION

KEY COMPONENT	DESCRIPTION
Location	Between Panama and Earling in Shelby County, Iowa
Geometry	Drainage Area to Structure = 83 km^2 Valley Slope = 0.014 Channel Length = 88 km Average Channel Width = 15 - 27 m Average Bankfull Depth = 0.6 - 1.5 m Sinuosity Before Channelization = 1.25 Sinuosity after Channelization = 1.05 Channel Is Deeply Incised from Degradation due to Channel Straightening in 1930s.
Soil	Windblown Loess Predominantly, D_{99} = - #200 , Cohesive, Erodible, Will Form a Vertical Face. Typically 6 - 15 m deep.
Vegetation	Extensive Farming and Grassland. Areas of Trees and Brush along Streambank. 97% Ground Cover $<$3% Canopy Cover
Climate	Average Yearly Precipitation $\approx$76 cm Snowfall $\approx$10% of Yearly Total Spring Rain $\approx$30% of Yearly Total Summer Rain $\approx$50% of Yearly Total Fall Rain $\approx$10% of Yearly Total Average January Temperature: -5°C Average July Temperature: 25°C
Hydrology	Perennial But Flashy Surface Flow with Some Groundwater
Sediment Yield	Surface and Soil Erosion from Cultivated Areas. Some Bank Sloughing and Channel Erosion As Creek Bed Degrades. Bedload and Suspended Load Transport of Silt Sized Particles.
Man's Influence	Channel Straightening Causing Degradation. Increased Erosion from Cultivated Areas.

the shear resistance of the various vertical clay layers will be required to determine the maximum depth of degradation.

QUANTITATIVE ANALYSIS OF POTENTIAL ADDITIONAL DEGRADATION

The analysis technique chosen for this case study is a technique that requires the application of engineering techniques as opposed to modeling techniques. Modeling techniques were not used because of the uncertainties and complexities involved with trying to predict transport rates in cohesive materials. To adequately predict these transport rates requires detailed laboratory analysis to determine the properties of bed and bank materials as well as the chemical composition of the water.

The approach used to determine the expected amount of degradation was to analyze the vertical change in critical shear stress of the layers of clay in core samples taken along Mosquito Creek. These critical shear stresses are then compared to actual shear stresses produced by flows typical to the particular reach of interest. This approach allows the determination of an anticipated maximum depth of degradation. Here, the procedure is only applied to the reach of Mosquito Creek between Panama and Earling, Iowa. This approach could easily be extended to the entire length of Mosquito Creek.

Data Requirements

Data required for the quantitative analysis includes channel geometric data, hydrologic data, core samples, and descriptions of channel roughness. Channel geometric data were obtained by surveying several sections within the reach of interest and averaging them. The composite section is shown in Figure C-24. Analysis of local hydrology produced the flow duration curve shown in Figure C-26. Typical flow rates range from 0.15 m^3/sec to 15 m^3/sec with flows of less than 2 m^3/sec 95 percent of the time. The design discharge computed for this reach of Mosquito Creek was 200 m^3/sec. Core samples were taken at several locations within this reach. The two samples shown in Figure C-27 are representative of the variation of materials through the reach. Sample A represents the vertical distribution of sediments in the vicinity of the downstream bridge, and Sample B represents the vertical profile upstream of the bridge. Channel roughness is based on Manning's roughness coefficient, which is estimated at 0.034 for the reach.

Vertical Stability Considerations

The first step in the stability analysis is to determine the critical shear stress of the six characteristic layers in the core samples. Each stratum is tested in a rotating cylinder apparatus by the method described by Sargunam et al. (1973). The resulting critical shear stresses are listed in Table C-12.

The second step is to calculate the actual shear stress produced by typical flows within the reach. Actual shear stress is computed with the following equation:

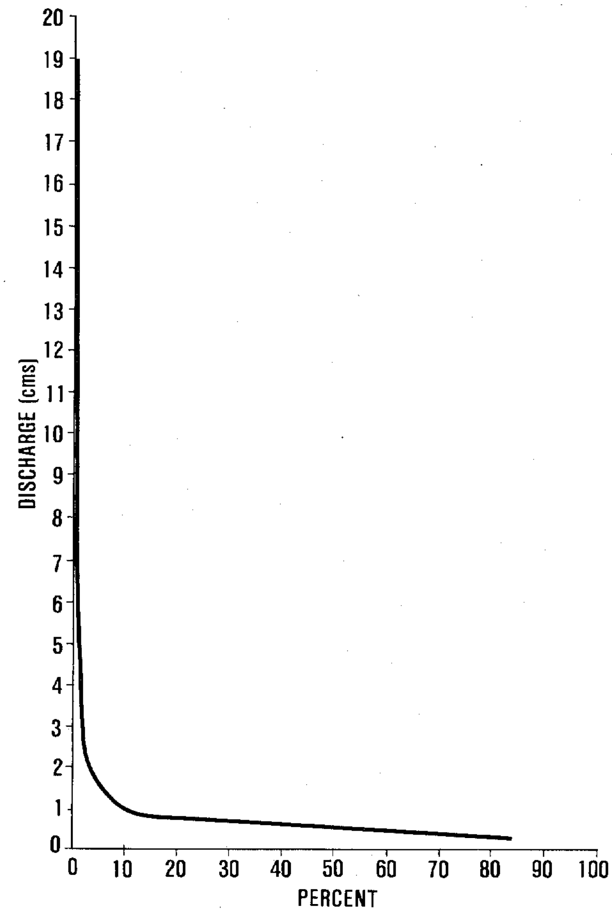


Figure C-26. FLOW DURATION CURVE, MOSQUITO CREEK BETWEEN PANAMA AND EARLING

$$\tau = \gamma R S .$$

The hydraulic radius (R) was evaluated for various flow conditions using Manning's flow resistance equation [Equation (4)]. The curves shown in Figure C-29 give the relationship between discharge and actual shear stress for various channel slopes. These curves are based on the geometry of the composite channel section in Figure C-24.

The third step is to compare the actual shear stresses from Figure C-28 with the critical shear stresses for the various levels in the core samples. The present reach slope is 0.0035. For the typical flows of 0.15 to 15 m^3/sec (those in the channel 99 percent of the time) bed shear stresses are 0.25 kg/m^2 to 5.37 kg/m^2. At the time of this investigation, the down-

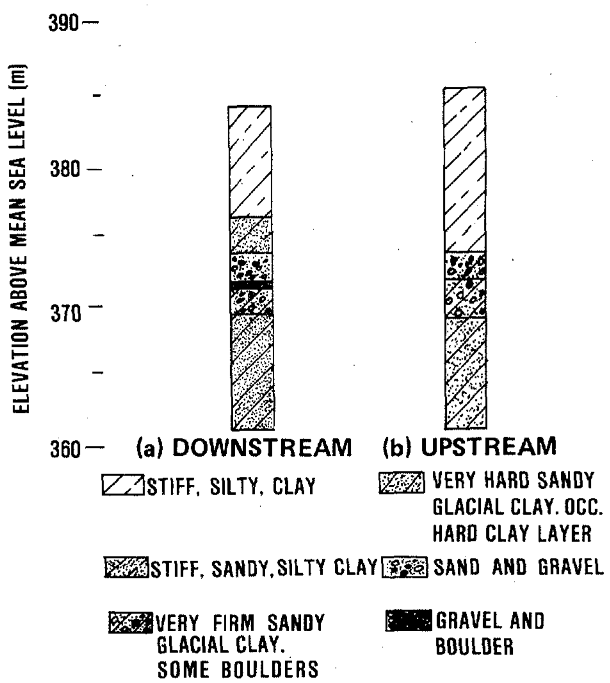


Figure C-27. TYPICAL CORE SAMPLES

Table C-12. CRITICAL SHEAR STRESS FOR VARIOUS CLAY STRATA

CORE STRATUM	CRITICAL SHEAR STRESS (kg/m^2)
Stiff, Silty Clay	2.34
Stiff, Sandy, Silty Clay	2.00
Very Firm, Sandy, Glacial Clay - Occasional Boulders	7.12
Very Firm, Sandy, Glacial Clay - Occasional Hard Clay	9.77
Sand and Gravel (D_m ~7 mm)	0.54
Gravel and Boulder (D_m ~46 mm)	3.56

stream end of the reach was in the stiff, sandy, silty, clay stratum [streambed elevation at the downstream bridge is at about 375.5 m (MSL)], while the upstream end of the reach was in stiff, sandy, clay [streambed elevation at the upstream bridge is approximately 377.6 m (MSL)]. Critical shear stresses for these layers are 2.34 kg/m^2 and 2.00 kg/m^2, respectively. Based on these data, the channel bed in this reach will be susceptable to degradation at flows above 2.8 m^2/sec. In a typical year the flow rate in Mosquito Creek exceeds 2.8 m^2/sec for 7 days. Since average annual peak flows often exceed 25 m^2/sec for several hours, most of the degradation in this reach will occur during a short period of time. However, the analysis does conclude that degradation will continue.

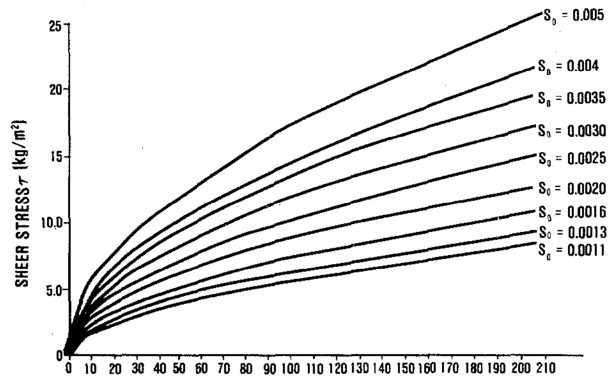


Figure C-28. FLOW RATE AS A FUNCTION OF SHEAR STRESS FOR VARIOUS CHANNEL SLOPES, S_o

To estimate the extent of degradation, Figures C-27 and C-28 and Table C-12 must be considered together. Remembering the qualitative discussion given to describe how the reach will degrade and combining this with the quantitative analysis of bed stability, the extent of degradation can be estimated. The downstream end of the reach is in a slightly less resistant bed material than the upstream end. Therefore, it will degrade slightly faster than the upstream reach; this will increase the local slope, which will in turn increase the bed shear stress. This will be a relatively slow process, taking several years depending on flow conditions. However, when the downstream end is lowered to the sand and gravel stratum (critical shear stress of 0.54 kg/m^2) degradation will be very rapid. The result will be a drastic increase in slope since the upstream end of the reach will still be in the stiff, sandy, clay stratum where degradation is much slower. The downstream end has now degraded to the gravel/boulder layer and becomes a bit more stable (critical shear of 3.56 kg/m^2) than the upstream end which will still be in the stiff, sandy clay layer. Degradation will continue in the upstream portion of the reach until the very firm, sandy, clay layer (critical shear stress of 7.81 kg/m^2) is reached or the slope is reduced to some stable value. The reach slope would be 0.0035 which would still be unstable. The slope would then reduce rapidly as the upstream end degrades through the sand and gravel layer. At this point both ends of the reach would be in the very firm, sandy, glacial clay and the reach slope would be extremely low, probably somewhere around 0.001. At this low slope, the critical shear stress of the very firm, sandy, glacial clay layer (7.12 - 9.76 kg/m^2) would be sufficient to resist erosion except under the most severe flow events (those above 85 m^3/sec).

The most probable depth of degradation for this reach based on the analysis above is down to the very firm, sandy, glacial clay layer at about 371 m (MSL). The total expected degradation depth below the 1940 channel bed elevation is approximately 8-10 m.

Lateral Stability

Lateral stability problems are caused by bank slumping and channel meandering. Based on the extent of degradation expected, bank slumping will be extensive throughout the entire reach.

The lateral extent of bank slumping is analyzed by considering the stable bank angle and the expected degradation depth. Bank angle measurements at several locations within the study reach revealed that the average stable bank angle is 1:0.67 (vertical to horizontal). Assuming that the base width of the channel remains the same as that given in Figure C-24, Figure C-29 documents the maximum channel width based on 5.5 m of additional degradation. The topwidth of the channel is assumed to increase from 15 m to 24 m or 4.5 m on each side of the channel.

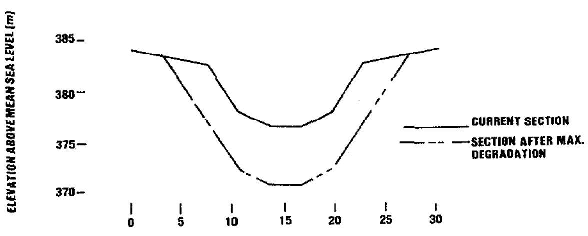


Figure C-29. ASSUMED CHANNEL GEOMETRY BEFORE AND AFTER MAXIMUM DEGRADATION

Channel meandering can also cause lateral stability problems. However, because the channel is so deeply incised in this reach, natural meandering will be minimal.

PHASE III: COUNTERMEASURES

The anticipated 5.5 m of additional degradation and 4.5 m of lateral bank slumping would both undermine and outflank the piers of both the upstream and downstream bridges (Figure C-24). Possible countermeasures would include putting down new pier foundations, heavily bulkheading the piers, constructing a grade control structure downstream of both structures, or replacing both bridges with structures that will span the 24 m topwidth without using piers. Of these alternatives, the construction of a grade control structure is the most economical choice. Grade control structures have also been successful in other similar cases.

Because the two bridges are only 0.5 km apart, one grade control structure placed downstream would protect both crossings. Figure C-30 shows the location of the grade control structure 150 m downstream of the highway crossings. The crest elevation was set at 379.8 m (MSL) with an 2.4 m wide flow section lowered 0.3 m to 379.5 m (MSL). Assuming the original channel slope within this reach to be the stable slope (0.0018), the grade control structure will cause the deposition of suspended material to an elevation of 379.8 m at the downstream crossing and 380.9 m at the upstream crossing. Comparing these elevations to the original channel bed elevations in Figure C-23, it is observed that this scheme will stablize the reach at near its original level. This of course assumes that there is sufficient sediment yield from the watershed to fill the channel behind the grade control structure. An analysis of upland sediment yields based on the universal soil loss equation reveals that for this area, the average annual soil loss is about 4.77 million tonnes per year. This would supply sufficient amounts of material to fill the area

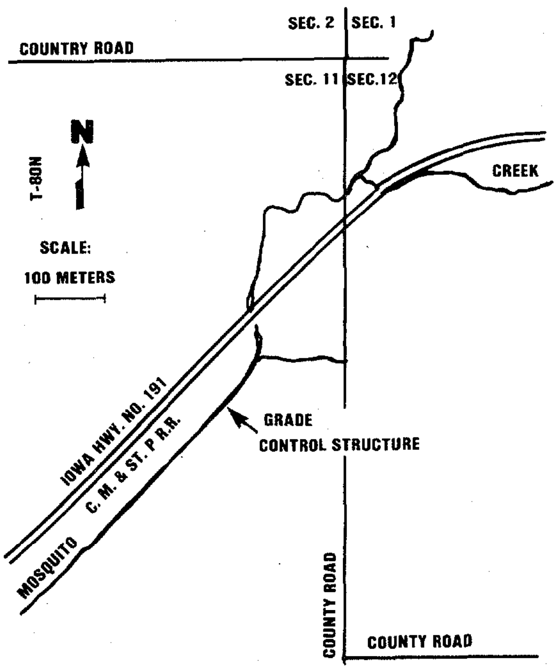


Figure C-30. LOCATION OF GRADE CONTROL STRUCTURE ON MOSQUITO CREEK

behind the grade control structure. This scheme will also protect the county road bridge and the two railroad bridges within this reach.

Figure C-31 shows design sketches for the grade control structure. Note that the sheet pile check dam extends 11 m on either side of the channel banks. This provides a factor of safety of 2 with respect to lateral erosion. The sheet pile is to be driven to a depth of 15 m, which puts its base well into the very firm, sandy, glacial clay at about elevation 367.5 m (MSL). The downstream slope of the check dam and the stilling basins at its base are lined with derrick stones that range in weight from 2500 to 2950 kg. This material was designed to resist and dissipate the energy produced in the 4.6 m drop downstream of the structure during the 200 m^2/sec design flood.

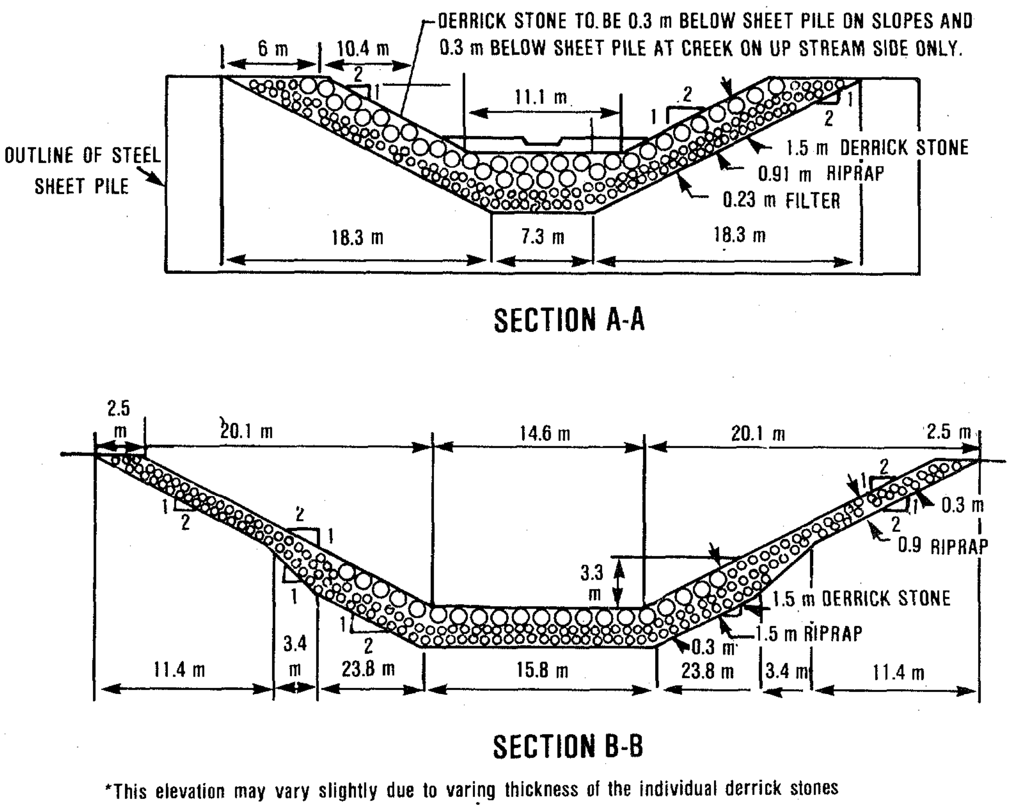
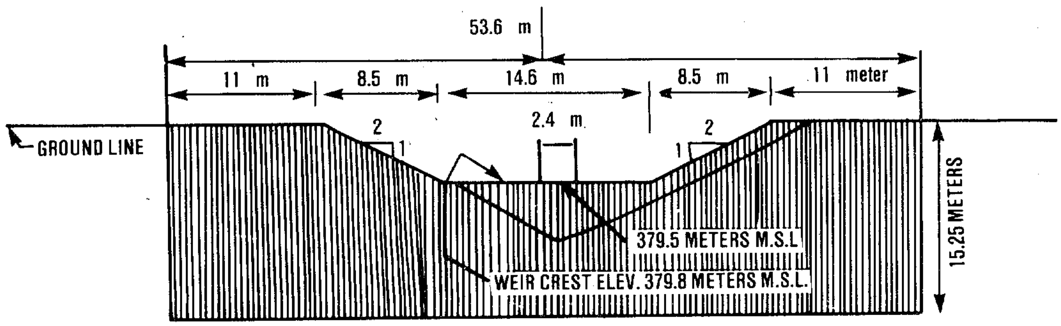
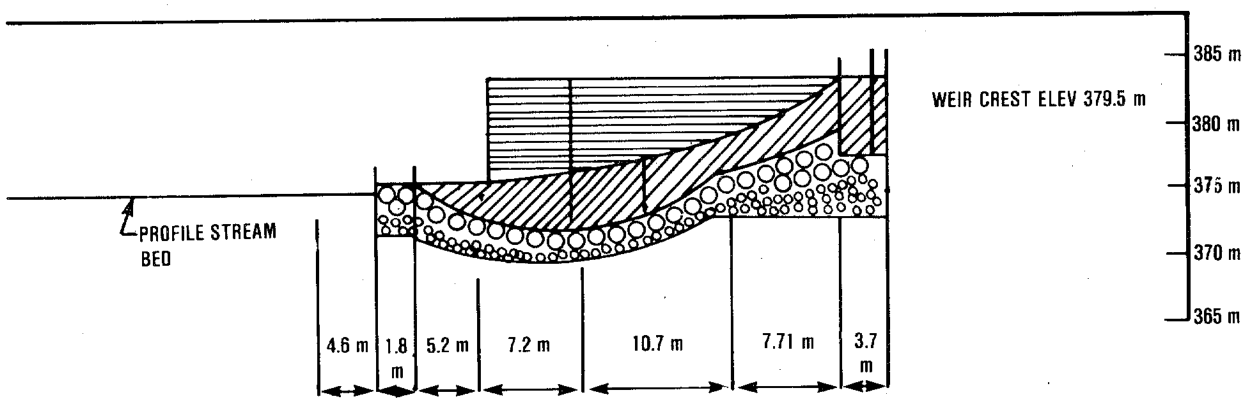


Figure C-31. DESIGN SKETCHES FOR GRADE CONTROL STRUCTURE ON MOSQUITO CREEK



SECTION ALONG STEEL SHEET PILE WALL



LONGITUDINAL SECTION ALONG STREAM

Figure C-31. DESIGN SKETCHES FOR GRADE CONTROL STRUCTURE ON MOSQUITO CREEK (Continued)

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FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH AND DEVELOPMENT

The Offices of Research and Development (R&D) of the Federal Highway Administration (FHWA) are responsible for a broad program of staff and contract research and development and a Federal-aid program, conducted by or through the State highway transportation agencies, that includes the Highway Planning and Research (HP&R) program and the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board. The FCP is a carefully selected group of projects that uses research and development resources to obtain timely solutions to urgent national highway engineering problems.*

The diagonal double stripe on the cover of this report represents a highway and is color-coded to identify the FCP category that the report falls under. A red stripe is used for category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, green for categories 6 and 7, and an orange stripe identifies category 0.

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion, and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by balancing the demand-capacity relationship through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements that affect the quality of the human environment. The goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

4. Improved Materials Utilization and Durability

Materials R&D is concerned with expanding the knowledge and technology of materials properties, using available natural materials, improving structural foundation materials, recycling highway materials, converting industrial wastes into useful highway products, developing extender or substitute materials for those in short supply, and developing more rapid and reliable testing procedures. The goals are lower highway construction costs and extended maintenance-free operation.

5. Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

Structural R&D is concerned with furthering the latest technological advances in structural and hydraulic designs, fabrication processes, and construction techniques to provide safe, efficient highways at reasonable costs.

6. Improved Technology for Highway Construction

This category is concerned with the research, development, and implementation of highway construction technology to increase productivity, reduce energy consumption, conserve dwindling resources, and reduce costs while improving the quality and methods of construction.

7. Improved Technology for Highway Maintenance

This category addresses problems in preserving the Nation's highways and includes activities in physical maintenance, traffic services, management, and equipment. The goal is to maximize operational efficiency and safety to the traveling public while conserving resources.

0. Other New Studies

This category, not included in the seven-volume official statement of the FCP, is concerned with HP&R and NCHRP studies not specifically related to FCP projects. These studies involve R&D support of other FHWA program office research.

* The complete seven-volume official statement of the FCP is available from the National Technical Information Service, Springfield, Va. 22161. Single copies of the introductory volume are available without charge from Program Analysis (HRD-3), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.