Short Course on
Engineering Analysis of Fluvial Systems

July 30 - 31, 1986

**ARIZONA
TRANSPORTATION
RESEARCH
CENTER**

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Region IX

Phoenix, Arizona

Reach 2 Typical section

Depth	Discharge	Topwidth	Area	
0.00	0.00	0.00	0.00	
1.00	106.00	150.00	75.00	
2.00	654.00	190.00	245.00	
3.00	1760.00	240.00	460.00	
4.00	3420.00	245.00	702.50	
5.00	5530.00	255.00	952.50	8.5
6.00	8040.00	270.00	1215.00	
7.00	10960.00	277.00	1488.50	
8.00	14270.00	285.00	1769.50	
9.00	17960.00	392.00	2108.00	
10.00	22040.00	300.00	2454.00	
11.00	26500.00	310.00	2759.00	9.6
12.00	31340.00	320.00	3074.00	
13.00	36570.00	330.00	3399.00	
14.00	42180.00	340.00	3734.00	
15.00	48180.00	350.00	4079.00	
16.00	54577.00	360.00	4434.00	
17.00	61370.00	370.00	4799.00	
18.00	68560.00	380.00	5174.00	

Reach 1 Typical Section

Depth	Discharge	Topwidth	Area
0.00	0.00	0.00	0.00
1.00	826.00	440.00	220.00
2.00	2710.00	500.00	690.00
3.00	5420.00	570.00	1225.00
4.00	9140.00	640.00	1830.00
5.00	14280.00	720.00	2510.00
6.00	20870.00	725.00	3232.50
7.00	28440.00	730.00	3960.00
8.00	36150.00	765.00	4707.50
9.00	44770.00	800.00	5490.00
10.00	54300.00	840.00	6310.00
11.00	64770.00	880.00	7170.00
12.00	77840.00	890.00	8055.00
13.00	91890.00	900.00	8950.00

ARIZONA TRANSPORTATION RESEARCH CENTER

SHORT COURSE

ENGINEERING ANALYSIS OF FLUVIAL SYSTEMS

JULY 30-31, 1986

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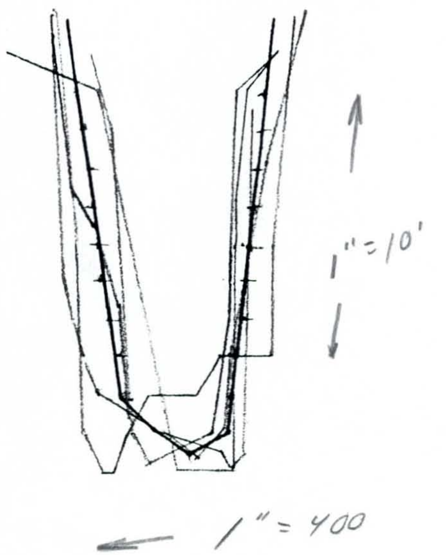
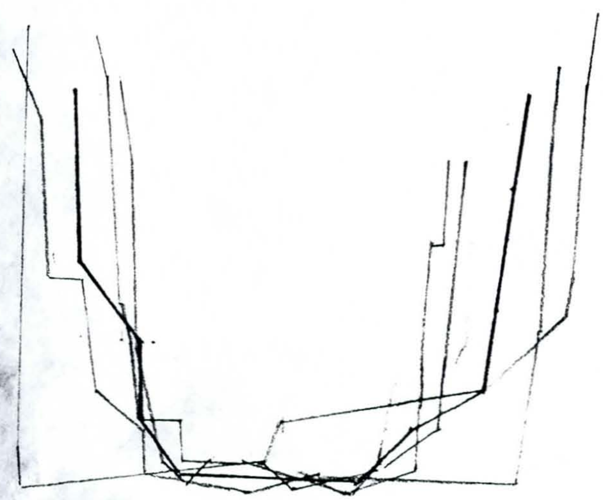
Reach 2



Dist	Elev.
1000	42
1120	22
1260	19.5
1395	20.5
1430	42.0

Reach 1



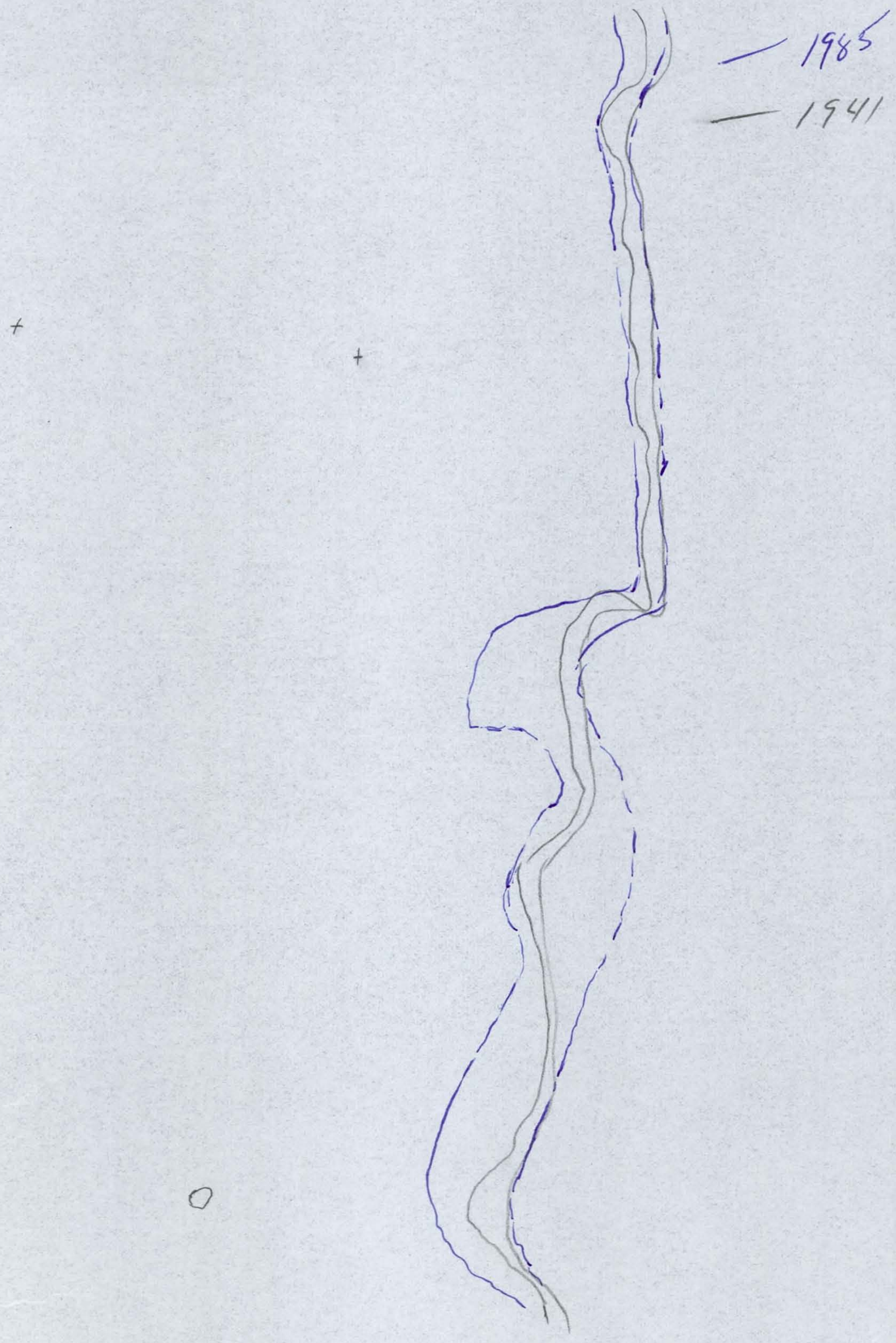
Dist	Elev.
1000	50
1010	41
1140	37
1150	33
1230	30
1620	30
1710	32.5
1865	34.5
1950	50.0

Average Channel Shapes Santa Cruz River

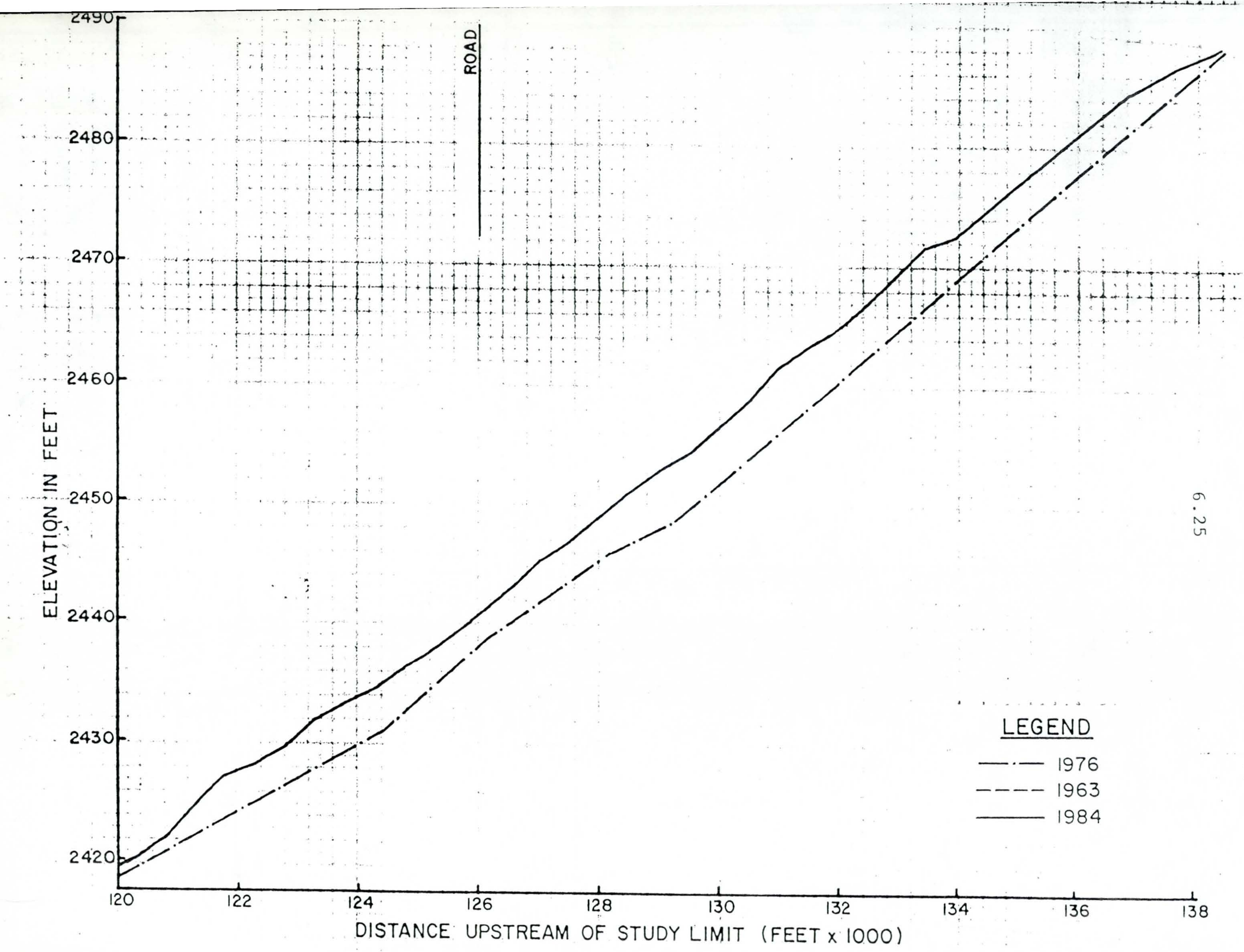
1985 AERIAL PHOTOGRAPH OF AN APPROXIMATELY
THREE-MILE-LONG REACH OF THE SANTA CRUZ RIVER
NEAR TUCSON, ARIZONA
Scale: (3" = 1 Mile)

1941 AERIAL PHOTOGRAPH OF AN APPROXIMATELY THREE-MILE-LONG REACH OF THE SANTA CRUZ RIVER NEAR TUCSON, ARIZONA

Scale: (3" ≐ 1 Mile)



1985
1941



- Historical Channel Bed Profiles

ARIZONA DEPARTMENT OF TRANSPORTATION
ARIZONA TRANSPORTATION RESEARCH CENTER

ENGINEERING ANALYSIS OF FLUVIAL SYSTEMS

A Short Course

JULY 1986

LECTURE NOTES

ENGINEERING ANALYSIS OF FLUVIAL SYSTEMS

July 30-31, 1986

Presented By

Simons, Li & Associates, Inc.

For

Arizona Transportation Research Center

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Dr. Li is recognized as a leader in the fields of mathematical modeling of watersheds and river systems. His principal fields of expertise are hydraulics, hydrology, watershed management, erosion and sedimentation, river mechanics, systems engineering, water resources development, stochastic processes, and water quality studies.

Dr. Robert C. MacArthur, P.E. - Vice-President, Simons, Li & Associates, Inc.
Dr. MacArthur has national and international experience in applied research, teaching, and engineering analyses dealing with problems in river mechanics, estuarial hydrodynamics, sedimentation engineering, water supply, environmental engineering, land use methods, and modeling of hydrological and other engineering systems. He has developed analytical methods for evaluating problems in hydrology, hydraulics and sedimentation engineering and conducted specialized training courses for the Corps of Engineers Hydrologic Engineering Center (HEC).

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During an eight year tenure with the Pima County Flood Control District, Mr. Zeller was actively involved in the development and implementation of drainage and flood control standards for Pima County. While with Simons, Li & Associates, he has been project manager for over 250 projects relating to hydrologic, hydraulic, erosion, and sediment transport problems within Arizona.

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Mr. Cotton has authored a number of design manuals implementing sedimentation technology. He has recently co-authored a revised version of the Federal Highway Administration design manual, HEC No. 15, "Drainage of Roadside Channels With Flexible Linings". This manual uses the tractive force approach method to design stable channel linings for small to moderate sized channels that are part of the highway drainage network. Other manuals that Mr. Cotton has co-authored include "Design of Sediment Control Measures for Small Areas in Surface Coal Mining" for the Office of Surface Mining and "Erosion Control Manual" for the Colorado Department of Highways.

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TABLE OF CONTENTS

SCHEDULE OF LECTURES

	Lecture	Topic
July 30		
8:00		Welcome
8:15	1	Multiple Level Analysis Approach
8:45	2	Hydrologic and Hydraulic Analysis
9:45		Break
10:00	3	Level I - Qualitative Geomorphic Analysis
11:00	4	Workshop 1, Qualitative Analysis
12:00		Lunch
1:00	5	Level II - Sediment Transport Equations
2:15	6	Overview of Sediment Transport Phenomenon Based on Arizona Case Histories
2:45		Break
3:00	7	Level II - Bed Adjustment Mechanisms
July 31		
8:00	7	Level II - Bed Adjustment Mechanisms (continued)
9:45		Break
10:00	7	Level II - Bed Adjustment Mechanisms (continued)
11:00	8	Workshop 2, Quantitative Analysis
1:00	9	Design Criteria and Agua Fria Case Study
2:45		Break
3:00	10	Case History, Oro Valley
4:00		Open Forum and Closing Comments
4:30		Course Evaluation

MULTI-LEVEL ANALYSIS APPROACH

LECTURE 1

MICHAEL E. ZELLER, P.E.

reference manual pgs 2.5 - 2.13

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July 1986

A Effecting fluvial system

1) hydrologic
2) hydraulic
3) geologic
4) soil
5) climatologic
6) biologic
7) man's influence

B) Long term impacts must be addressed

a) Life cycle costs

1) sediment routing as well as flood routing

LECTURE 1: Multi-Level Analysis Approach

OBJECTIVE: This lecture is intended to provide an overview of the technique of multi-level analysis for the solution of fluvial-system problems. The individual steps of such an analysis will be presented, with the emphasis of the lecture focused upon the first two components: Qualitative Geomorphic Analysis (Fluvial Geomorphology) and Quantitative Geomorphic Analysis (Basic Engineering). The third component of this multi-level approach, Mathematical Modeling, will be discussed briefly at the end of the lecture.

I. GENERAL INTRODUCTION

The analysis of, and solution to, problems involving complex fluvial systems generally are best approached through a step-by-step process which defines each component of the problem and finds a solution to same. The solution approach presented here is composed of three general levels of analysis which utilize analytical tools from the disciplines of geomorphology, hydrology, and hydraulics. The solution techniques employ both qualitative and quantitative methods, thereby providing the engineer with a means to develop a general overview of the fluvial system, as well as addressing specific items of concern. Although each step in such a process is an independent one, the final solution to a complex problem sometimes requires an iterative process which uses this multi-level approach as a check-and-balance procedure to assure that convergence to the "true" solution will in fact occur.

However, it is not always necessary to complete all three levels of a multi-level analysis approach in order to obtain meaningful results. Depending upon the data and the resources available, the solution to fluvial-system response can often be obtained through the application of individual components of the multi-level analysis approach. These components have been labeled as "levels" of analysis, and are described as follows:

Level I - A qualitative analysis, based upon general geomorphic concepts

Level II - A quantitative engineering analysis, based upon fundamental hydrologic and hydraulic relationships

Level III - A detailed quantitative analysis, utilizing mathematical modeling to describe complex watershed and river processes

While each ascending level of analysis requires an increased commitment of resources, it also produces meaningful results in its own right. Therefore, when applied as an itegrated approach, this multilevel approach constitutes a powerful methodology for the solution to complex fluvial-system problems, especially when evaluating short-term and/or long-term responses of watershed and river systems.

fluvial-geomorphic
effects - long term

II. LEVEL I - QUALITATIVE GEOMORPHIC ANALYSIS

A. Use of General Relationships

- Well-Known Relationships (See Figures)
  - Lane
  - Leopold and Wolman
  - Henderson
  - Schumm

B. Use of Classification Systems

- Straight
- Meandering
- Braided
- Combination

C. Use of Aerial Photography

D. Interpret Changes in Land Use

E. Interpret Development Activities of Man

F. Assessment of Changes in Plan and Profile

III. LEVEL II - QUANTITATIVE GEOMORPHIC AND BASIC ENGINEERING ANALYSIS

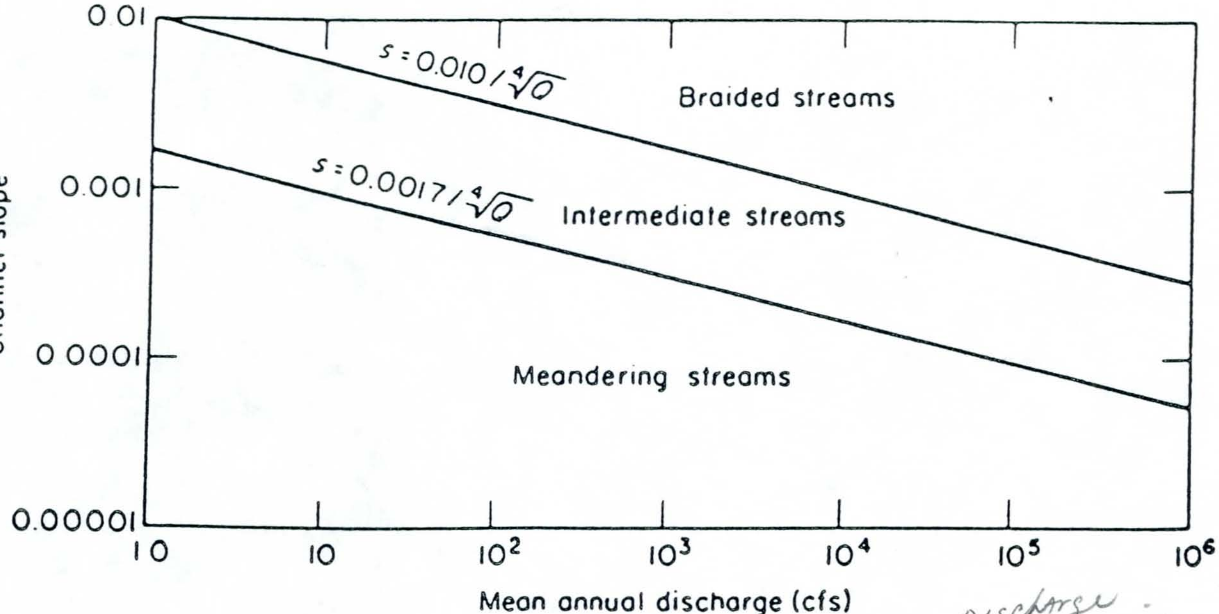
rigid boundary
basic sediment balancing
in downstream

A. Hydrologic Analysis

- Changes in Hydrologic Processes
- Changes in Flow Regime
- Changes in Sediment Yield and Transport Rate
- Design Discharge
- Dominant Discharge

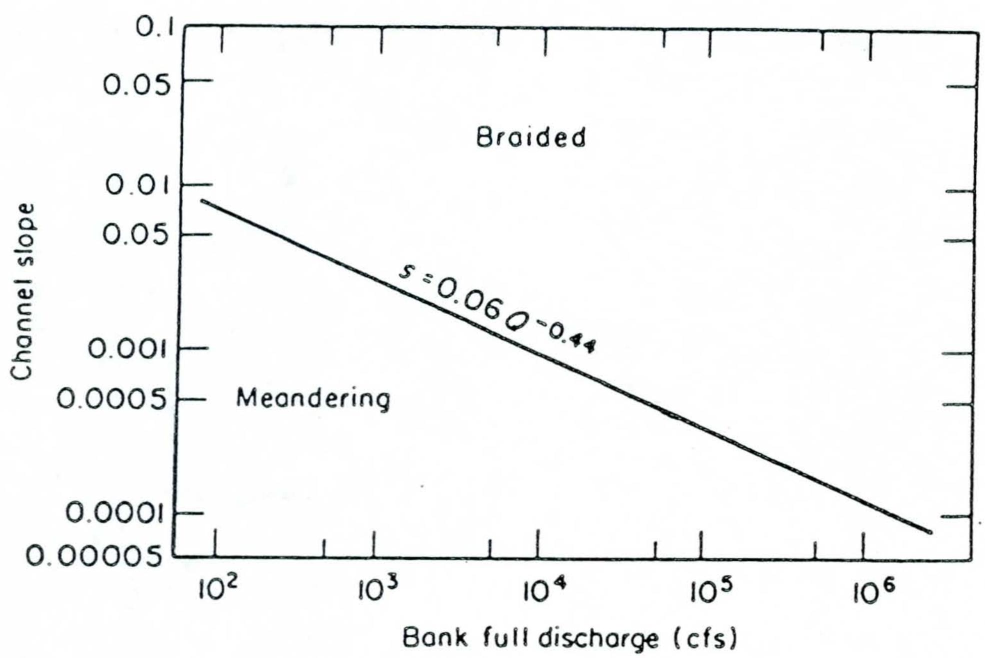
B. Hydraulic Analysis

- Water-Surface Profiles



DOMINANT DISCHARGE used to design (bank full or 10yr flood whichever is less)

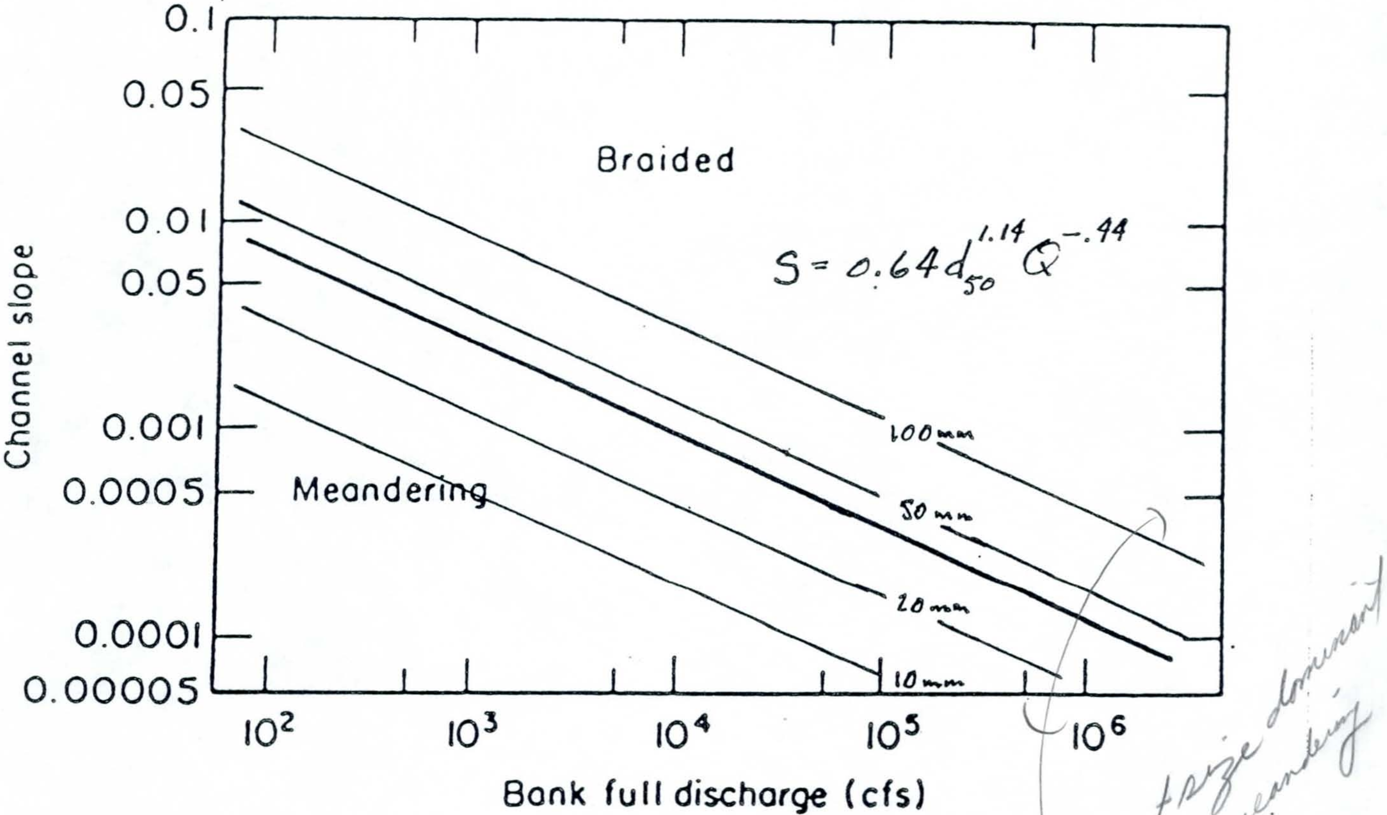
Lane's discharge relation



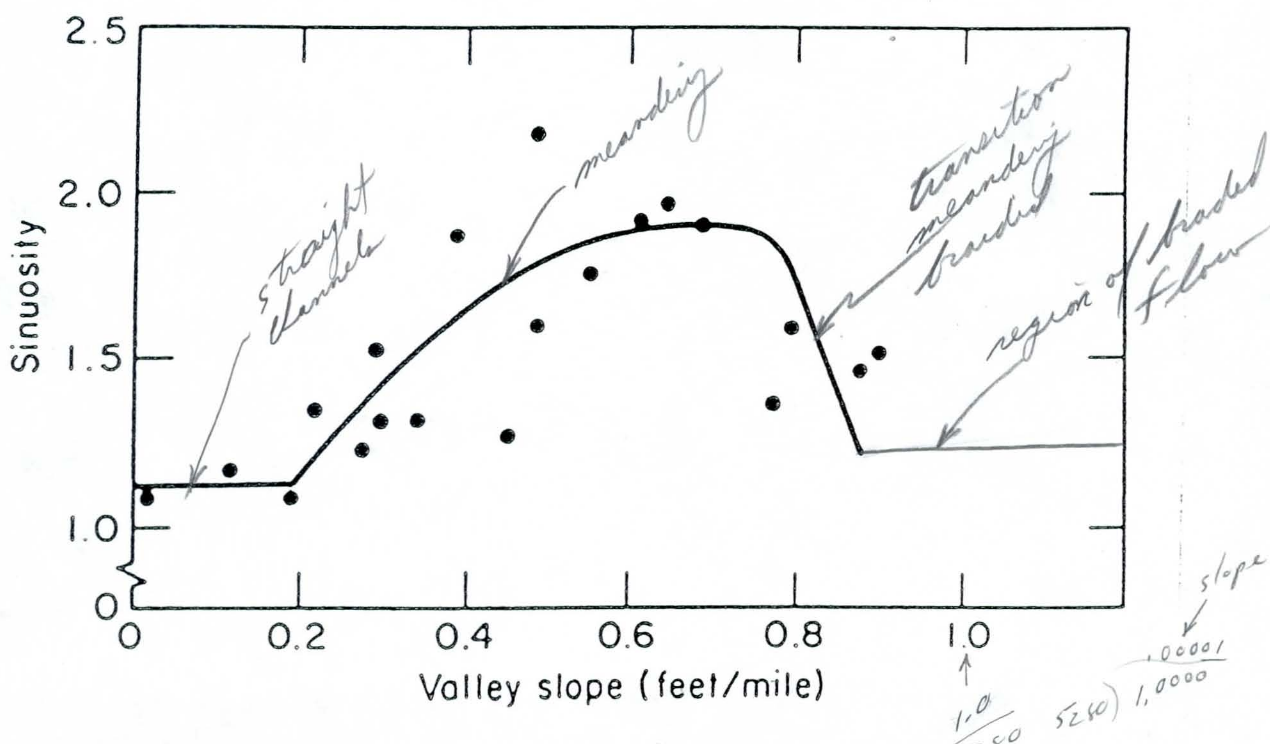
Leopold and Wolman's discharge relation

Studies not in range of sand size

1) estimate D_{50} for ball park usage



Henderson's discharge relation



Schumm's sinuosity relation

Mississippi River

- Flow Distribution

- Depth

- Velocity

° Distribution of Bed and Bank Sediments

gradation coefficient
$G = \frac{1}{2}\left(\frac{D_{50}}{D_{16}} + \frac{D_{84}}{D_{50}}\right)$

- Sieve Analyses (See Figures)

- PI and UCS Tests, if required

- Visual Analysis (Pebble Count, Grid Photography)

Sand bed in S.W.
coefficient = 3+4

° Estimate Sediment-Transport Rate

- From Actual Measurements

- From Transport Equations(s)

° Compute Equilibrium Slope

- Using Simplified Transport Relationships

- Applying Principle of Sediment Continuity

° Predict Aggredation/Degradation Trends of Fluvial System

° Predict Lateral Migration/Meandering Trends of Fluvial Systems

limited to singular event floods!!

IV. LEVEL III - MATHEMATICAL MODELING

A. Definition

° Quantitative Means of Describing the Complex Interaction of Physical Processes Related to Hydrology, Hydraulics, Erosion and Sedimentation Which Occurs Within a Fluvial System

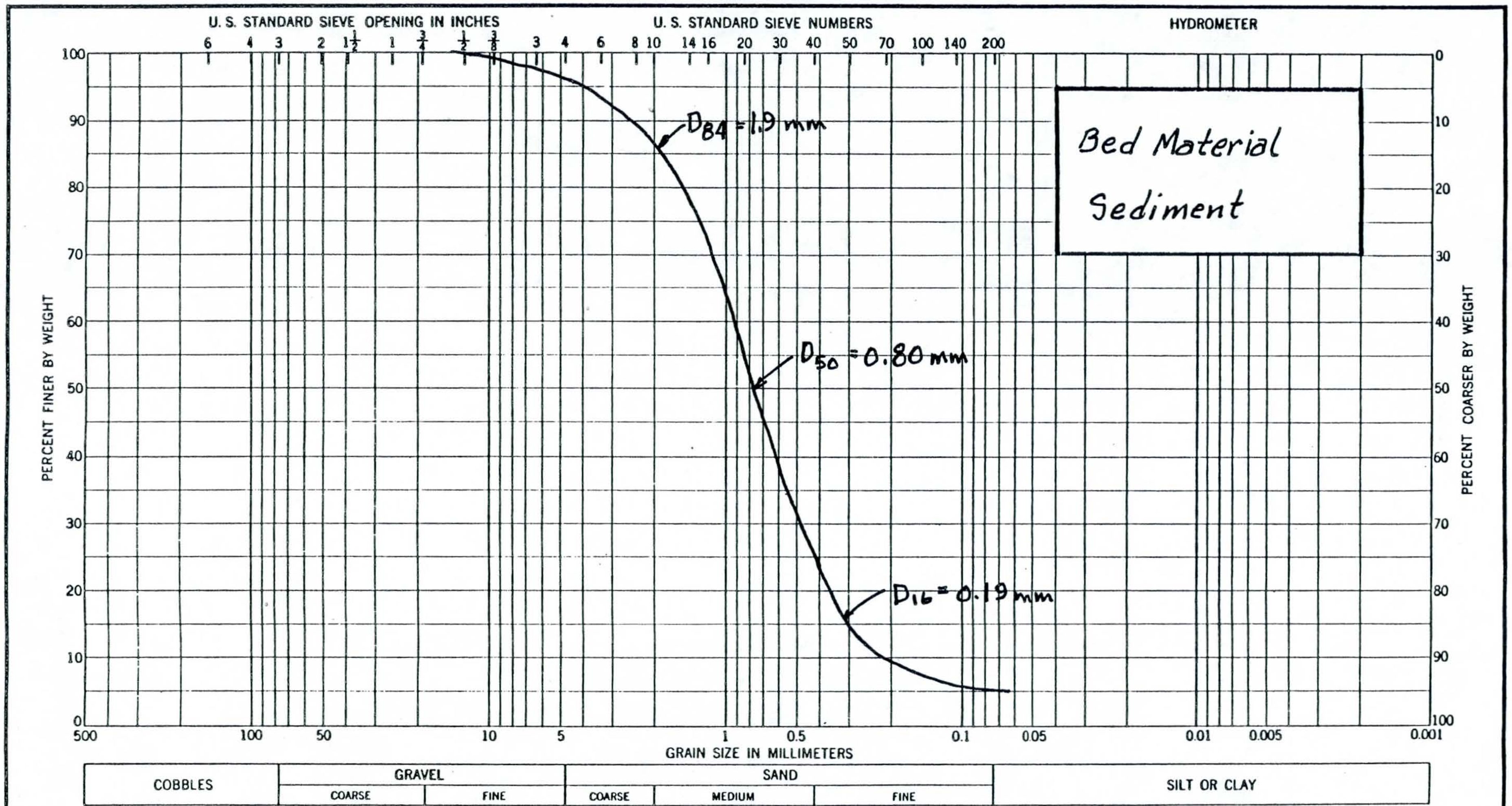
° A convenient Means of Rapidly Executing Complex Numerical Procedures (i.e., a "Number Cruncher")

B. Benefits

° Can Play "What-If" Games

° Economical to Run Multiple Alternatives, Once Calibrated

° Can Evaluate Uncertainties of Assumptions and Decisions



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI

Project

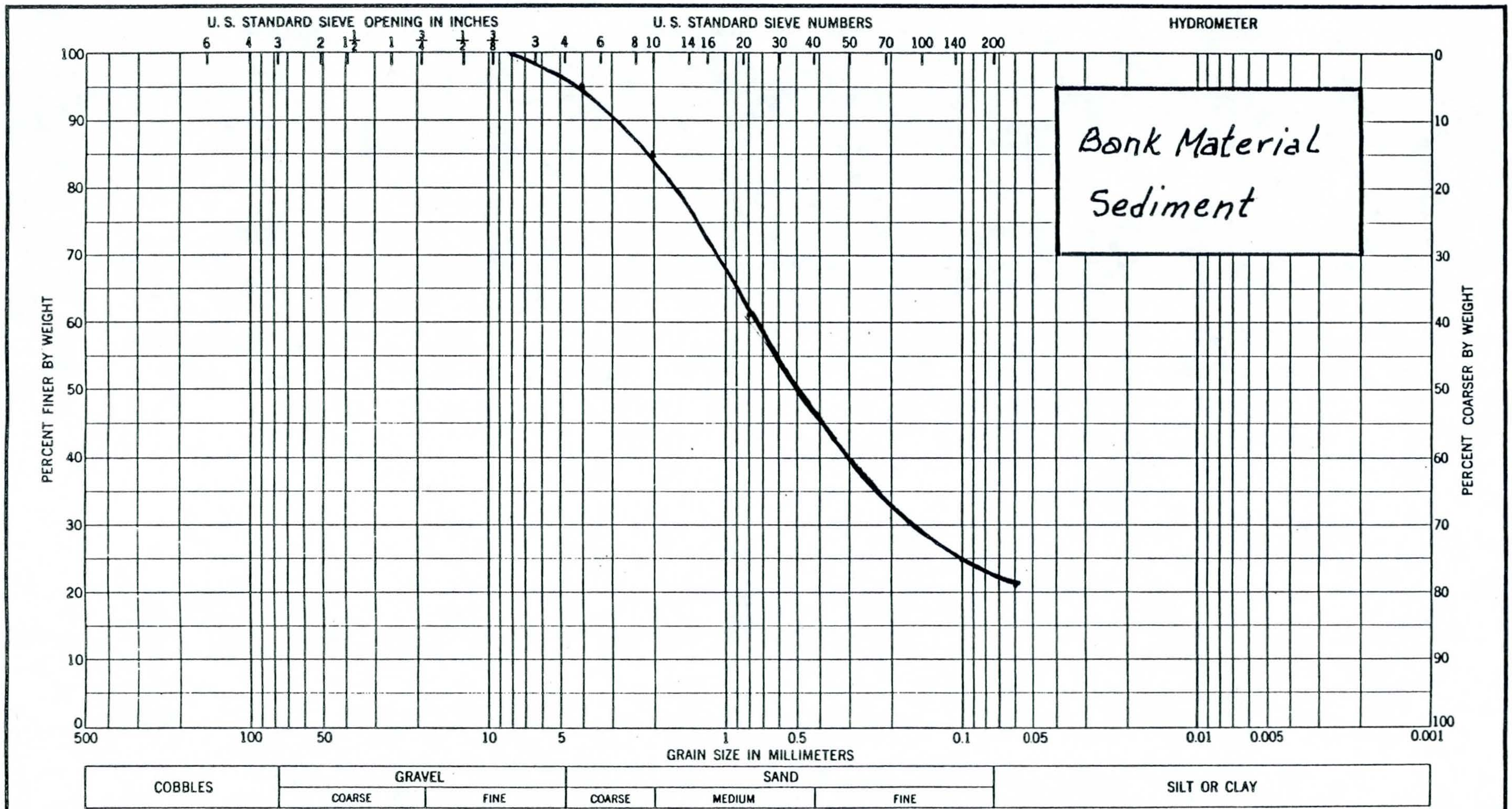
Area

Boring No.

Date

GRADATION CURVES

ENG FORM 1 MAY 63 2087



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI

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Date

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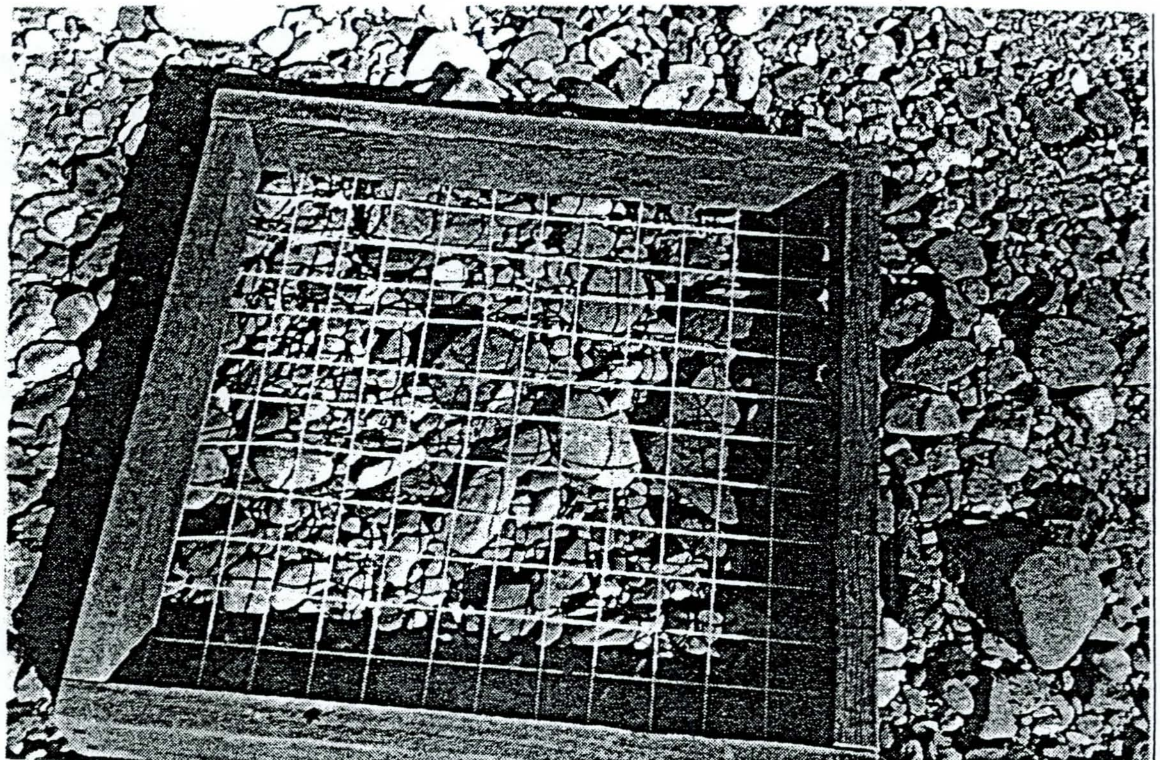
SLA, INC.



particle size sampling

C. Costs

- More Complex Models Require Added Data and Generate Increased Cost

- The Uninformed User Can Misapply Modeling and Unknowingly Produce Inaccurate Results

  - The Multi-Level Approach Described During This Lecture Enables the User to Make System-Modeling Decisions on a More-Informed Basis

comparisons must be made between all three levels for compatability & continuity

HYDROLOGIC AND HYDRAULIC ANALYSES

LECTURE 2

MICHAEL E. ZELLER, P.E.

covered in Design Manual
Sections = 3.1 - 3.21 hydrology
4.1 - 4.36 hydraulic!!
water & sediment interactions

Simons, Li & Associates, Inc.
120 West Broadway, Suite 120
P.O. Box 2712
Tucson, Arizona 85702

July 1986

impacts

- climate
  - ↳ slow - long term
- man
  - ↳ short term - fast

Dynamic equilibrium

1) slow enough where they are considered steady even though are changing

Thunderstorms

1) maximum size 100 sq mile
2) short duration high intensity

LECTURE 2: Hydrologic and Hydraulic Analyses

OBJECTIVE: This lecture is intended to present an overview of the various techniques available to the practicing engineer when undertaking hydrologic and hydraulic analyses of fluvial systems. Emphasis is placed on aspects of such analyses which are especially relevant within the desert southwest.

I. GENERAL INTRODUCTION

The semi-arid climate of the southwestern United States presents a challenge to any surface-water hydrologist and/or drainage design engineer attempting to analyze and correctly predict the behavior of an alluvial channel system. A large percentage of the difficulties encountered with such analyses are related to sediment-transport phenomena, and it is therefore essential that the interaction between water and sediment be considered when conducting hydrologic and hydraulic analyses in the desert southwest. One example is the variation of resistance to flow in alluvial channels which can occur as a result of the development of bed-forms. However, it is also essential that the sound application of fundamental hydrologic and hydraulic concepts serve as a prerequisite to the more-detailed examination of erosion and sedimentation processes likely to occur within alluvial channel systems.

II. HYDROLOGIC ANALYSES

A. Relation to Other Analyses

- "First Step"
- Rainfall
  - Spatial
  - Temporal

3.10 + 3.11

flood routing

rainfall interval ≠ runoff interval (??)

- Runoff
  - Peak
  - Volume

MAXIMUM A3
2-3K cfsm
based upon 20 sq mi
3 in/hr rainfall

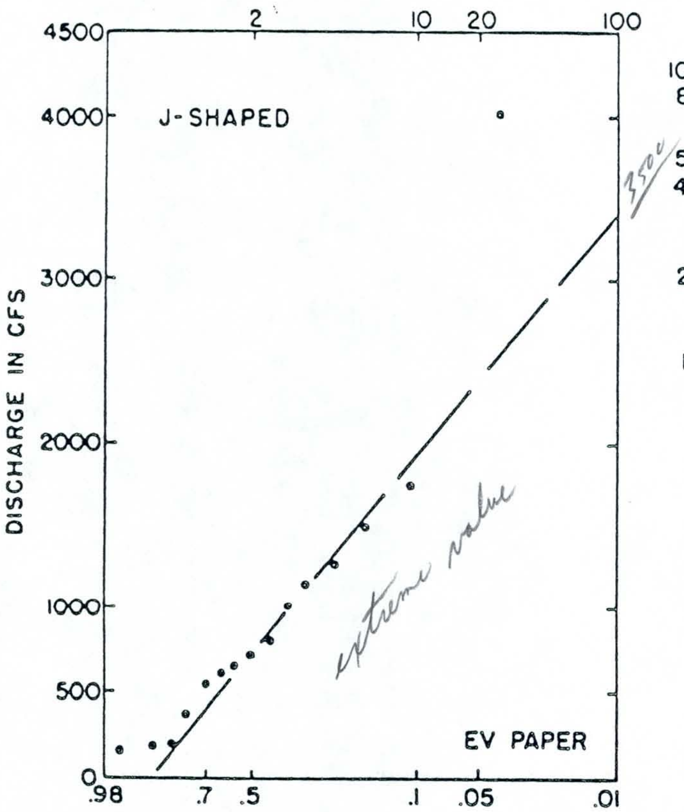
B. Sources of Data

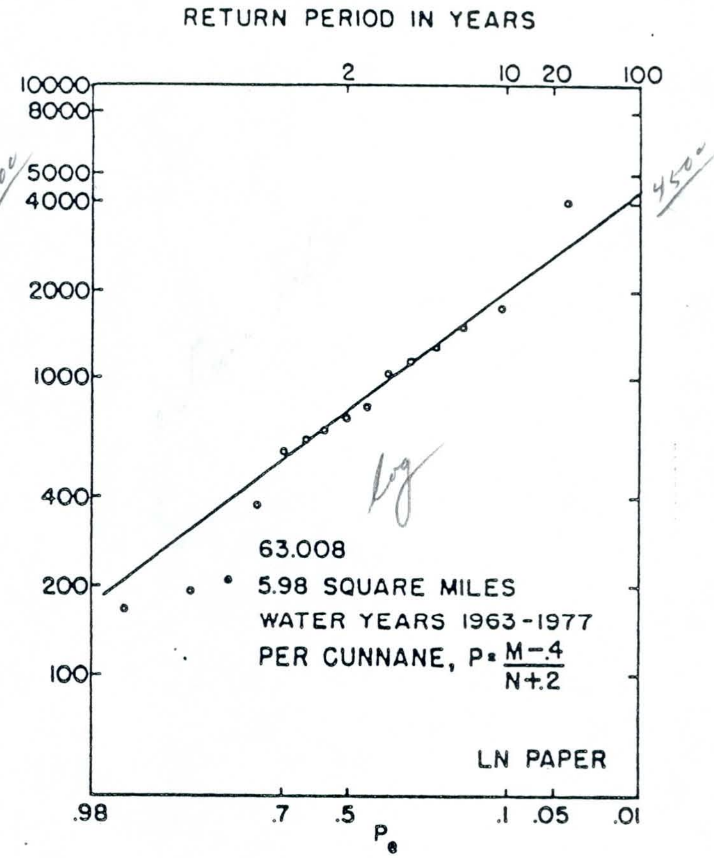
- United States Geological Survey
  - District Office of Water Resource Division (Tucson)
  - National Headquarters (Reston, Virginia)

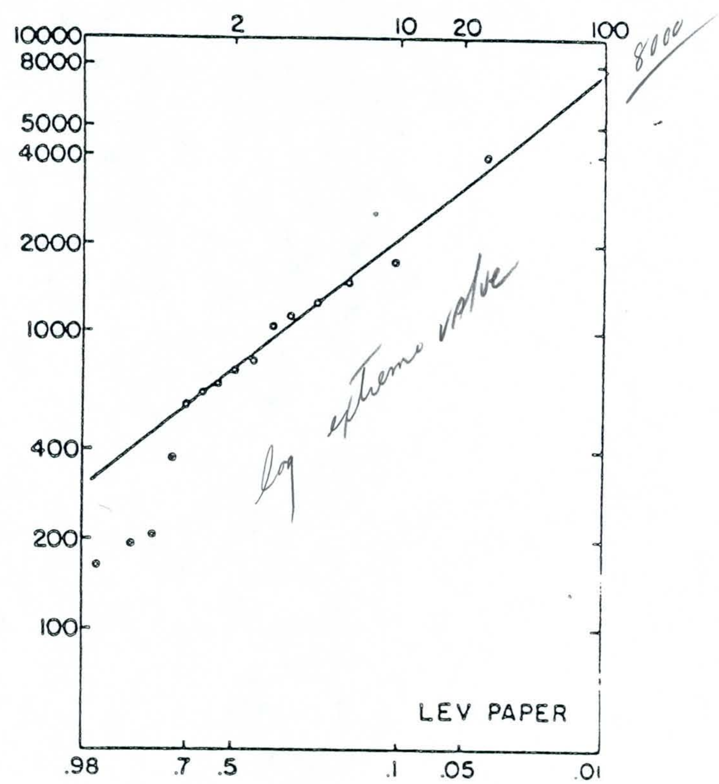
- United States Department of Agriculture
  - Soil Conservation Service
  - Agricultural Research Service (Southwest Watershed Rangeland Research Center)
- National Weather Service
- Corps of Engineers (L.A. District)
- Bureau of Reclamation (Denver, Colorado)
- United States Forest Service
- United States Bureau of Land Management
- Federal Emergency Management Agency
- State Department of Water Resources
- State Department of Transportation
- County Flood Control Districts
- County Highway Departments
- City Engineering Departments
- Irrigation Districts
- Special District (e.g., Salt River Project and (CAWCD)
- Public and Private Universities
- State Climatologist
- Private Individuals
- News papers

C. Peak-Discharge Determination

- At-Gage Statistical Analyses (See Figures)
  - Extreme Value, Log Normal, Log Extreme Value, Gumble
  - Log-Pearson Type III
- Regional Methods (See Figures)
  - United States Geological Survey Flood-Frequency Analysis

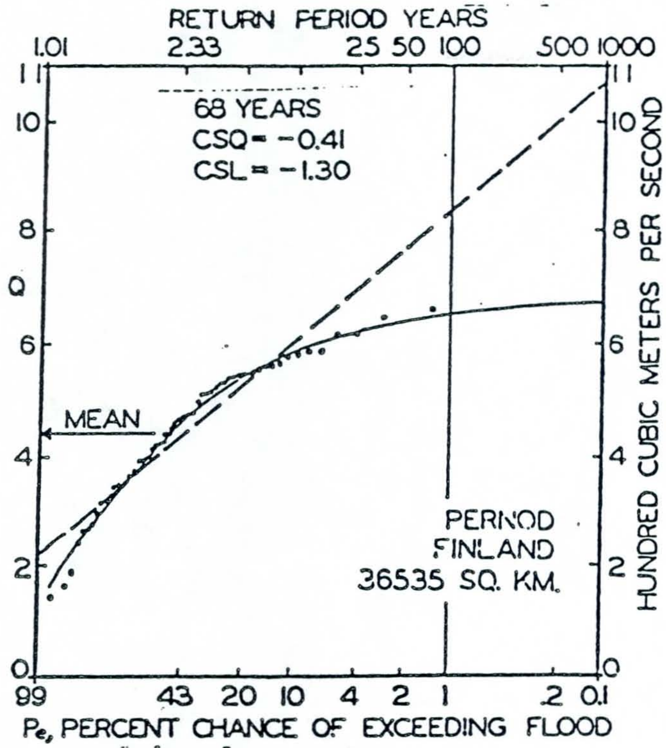






Comparison of probability papers

for calibration to mathematical models



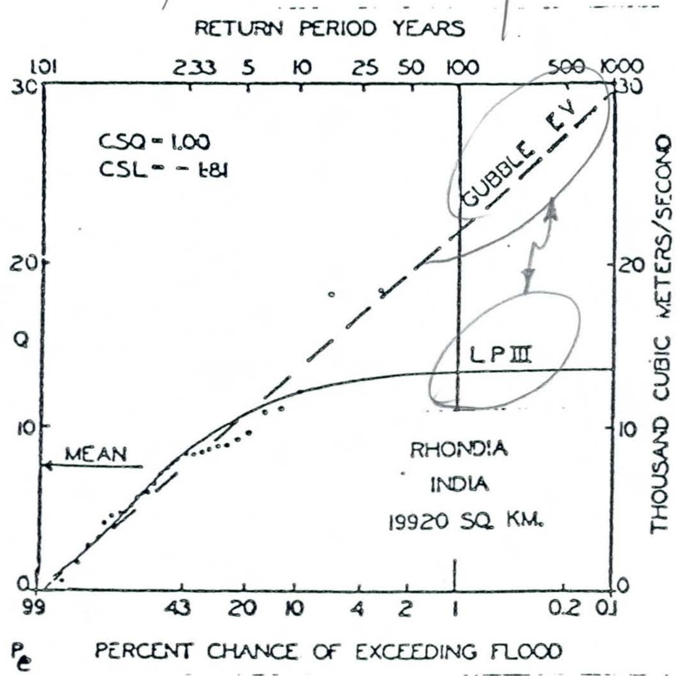
LP III flexibility

develop confidence interval!!

most data developed mean value only

find right distribution interval

LP III problems



Flood Characteristics of Urban Watersheds in the United States

United States Geological Survey Water-Supply Paper 2207

Prepared in Cooperation with U.S. Department of Transportation Federal Highway Administration

U.S. DEPARTMENT OF THE INTERIOR
March 3, 1849

ARIZONA DEPARTMENT OF TRANSPORTATION

DEPARTMENT OF TRANSPORTATION DOT STATE OF ARIZONA

REPORT: ADOT-RS-15(121) FINAL REPORT

Glo ____
File ____

METHODS FOR ESTIMATING THE MAGNITUDE AND FREQUENCY OF FLOODS IN ARIZONA

BY R. H. R

Prepared by:
United States Geological Survey
Water Resources Division
301 W. Congress
Tucson, Arizona 85701
September 1978

Prepared for:
Arizona Department of Transportation
206 South 17th Aven
Phoenix, Arizona 850

In cooperation with t
United States Depart
Federal Highway Admi

two separate manuals

TECHNIQUES FOR ESTIMATING FLOOD HYDROGRAPHS FOR UNGAGED URBAN WATERSHEDS

U.S. GEOLOGICAL SURVEY
OPEN-FILE REPORT 82-365

PREPARED IN COOPERATION WITH

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

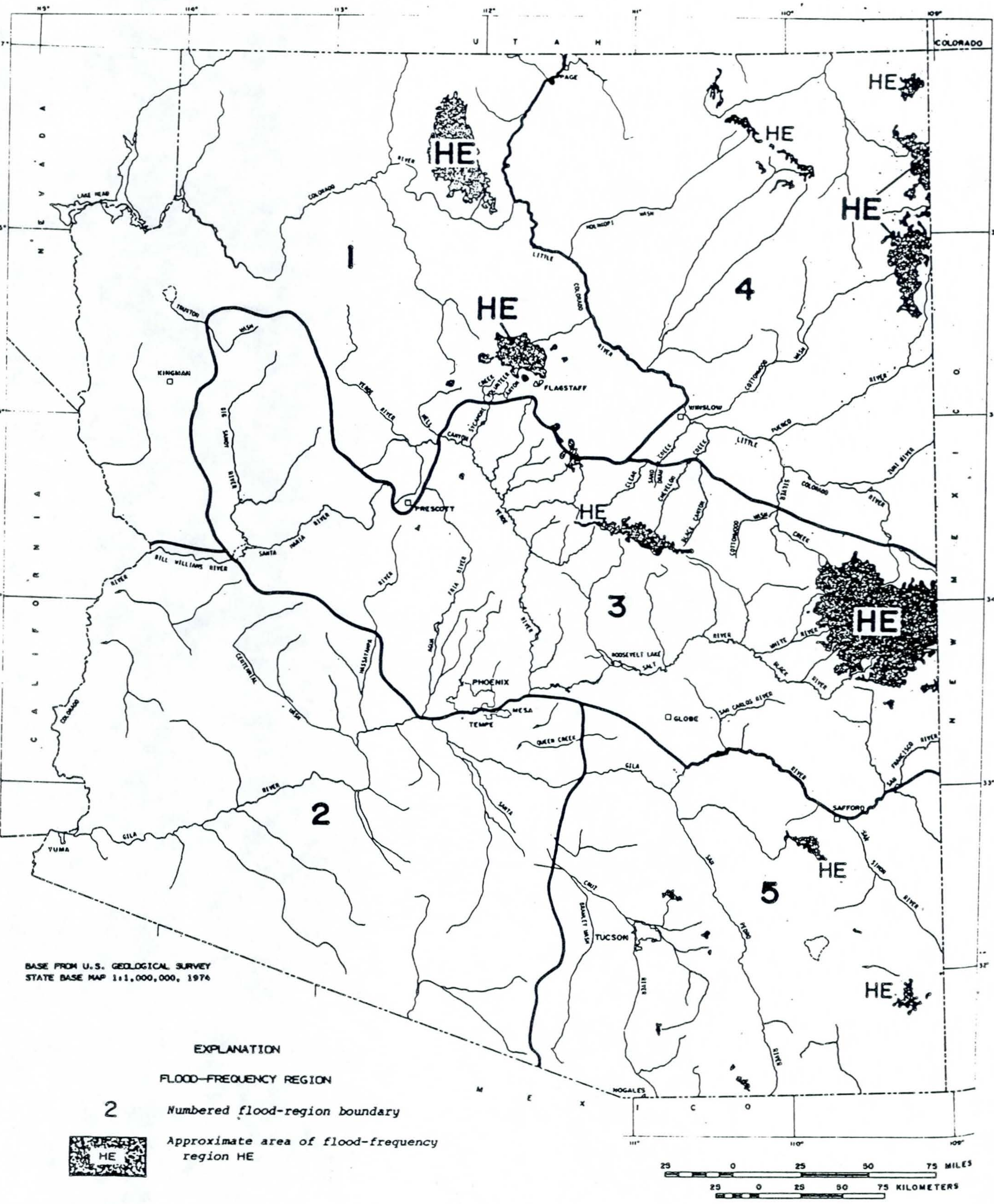


Figure 1.--Flood-frequency regions.

Table 1.--Regression equations for flood magnitudes at selected recurrence intervals and corresponding standard error of estimate—Continued

Equation	Standard error of estimate, in percent
REGION 3—CENTRAL MOUNTAIN AREA (87 STATIONS)	
$Q_2 = 5.66A^{0.673}E^{-0.605}P^{1.03}$	81
$Q_5 = 31.6A^{0.650}E^{-0.868}P^{0.987}$	64
$Q_{10} = 74.7A^{0.638}E^{-1.00}P^{0.971}$	58
$Q_{25} = 186A^{0.626}E^{-1.14}P^{0.944}$	58
$Q_{50} = 329A^{0.617}E^{-1.22}P^{0.933}$	61
$Q_{100} = 553A^{0.610}E^{-1.30}P^{0.915}$	66
$Q_{500} = 1{,}530A^{0.595}E^{-1.45}P^{0.886}$	78
REGION HE—HIGH-ELEVATION REGION (16 STATIONS)	
$Q_2 = 8.78A^{0.853}$	43
$Q_5 = 19.9A^{0.826}$	33
$Q_{10} = 29.6A^{0.816}$	33
$Q_{25} = 44.9A^{0.805}$	38
$Q_{50} = 58.2A^{0.799}$	42
$Q_{100} = 72.9A^{0.795}$	45
$Q_{500} = 113A^{0.787}$	55

area, elevation (mean basin in 1000 feet) average precipitation

variation explained by area only

Tool calibrating mathematical model

no good areal reduction curves
for short duration storms

hydrologic model for
San Cruz
critical storm

Tucson
Pima
County

check into

- Other Regional Methods

process of being revised

- ° SCS TR-55 Method
- ° ADOT Methods
- ° Pima County/City of Tucson Methods

Kinematic wave

- ° Rational Method (See Figures)

D. Flood-Hydrograph Determination

- ° Rainfall/Runoff Models
  - HEC-1 (USACE)
  - TR-20 (SCS)
  - Penn State Urban Runoff
  - Others

underestimate floods in Az due to grass etc in calibration (1) evaluate storm characteristics

- ° Rainfall/Runoff Processes
  - Storm Characteristics (Depth, Spatial and Temporal Characteristics)
  - Runoff Volume (Interception/Infiltration)
  - Overland Flow (LAMINAR flow)
  - Collector Flow
  - Channel Flow

E. Selection of Design Event

- ° Size of Watershed
- ° Seasonal Impacts
- ° Risk
- ° Long-term vs. Short-term Impacts (Dominant Discharge)

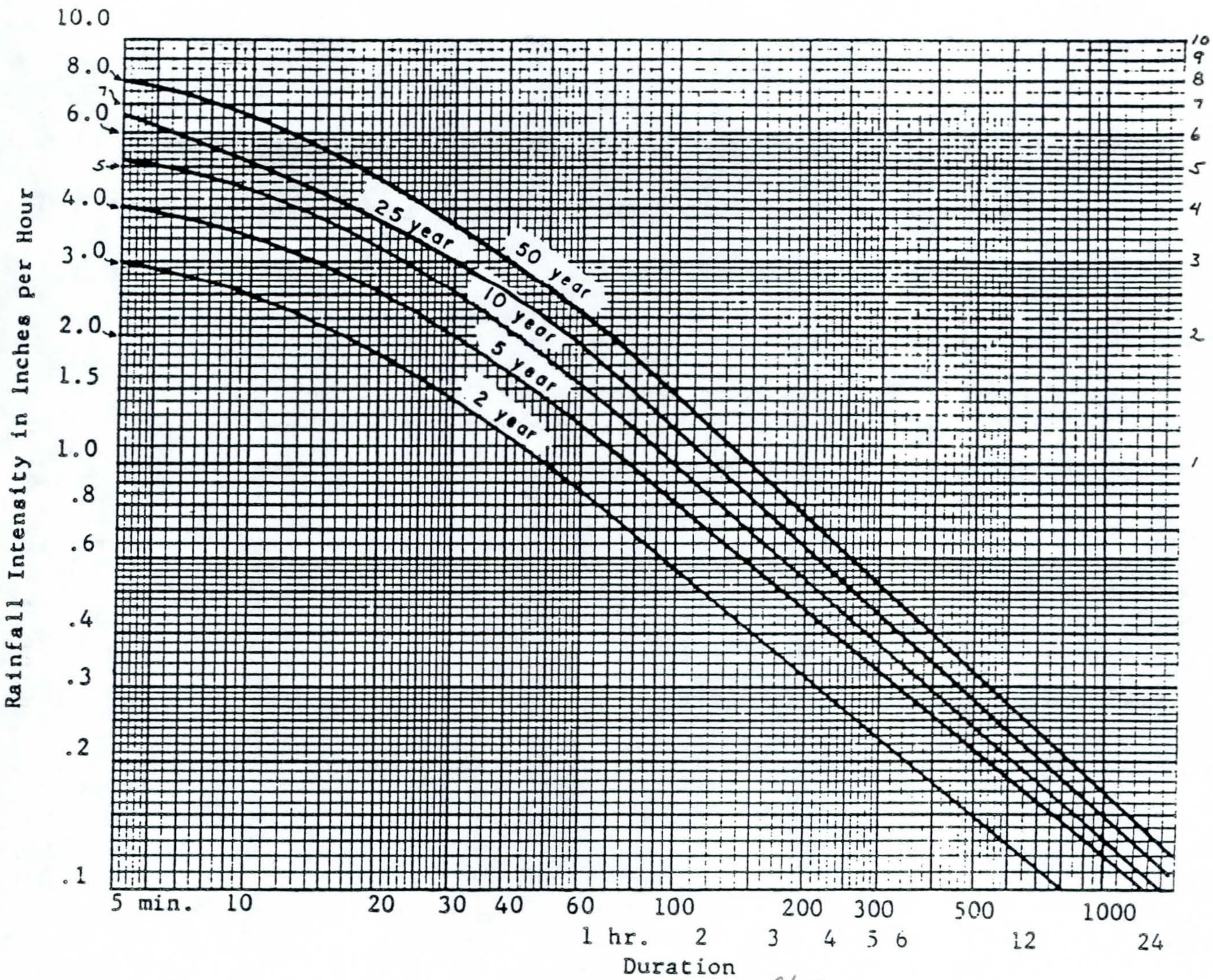
III. HYDRAULIC ANALYSES

By product to hydrology

4.1-4.36 in MANUAL

A. Resistance to Flow

- ° Darch-Weisbach Friction Factor "f"



RAINFALL INTENSITY - DURATION - FREQUENCY RELATION
FOR MESA, ARIZONA

(Partial Duration Series)

Curves are based on methods of U.S. Weather Bureau Technical Paper No. 40, Technical Memorandum WBTM WR-44, and rainfall data from pages 37c through 39 of the Arizona Highway Department Hydrologic Design Manual (1970 Revision).

99

need modified rational method reflects design storm factor

RUNOFF CO-EFFICIENTS FOR USE WITH THE RATIONAL FORMULA $Q = CIA(C_i)$

Land Use	"C" Value
Paved Street or Parking Lot	0.95
Commercial Areas	0.90
Residential Areas (Average lot zoning)	0.45
Townhouses	0.55
Apartments and Condominiums	0.65
Parks and Grassed Areas (no irrigation)	0.20 ( )
Railroad Yards	0.25 (2-3) = 100yr
Undeveloped Desert	0.35 ( )
Mountain Terrain — slopes greater than 10%	0.70
Industrial Areas	0.90
Agricultural Areas	0.20 ( )

2-3 times higher for 100yr

10yr event vs. 100yr event

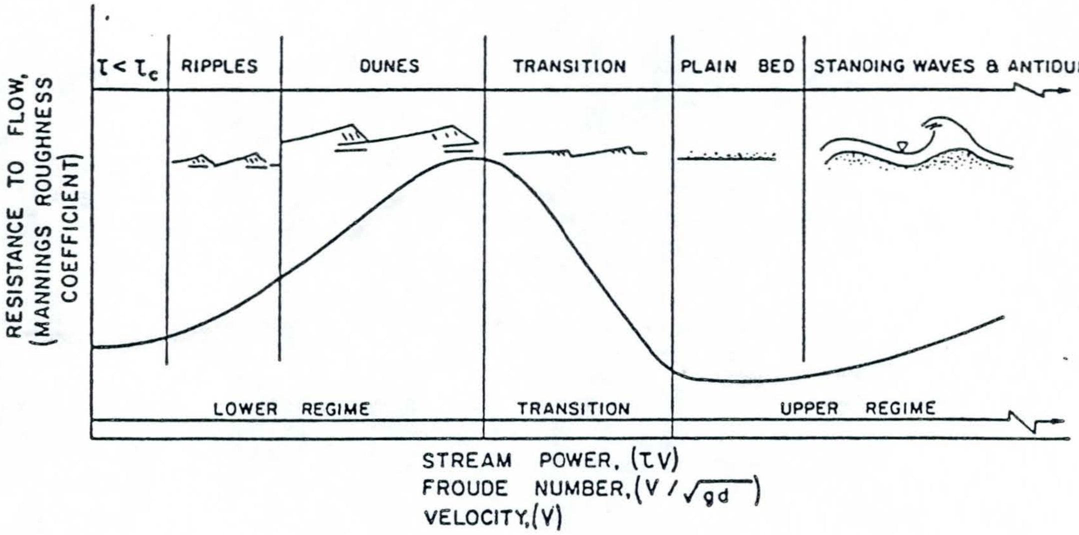
- Chezy Resistance Factor "C"
- Manning Roughness Coefficient "n"
- Inter-relationships
  - $C = (8g/f)^{1/2}$
  - $n = 0.0926R^{1/6}f^{1/2}$
- n-values for Rigid-Boundary Conditions (See Figures)
- n-values for Mobile-Boundary Conditions (See Figures)
  - Fine-grained Channels (Bed Forms)
  - Bed Forms as a Function of Stream Power
  - Considerations for Lined Channels Carrying Sediment-Laden Flows
  - Cobble-bed and Boulder-bed Channels (Bed Forms?)
- n-values for Cobble-bed and Boulder-bed Channels
  - Drag Forces
  - Porous-Media Flow
  - Complex Resistance Equation (Bathurst)

B. Boundary Shear Stress

- Basic Relationship, $\tau = \gamma RS$ (Alternate, $\tau = 1/8\rho fV^2$)
- Distribution Along the Boundary
  - Variation in a Trapezoidal Cross Section (See Figures)
  - Maximum Unit Tractive Force Versus Aspect Ratio (See Figures)
- Shear Stress in a Bend
  - Distribution (See Figures)
  - Variation (See Figures)
  - Downstream Impacts
- Permissible Shear Stress

TABLE 5-6. VALUES OF THE ROUGHNESS COEFFICIENT *n* (*continued*)

Type of channel and description	Minimum	Normal	Maximum
C. EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
D. NATURAL STREAMS			
D-1. Minor streams (top width at flood stage <100 ft)			
a. Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150



use lower values for sediment routing

higher values for peak discharge (maximum depths)

Bed Roughness	Typical Range	Recommended Value for Flood Studies	Recommended Value for Sediment Transport Studies
Ripples	0.018-0.030	0.030	0.022
Dunes	0.020-0.035	0.035	0.030
Transition	0.014-0.025	0.030	0.025
Plane Bed	0.012-0.022	0.030	0.020
Standing Waves	0.014-0.025	0.030	0.020
Antidunes	0.015-0.031	0.030	0.025

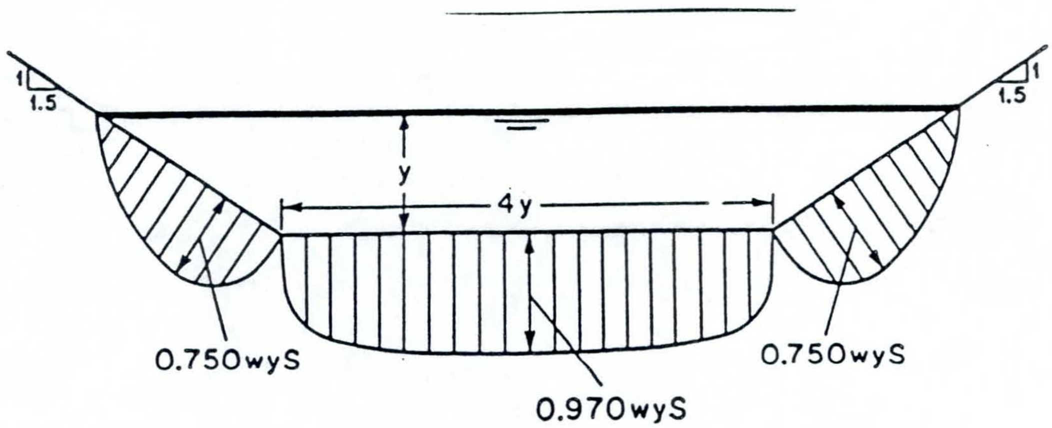


FIG. 7-6. Distribution of tractive force in a trapezoidal channel section.

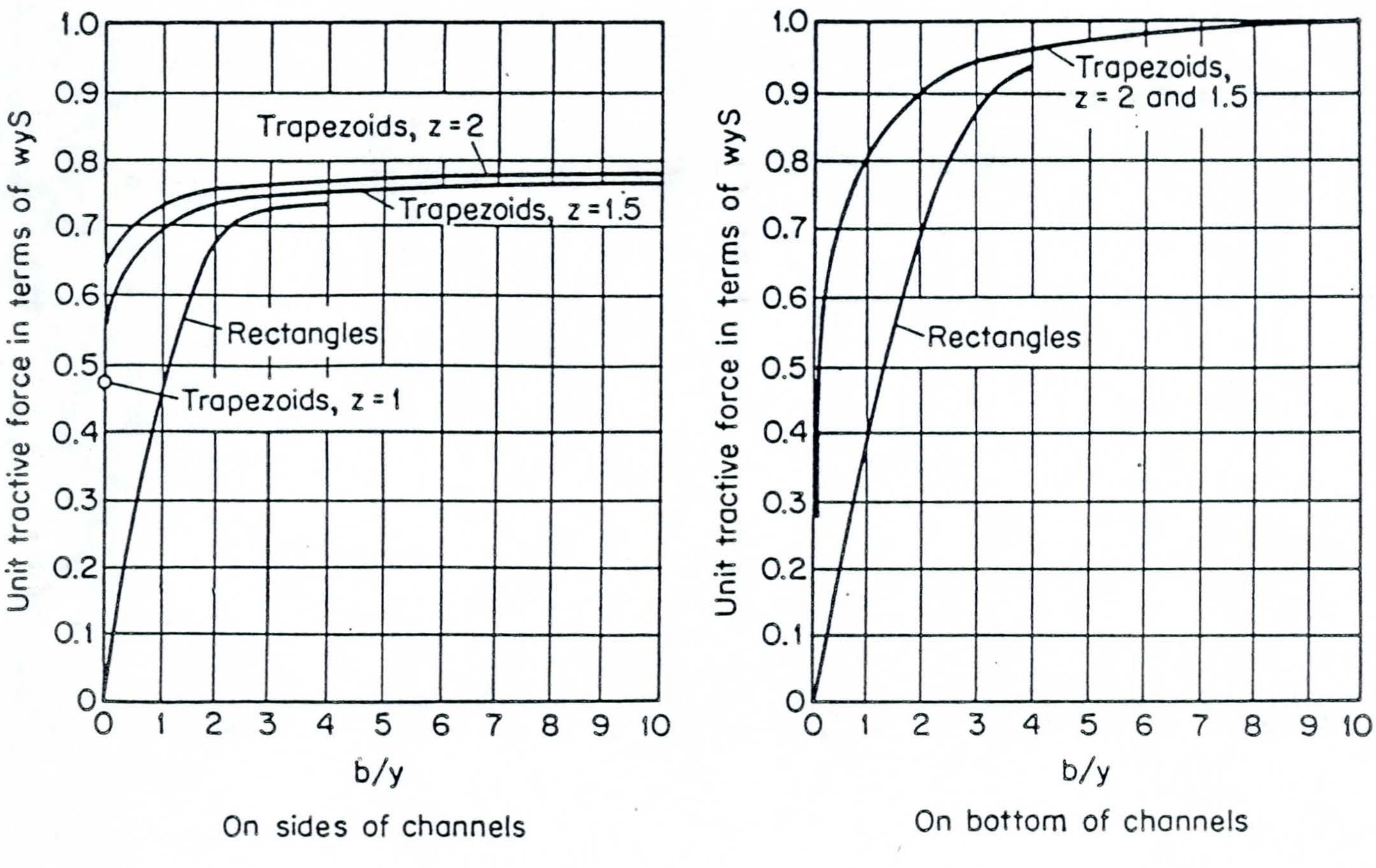
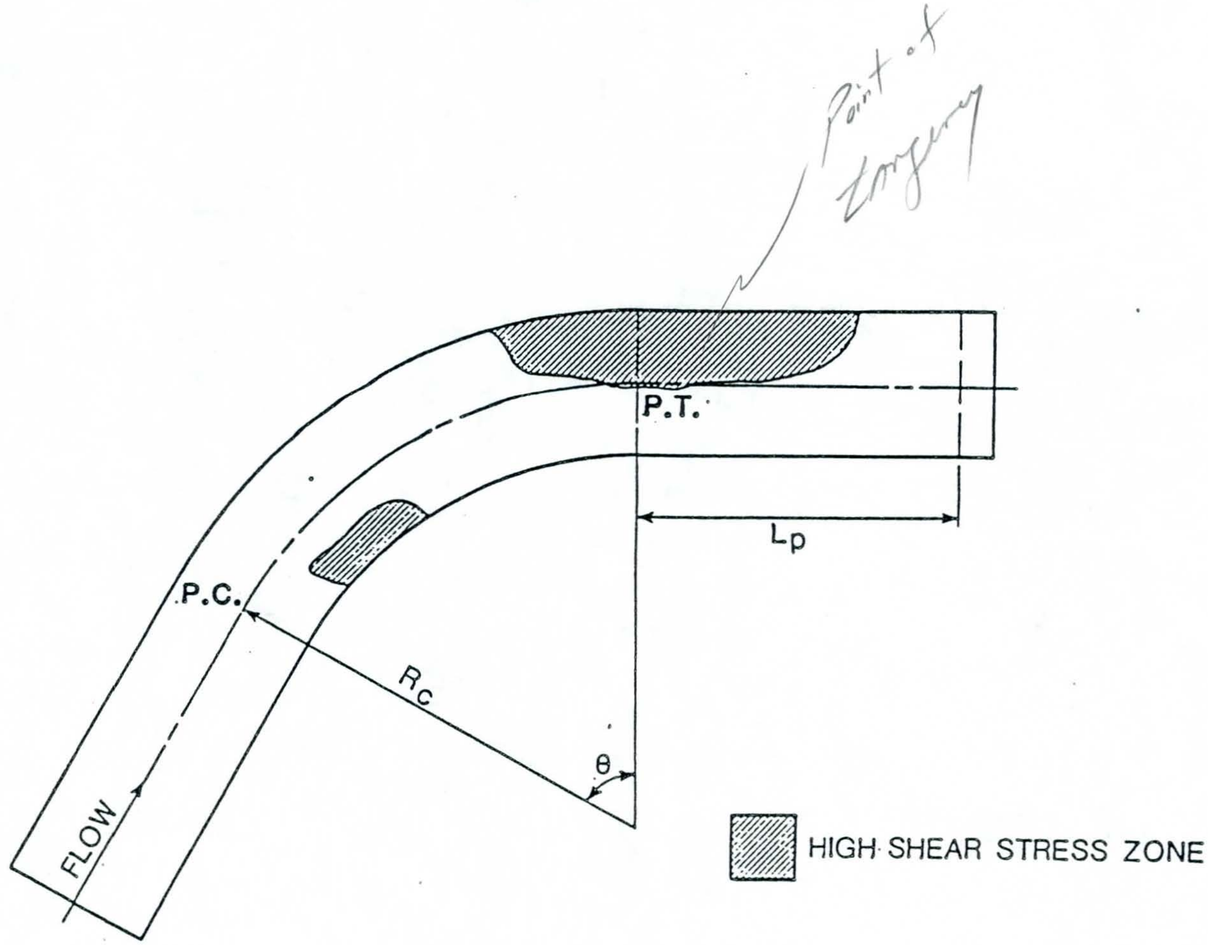
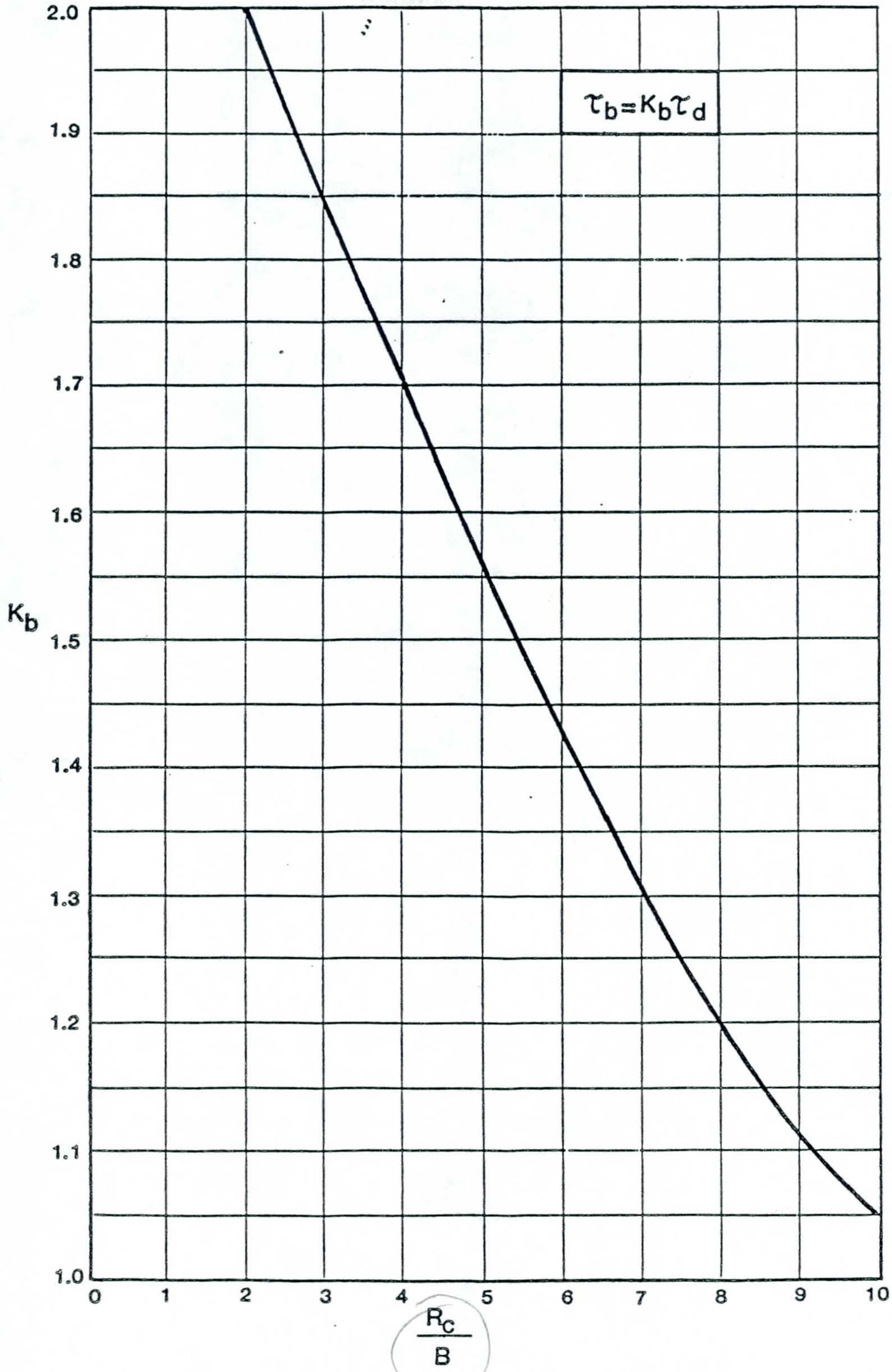


FIG. 7-7. Maximum unit tractive forces in terms of wyS.



Shear Stress Distribution in a Channel Bend

Chart 10



radius of curvature to bottom width

Chart 10. K_b factor for maximum shear stress on channel bends. (12)

- Cohesive Materials
- Non-cohesive Materials
- Impacts of Vegetation
- Impacts of Cementation
- Typical Values (See Figures)

° Shear Stress as Related to Flow Velocity

- Maximum Versus Mean Shear Stress/Velocity
- Permissible Velocity as a Function of Permissible Shear Stress

C. The Concept of Flow Control

° Uniform Flow (Normal Depth)

- Resistance to Flow Balanced by Gravational Force
- Slope of Energy Grade Line Equals Bed Slope

° Crictical Flow (Critical Depth)

- Minimum Engergy
- Point of Control
- Froude Number, F, = 1.0 ($F = V/(g\gamma h)^{1/2}$)

° Subcritical Flow (Greater than Critical Depth)

- Control is Downstream

° Supercritical Flow (Less than Critical Depth)

- Control is Upstream

° Water-Surface Profiles

- Gradually-varied Flow
- HEC-2 (USACE)
- HY-7 (USGS)

D. Additional Influences Upon Flow Depth

Chart 1

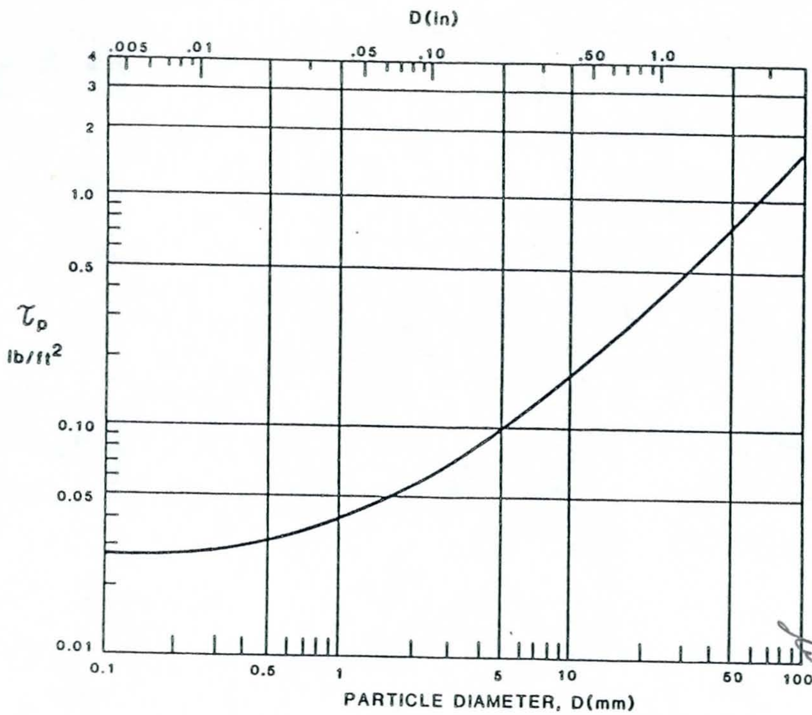


Chart 1. Permissible shear stress for non-cohesive soils. (after 15)

unsuitable to SW soil
due to cementation

Chart 2

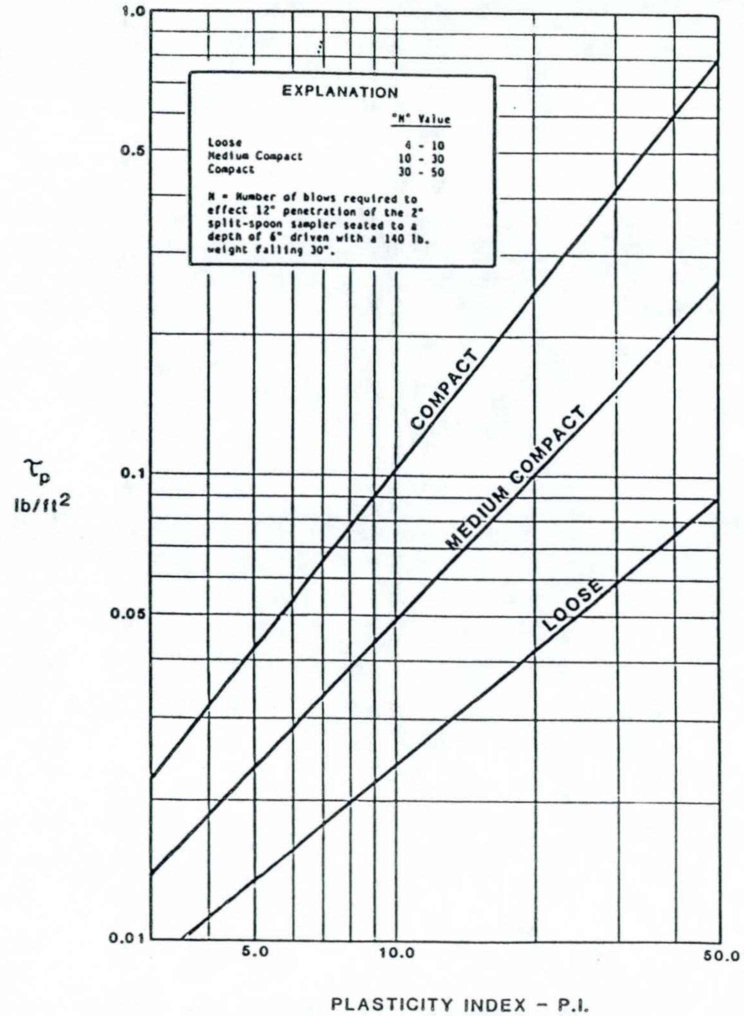


Chart 2. Permissible shear stress for cohesive soils. (after 16)

locate controls

1) critical depth

- Bed Forms
  - Dunes (1/2 Y, bed)
  - Antidunes ($0.027V^2$, bed or surface, but no greater than 1/2 Y
- Superelevation in Bends
  - Rectangular Channels
  - Trapezoidal Channels
  - Formula (See Figures)
- High-Velocity Flow Separation in Sharp Bends significant factor
  - $\Delta Y = 0.25V^2/2g$
  - Applies When $W/r_c > 0.33$
- Debris Accumulation
  - At Bridge Crossings
  - At Culvert Crossings
  - At Fences and/or Other Barriers
- Long-term Aggradation
  - Filling of Channel by Sediments Due to Sediment-Transport Discontinuity

E. Total Freeboard Requirement

- Bank Lining Alone, Use First Three Components Described Under III. D.
- Channel Wall Height, Use All Five Components Described Under III. D.
- Formulas
  - $F.B_{BL} = 1/2ha + \Delta Y_{se} + \Delta Y_s$

    ha = Antidune Height (standing waves)

    ΔY_{se} = Superelevation

    ΔY_s = Flow Separation (Bank Lining Only)

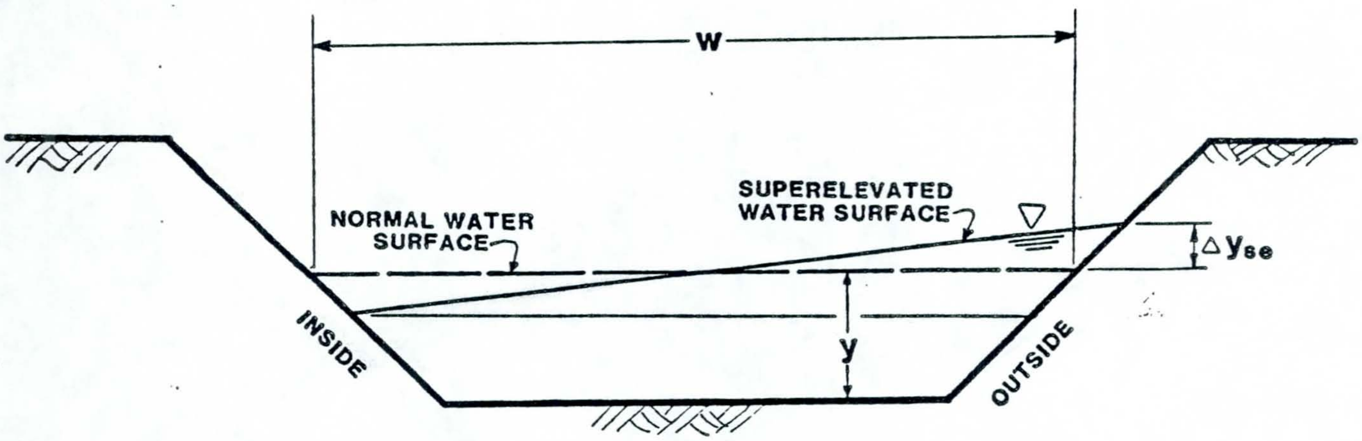


Figure 4.8a Definition sketch of superelevation in a channel bend.

undulation motion with supercritical requires twice the height

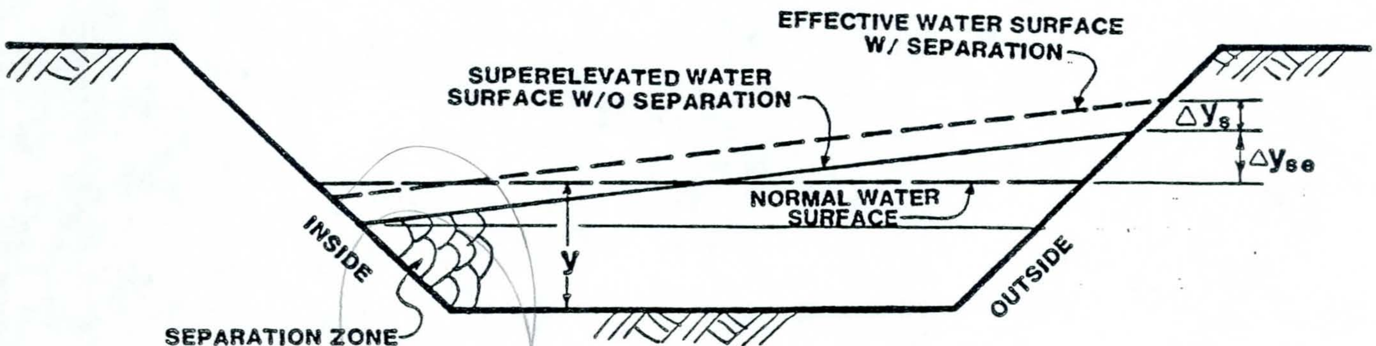


Figure 4.8b Definition sketch of superelevation and flow separation conditions in a short radius bend.

usually ignored

$$- F.B_{TOT} = F.B_{BL} = \Delta Y_d + \Delta Y_{agg}$$

ΔY_d = Debris Accumulation

ΔY_{agg} = Long-term Aggradation

3 main mechanisms (Keown et Al., 1977)

1) widening
2) deepening
3) Sinuosity change

Sediment Transport Process

(1) - Erosion:
(2) - Entrainment
(3) - Transportation
(4) - Deposition
(5) - Compaction

All are seeking Equilibrium (minimum energy)

QUALITATIVE GEOMORPHIC ANALYSES

LECTURE 3

ROBERT C. MACARTHUR, Ph.D., P.E.

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3555 Stanford Road
P.O. Box 1816
Fort Collins, Colorado 80522

July 1986

LECTURE 3: Level I - Qualitative Geomorphic Analysis

OBJECTIVE: This lecture presents the practical use of geomorphic principles for the analysis of fluvial systems. The geomorphic discipline provides a number of concepts and tools that greatly assist the engineer in the planning and design tasks associated with rivers. These tools are able to address both long- and short-term changes in the river environment. The critical problem of man's influence on the river environment can be addressed by considering the external stresses that man's activities cause. The gradual increase in external stress may eventually produce a dramatic response in the system.

I. GENERAL INTRODUCTION

Geomorphology is a field in the discipline of geology that is concerned with the physical and chemical processes that shape the earth's land forms. Fluvial geomorphology concentrates on the morphology (shape) of rivers and river systems. This broad overview of the river system is important from an engineering standpoint, since it is rarely possible to understand even a short reach of an alluvial channel in isolation from its upstream and downstream controls. The engineering analyses of a river reach requires integration of upstream geology, morphology, and hydrology. The geomorphic level of analysis can be divided into two parts:

1. Plan form analysis.

2. Longitudinal profile analysis.

The key objective of the geomorphic analysis is the classification of the river with the goal of identifying areas that are unstable or that constitute a hazard to human activity. Geomorphic techniques are available that can identify trends in river processes and unstable conditions. These techniques rely on a substantial data base that includes aerial photography, maps, and field measurements.

II. DATA REQUIREMENTS

Data requirements include information on river geometry; geologic conditions; historic data on flood events, river development and changes in land use; hydrologic conditions; and sediment characteristics.

A. Geologic Data

1. Structure
2. Major formations
3. Tectonic activity

B. River Geometry

1. Channel cross-sectional characteristics
2. Channel gradient
3. Valley gradient and channel sinuosity
4. Radius of channel bends

C. Historic Data

1. Flood history
2. Gage record (changes in datum)
3. Channel alignment history (from aerial photos)
4. Land-use changes
5. Records of cross-sectional changes

D. Hydrologic Data

1. Frequency and duration of floods
2. Estimation of bankfull discharge
3. Dominant discharge
4. Watershed area
5. Response time for rainfall/runoff
6. Soil type
7. Vegetative cover
8. Infiltration characteristics

E. Sediment Characteristics

1. Channel bed and bank material characteristics
2. Sediment sources in the watershed
3. Measurements of sediment yield
4. Deposition in reservoirs or detention ponds
5. Measured sediment concentration in the river

F. Data Sources

1. Topographic maps
2. Planimetric maps
3. Aerial photographs (See Table 1 for agencies who provide information on aerial photos)
4. Transportation maps
5. Triangulation and benchmarks
6. Geologic maps
7. Soil data
8. Climatological data
9. Streamflow data
10. Sedimentation data
11. Water-quality data
12. Irrigation and drainage data
13. Flood-control data
14. Hydro-power data

Table 1. Agencies with Information on Aerial Photographs.

EROS Data Center
U.S. Geological Survey
EROS Data Center
User Services Section
Sioux Falls, South Dakota 57198
Telephone: (605)594-6151

NCIC Headquarters
National Cartographic Information Center
U.S. Geological Survey
507 National Center
Reston, Virginia 22092
Telephone: (703)860-6045

NCIC Offices
Eastern Mapping--NCIC
U.S. Geological Survey
536 National Center
Reston, Virginia 22092
Telephone: (703)860-6336

Mid-Continent Mapping Center--NCIC
U.S. Geological Survey
1400 Independence Road
Rolla, Missouri 65401
Telephone: (314)341-0851

National Cartographic Information Center
U.S. Geological Survey
National Space Technology Laboratories
NSTL Station, Mississippi 39529
Telephone: (601)688-3544

Rocky Mountain Mapping Center--NCIC
U.S. Geological Survey
Box 25046, Stop 504 Federal Center
Denver, Colorado 80225
Telephone: (303)234-2326

Western Mapping Center--NCIC
U.S. Geological Survey
345 Middlefield Road
Menlo Park, California 94025
Telephone: (415)323-8111, ext. 2427

National Archives Cartographic Division
Attention: Richard Spurr
841 South Pickett Street
Alexandria, Virginia 22304
Telephone: (703)756-6704

15. Basin and project reports
16. Environmental reports
17. Personal interviews
18. Paleohydrologic evidence
19. Dairies and inspection records
20. Field investigation

G. Other Important Factors to Consider

1. Movable versus fixed boundaries; concept and approach
2. Alluvial channels
3. Why worry about sediment problems?

    - Can decrease degree of flood protection
    - Removes valuable storage
    - Interferes with navigation
    - Can cause unstable banks and undermine channel crossings
    - Can adversely impact the environment

4. Important sedimentation process

    - Erosion, land surface, stream channel and banks
    - Entrainment
    - Transportation
    - Deposition
    - Compaction

H. All of these Processes are Equilibrium Seeking and are Time Dependent

1. Different scales of time are important

    - Geologic time (10^4 to 10^7 years)

        ° Basins filling, mountains and plains degrading, swamps forming, changing river location and plan

    - Project time (100 years or less)

        ° River meandering, channel degradation or aggradation, channel widening, etc.

    - Major flood (hours to several days)

        ° Changes in plan form and channel geometry, meander, bar movement

    - Instantaneous (hours)

        ° Changes in river discharge and sediment loads

Sedimentation Problems

1) overlooked

2) misunderstood

Solutions

Multilevel Approach for Evaluating Sedimentation

1- Identify problems + conduct qualitative

2- Refine problem

3- Conduct thorough Quantitative analysis

III. PLAN FORM ANALYSIS

Natural channel plan form of rivers cover a continum of patterns, ranging from straight, to meandering, to braided characteristics. Plan form of a channel is determined by the interaction of numerous variables whose range in nature is continuous, one should not be surprised at the existence of a complete range of channel patterns. A river reach, then, may exhibit both braiding and meandering, as the alteration of controlling parameters changes the character of a given river plan form.

Rivers can be classified broadly in terms of channel pattern, that is, the configuration of the river as viewed on a map or from the air. The patterns are straight, meandering, braided, or some combination of these (Figure 1).

A straight channel can be defined as one that does not follow a sinuous course. Leopold and Wolman (1957) have pointed out that truly straight channels are rare in nature. Although a stream may have relatively straight banks, the thalweg, or path of greatest depth along the channel, is usually sinuous (Figure 1.b). As a result, there is no simple distinction between straight and meandering channels.

The sinuousity of a channel, defined as the ratio between the thalweg length and the down-valley distance, is most often used to distinguish between straight and meandering channels. Sinuosity varies from a value of unity to a value of three or more. Leopold, Wolman and Miller (1964) took a sinuosity of 1.05 as the division between meandering and straight channels. It should be noted that in a straight reach with a sinuous thalweg developed between alternate bars (Figure 1.b), a sequence of shallow crossings and deep pools is established along the channel.

A braided stream or river is generally wide with poorly defined and unstable banks, and is characterized by a steep, shallow course with multiple channel divisions around alluvial islands (Figure 1.a). Braiding was studied by Leopold and Wolman (1957) in a laboratory flume. They concluded that braiding is one of many patterns which can maintain quasi-equilibrium among the variables of discharge, sediment load, and transporting ability. Lane (1957) concluded that, generally, the two primary causes that may be responsible for the braided condition are (1) overloading, that is, the stream may be supplied with more sediment than it can carry, resulting in deposition of part of the load; and (2) steep slopes, which produce a wide, shallow channel where bars and islands form readily.

A meandering channel is one that consists of alternating bends, giving an S-shape appearance to the plan view of the river (Figure 1.c). More precisely, Lane (1957) concluded that a meandering stream is one whose channel alignment consists principally of pronounced bends, the

8
20
1600
50
8000
47
50000

25
40
1000
52000

1- Historical
2- Site Reconnaissance
3- Qualitative assessment

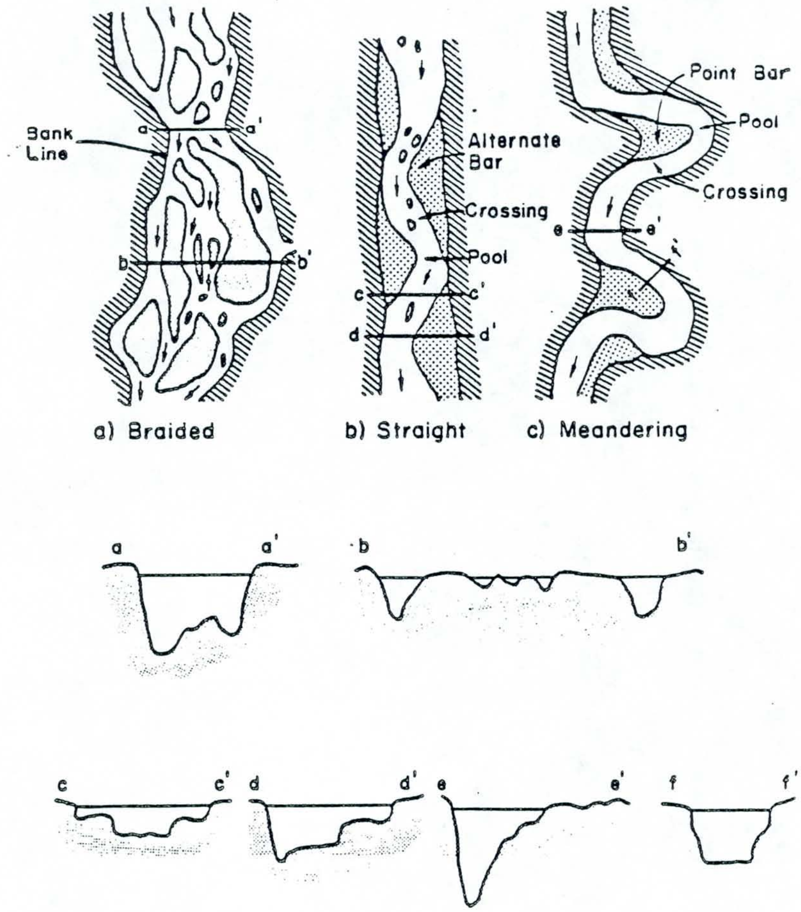


Figure 1. River channel patterns.

shapes of which have not been determined predominantly by the varying nature of the terrain through which the channel passes. The meandering river consists of a series of deep pools in the bends and shallow crossings in the short straight reach connecting the bends. The thalweg flows from a pool through a crossing to the next pool forming the typical S curve of a single meander loop.

A number of classification schemes have been developed to assist in recognizing various river patterns. There are also quantitative relationships that can be used to categorize river form. Figure 2 illustrates the geomorphic information that is readily obtained from a sequence of aerial photographs. Use of aerial photographs is particularly valuable in identification of river plan form. When aerial photographs are available over a period of years they can provide a valuable record with which to identify channel form and assess channel stability. Plotting overlays of channel pattern and movement as a function of time often reveals alarming instabilities.

The objective of plan form analysis is to identify conditions of channel instability. Changes in channel classification signal lateral instability of the channel, which can cause significant problems at river structures. Plan form analysis seeks to identify these problems.

A. Classification of River Channels (Figure 3)

1. Channel width
2. Hydrologic regime
3. Bed material
4. Valley setting
5. Flood plain
6. Degree of sinuosity
7. Degree of braiding
8. Degree of anabranching
9. Variability of width and development of bars
10. Apparent incision
11. Cut banks
12. Bank material
13. Vegetative and tree cover on banks

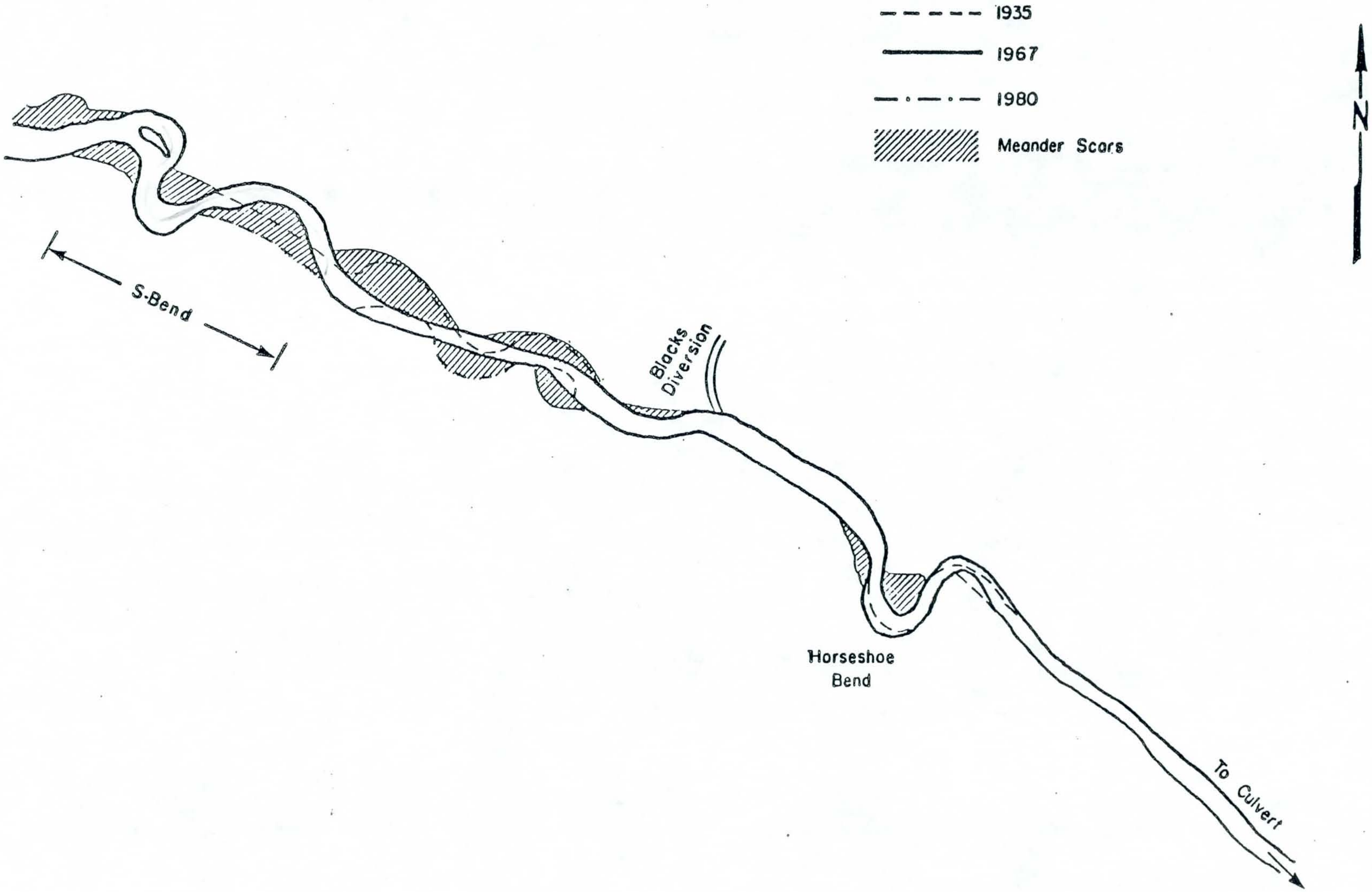


Figure 2. Example illustrating qualitative information derived from time sequence analysis of aerial photographs.

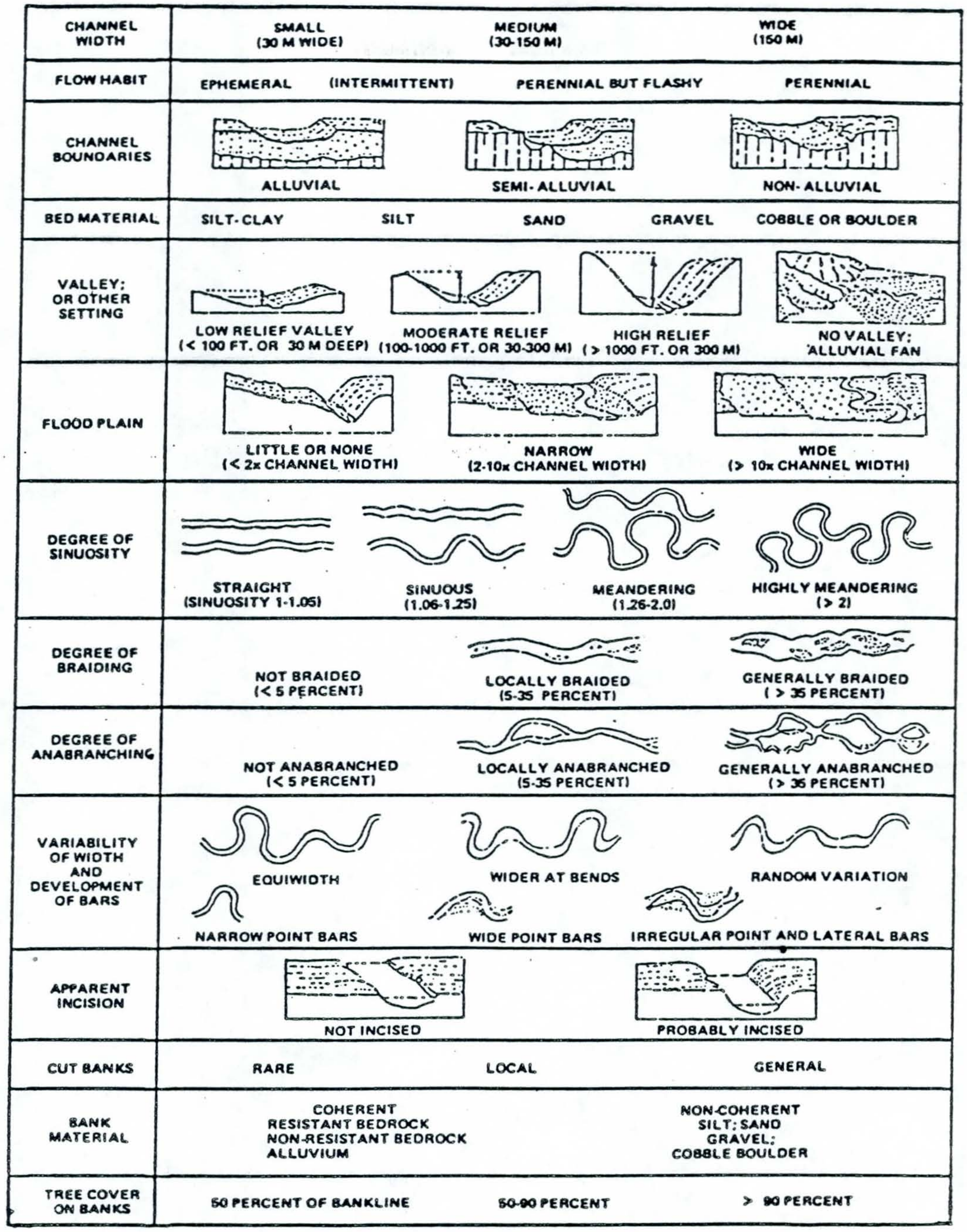


Figure 3. Stream properties for classification stability analysis.

B. Investigations have also focused on the relationship between channel characteristics, such as slope and sinuosity, and channel patterns (straight, meandering, braided). Results of Friedkin (1945), Leopold and Wolman (1957), and Lane (1957) suggest that for a given discharge there is a threshold slope separating braided and meandering channels. Figure 4 summarizes the various results, which in general can be fitted by equations of the form

$$S\ Q^{\alpha} = K$$

where S is the channel slope, Q is the discharge, α is a coefficient and K is a constant. The data used to develop these relationships included both laboratory results and field measurements for predominantly sand-bed channels.

1. $SQ^{1/4} = K$ for sand-bed channels
2. $K < 0.0017$ meandering pattern
3. $K > 0.01$ braided pattern
4. $0.0017 < K < 0.01$ intermediate pattern

C. Concept of Kahn's Relationship (Figure 5)

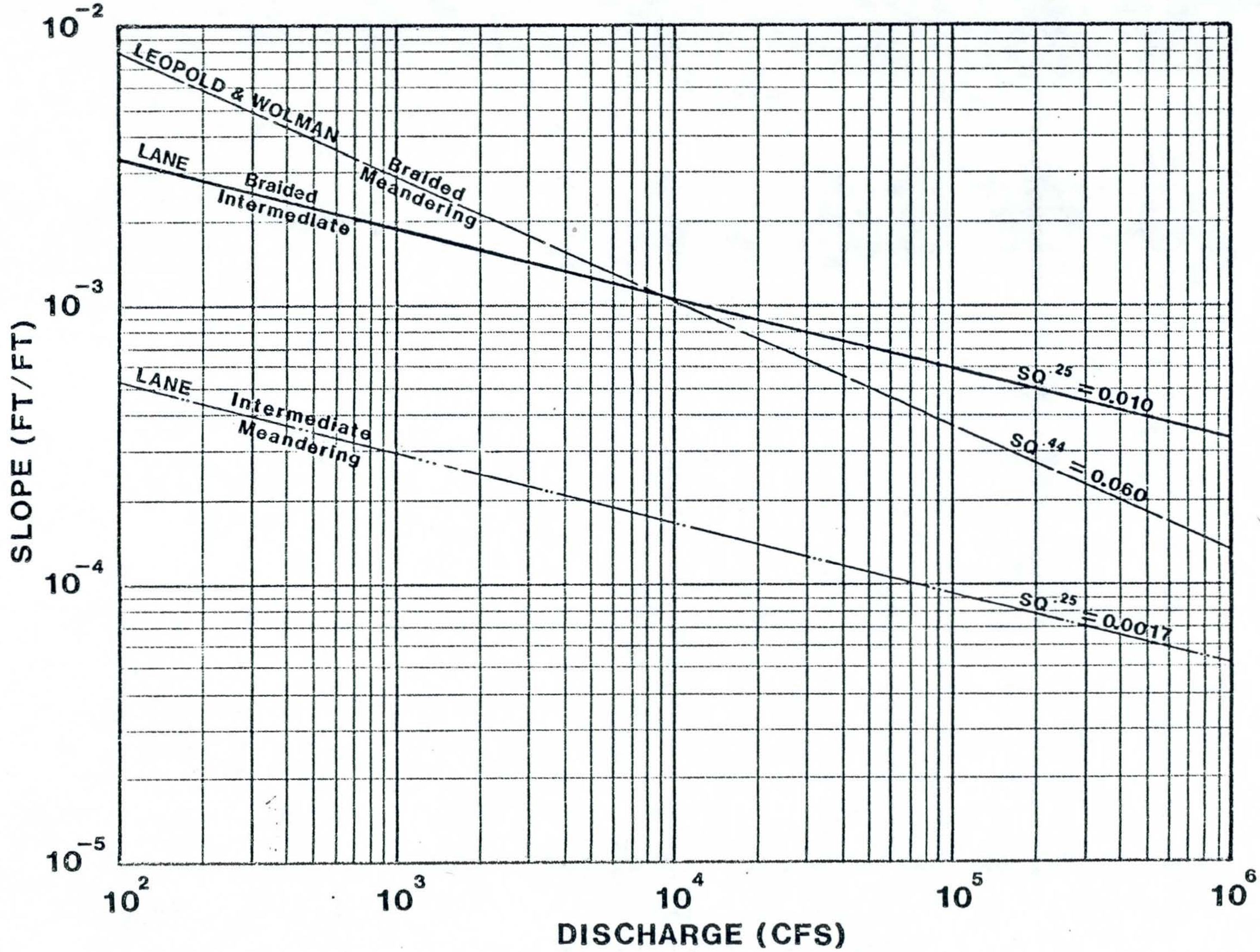
1. Shift from intermediate pattern to braided
2. Qualitative, based on laboratory tests

IV. LONGITUDINAL PROFILE ANALYSIS

The longitudinal profile of a stream shows its slope, or gradient, and provides a visual representation of the ratio of the fall of a stream to its length of a given reach. Rivers are generally steepest in the upper reaches, with milder profile gradients in the downstream direction. Figure 6 illustrates this. The shape of the profile is the result of a number of interdependent factors. On the average, it represents a balance between the sediment-transport capacity of the river and the size and quantity of the sediment load supplied.

The longitudinal profile is dynamic and adjusts continually to changes in water discharge and sediment load. If a river is unable to transport the incoming sediment load, a deposition of the sediment load will take place that will build up the channel bed (Figure 7). This deposition process will cause an increased channel gradient that in turn will increase the sediment-transport capacity of the river. If a river develops an excess ability to transport sediment, a scouring of sediment from the channel bed will take place (Figure 7). This scour process will cause a decrease in channel gradient that in turn will decrease the sediment-transport capacity of the river.

dominant discharge
vs
extreme discharge



energy slope

Figure 4. Slope-discharge relations.

Sclumm

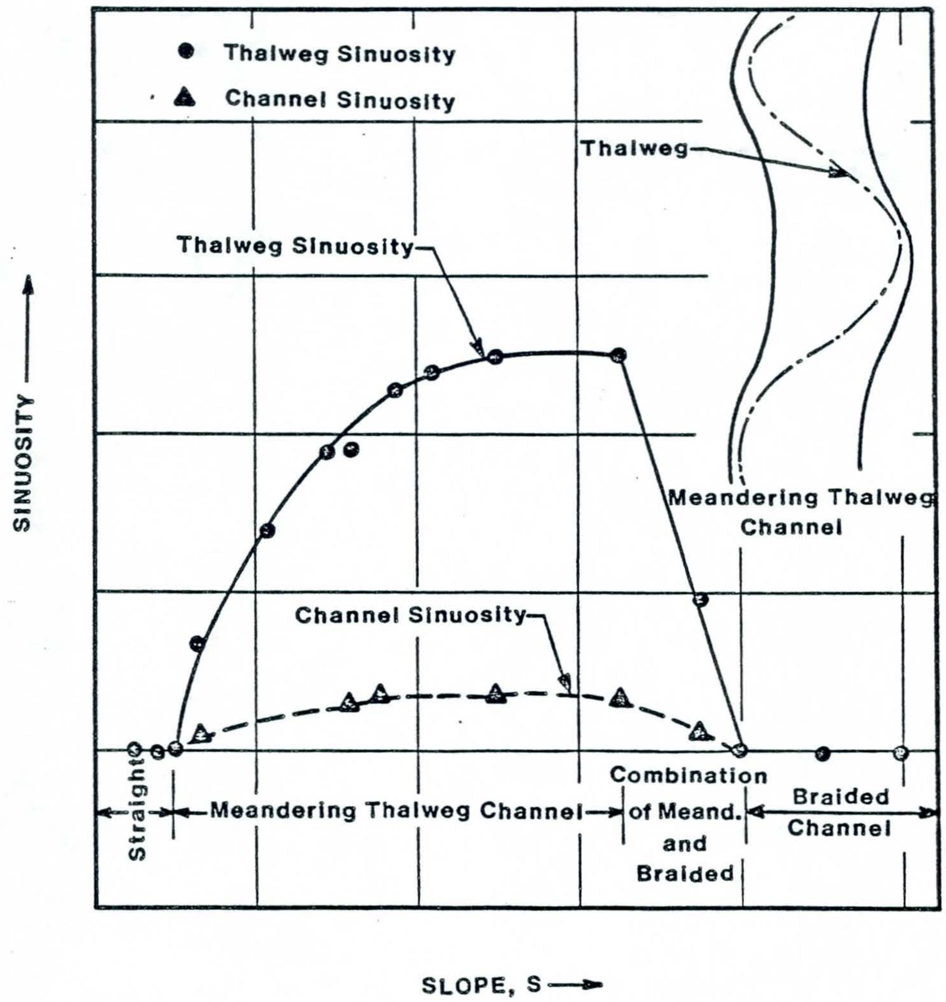


Figure 5. Channel pattern versus slope and simuosity (Kahn, 1971)

hydraulic energy

Function / 1) slope
2) QP
(3) source

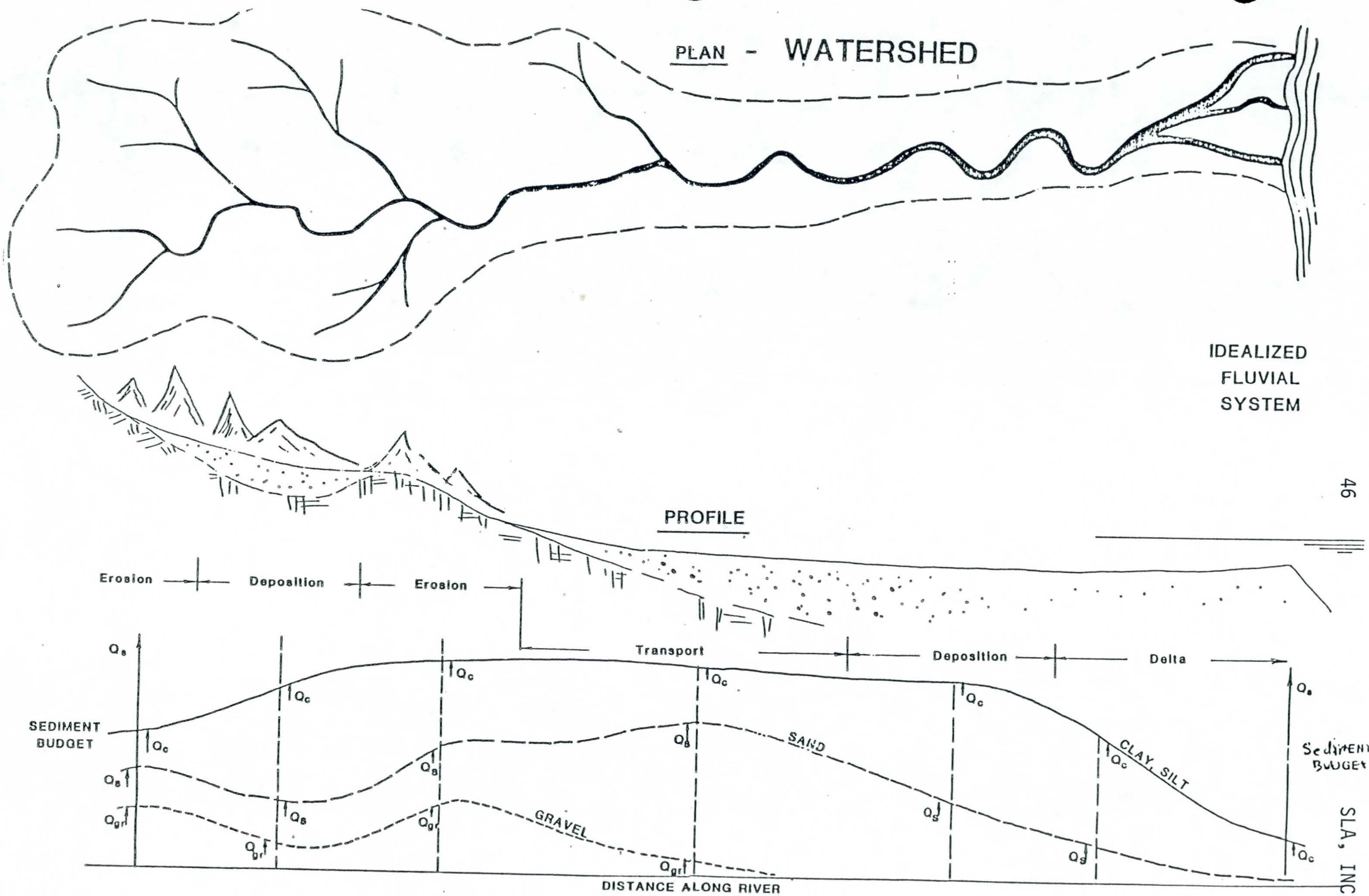

PLAN - WATERSHED
IDEALIZED FLUVIAL SYSTEM
PROFILE
Erosion
Deposition
Erosion
Transport
Deposition
Delta
SEDIMENT BUDGET
SAND
CLAY, SILT
GRAVEL
Sediment Budget
DISTANCE ALONG RIVER

Figure 6.

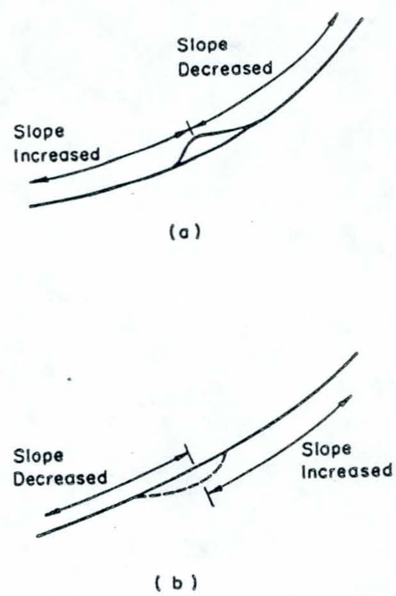


Figure 7. Adjustments in longitudinal profile (Morisawa, 1968).

Lane's

$Q_s * \frac{D_{50}}{C_1} \approx \frac{Q * S_f}{C_2}$

assumed constant

either from bed or banks

Q_s = Bed Material

D_{50} = Average Sediment Size

Q = Discharge

S_f = E.G.L. slope

A. Time-Sequenced Thalweg Profiles

1. Developed from available mapping

2. Or, from historic profile or cross-sectional measurements

B. Lane's Relationship

1. Investigations of channel response have found the following relationships:

   a. Depth of flow is proportional to water discharge and inversely proportional to sediment discharge

   b. Channel width is directly proportional to both water and sediment discharge

   c. Channel shape (expressed as width to depth ratio) is directly proportional to sediment discharge

   d. Channel slope is inversely proportional to water discharge and directly proportional to sediment discharge and mean sediment grain size

   e. Sinuosity is directly proportional to valley slope and inversely proportional to sediment discharge

   f. Transport of bed material is proportional to stream power and concentration of wash load, and inversely proportional to the fall velocity of the bed material

2. The resulting qualitative relationship is very useful for predicting river response.

$$Q_s \propto \frac{(\tau V)\, W C_f}{D_{50}}$$

where τ = shear stress on the channel bed
V = average channel velocity
C_f = wash load concentration
W = channel top width
D_{50} = mean sediment grain size
Q_s = sediment discharge

3. Lane simplified this relation to obtain:

$$QS \propto Q_s D_{50}$$

qualitative relationship

C. Application of Lane's Relation (summarized in Figure 8)

1. Degradation below a reservoir
2. Lowering of base level for a tributary
3. Straightening of a channel

D. Examples of Lane's Relation can be used as a good qualitative assessment tool are provided in Figures 9 through 14.

V. GENERAL OPEN-CHANNEL DESIGN CONCEPTS TO KEEP IN MIND DURING YOUR LEVELS I AND II STUDIES

The ideal channel is a natural one carved by nature over a long period of time. The benefits of such a channel are that:

- Velocities are usually low, resulting in longer concentration times and lower downstream peak flows.

- Channel storage tends to decrease peak flows.

- Maintenance needs are usually low because the channel is somewhat stabilized.

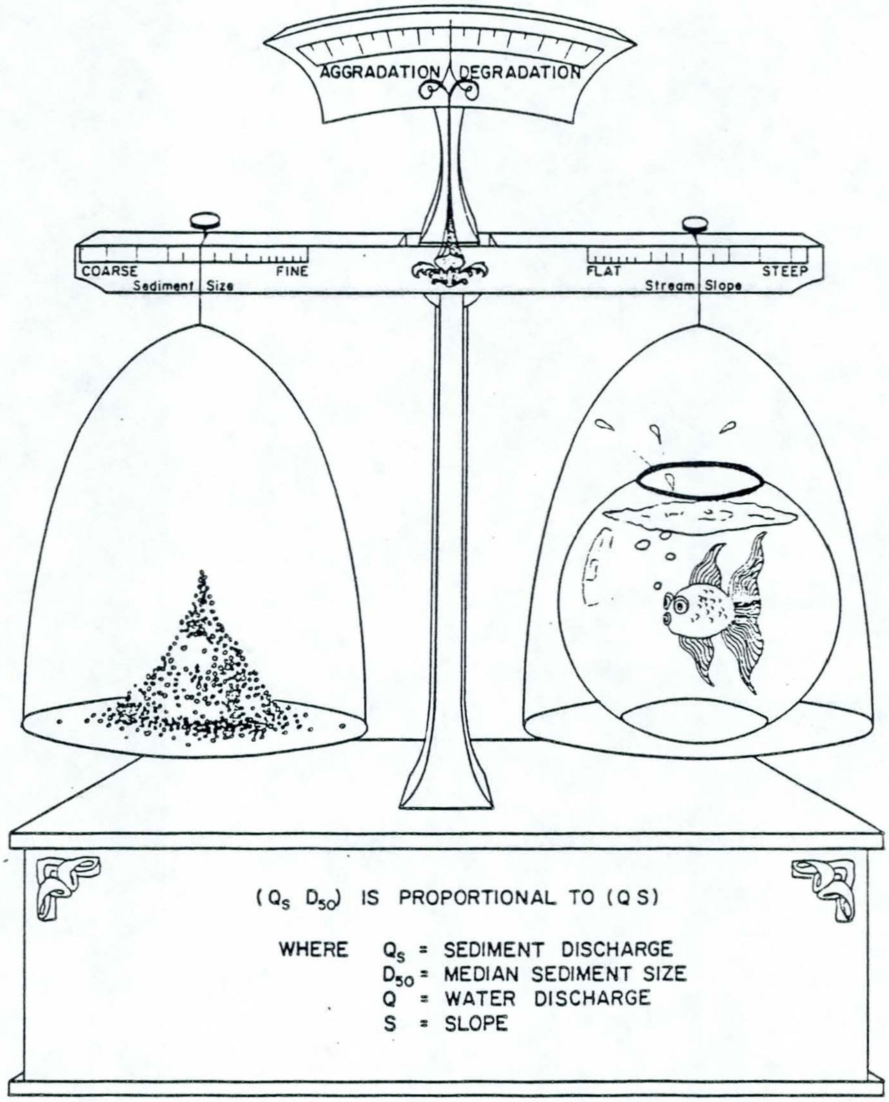
The closer an artificial channel character can be made to that of a natural channel, generally the better the artificial channel.

Channel stability is a well recognized problem in urban hydrology because of the significant increase in low flows and peak storm runoff flows. A natural channel must be studied to determine what measures are needed so as to avoid future bottom scour and bank cutting. Erosion-control measures can be taken which will preserve the natural appearance, not be costly, and function properly.

A. Choice of Channel

The choices of channels available to the designer are almost infinite, depending only upon good hydraulic practice, environmental design, sociological impact, and basic project requirements. However, from a practical standpoint, the basic choice to be made initially is whether or not the channel is to be a lined channel for higher velocities, a grassed channel, or a natural channel already existing.

The actual choice must be made upon a variety of multidisciplinary factors and complex consideration which include, among others:



$Q_s * D_{50} \cong Q * S$

Figure 8. Schematic of the Lane relationship for qualitative analysis.

$Q_A^5 * D_M^5 = Q_M D_A^4$

EXAMPLES:

GENERAL SEDIMENT TRANSPORT EFFECTS

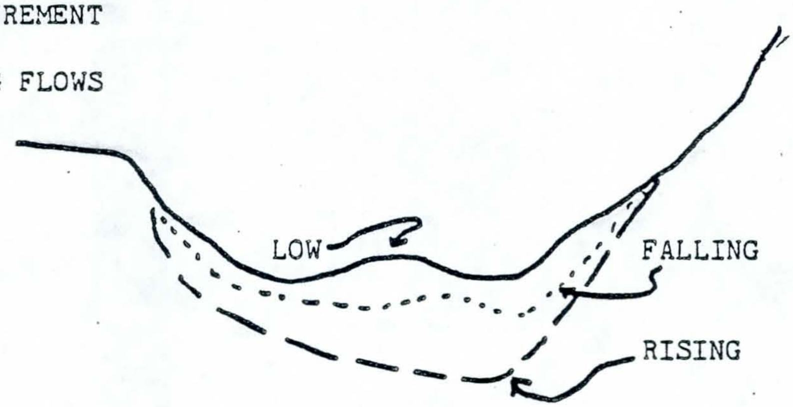
-DEVELOPING WATER SURFACE ELEVATIONS

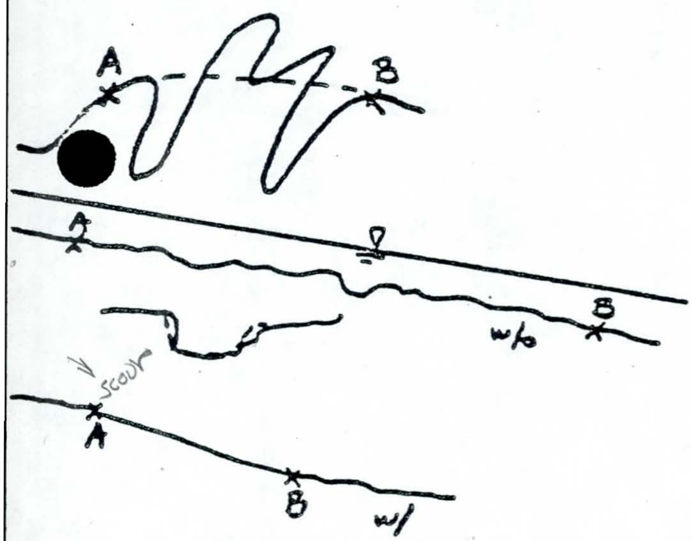
USUALLY RIGID BOUNDARY ASSUMPTION

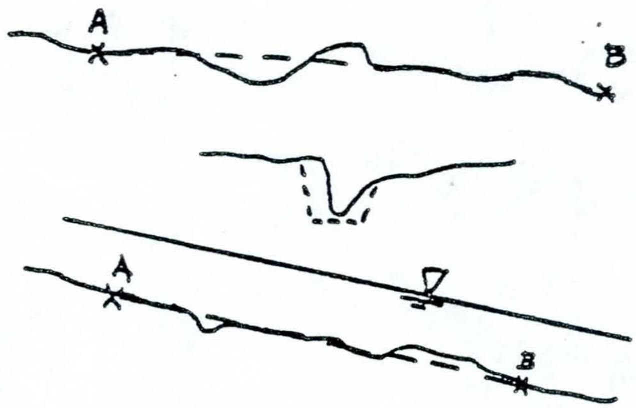
BED CHANGES DURING THE YEAR

TIMES OF MEASUREMENT

RISING/FALLING FLOWS







-CHANNELIZATION

SHORTEN, DEEPEN, WIDEN CHANNEL

STREAM WILL RESEEK EQUILIBRIUM

TRIBUTARY EFFECTS

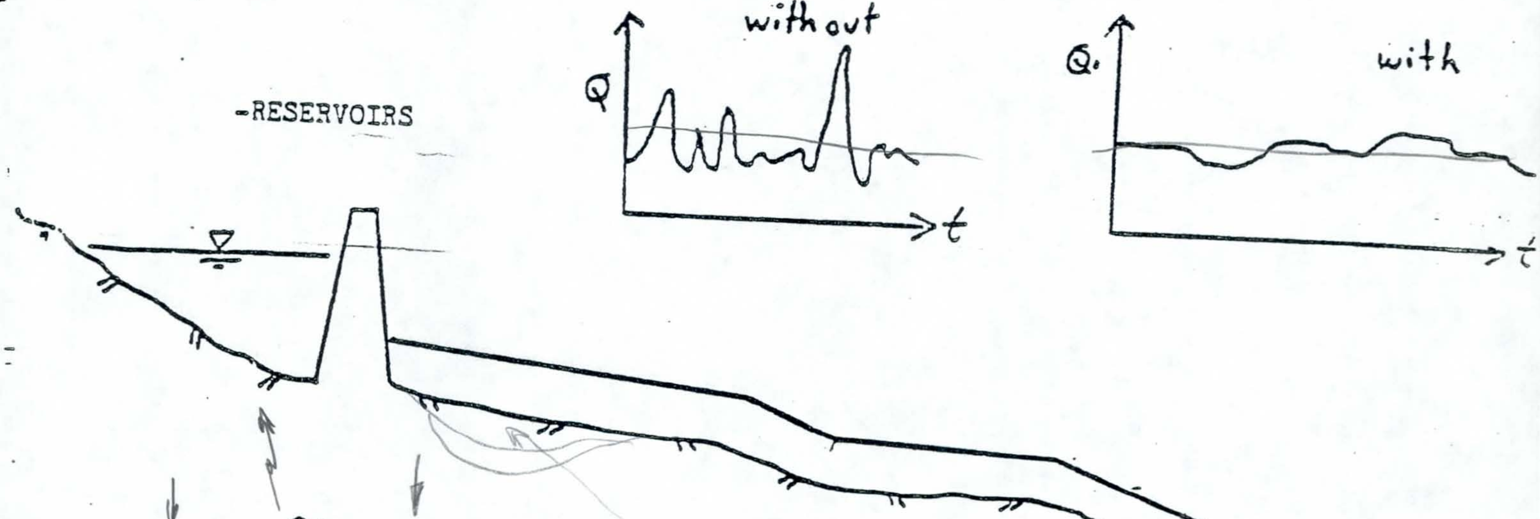
-CHANNELIZATION CAN CAUSE:

Headcut UPSTREAM

Deposition IN IMPROVED REACH AND DOWNSTREAM

Figure 9.

EXAMPLES (continued)



$Q_s * D_{50} \simeq Q * S_f$

A RESERVOIR CAN CAUSE:

$Q_s * D_{50} = Q * S_f$

* _______________ IN POOL

* _______________ DOWNSTREAM

*HOW FAR DOWNSTREAM?

CHANGED FLOW REGIME CAUSES:

Lane's Relationship

*INCREASED TRANSPORT

*DECREASED TRANSPORT

*BOTH

*TRIBUTARY EFFECTS

USUAL EFFECTS: IMMEDIATE DEGRADATION DOWNSTREAM, THEN DEPOSITION IN LOWER REACHES

Figure 10.

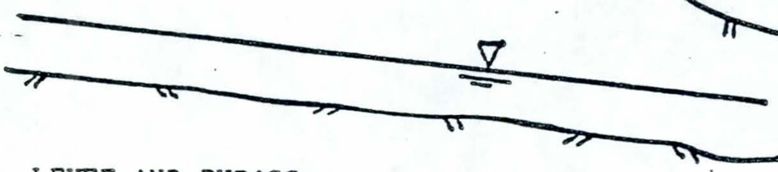
EXAMPLES (continued).

-A LEVEE CAN CAUSE:

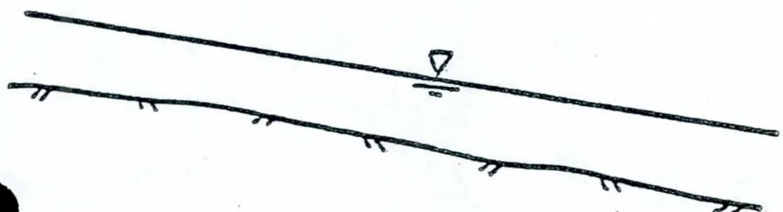
____________ UPSTREAM

____________ IN LEVEED REACH

____________ DOWNSTREAM



-LEVEE AND BYPASS



DIVERT FLOW THAT WOULD HAVE GONE INTO OVERBANK STORAGE

DIFFICULT TO PREDICT SCOUR-DEPOSITION

-BYPASS

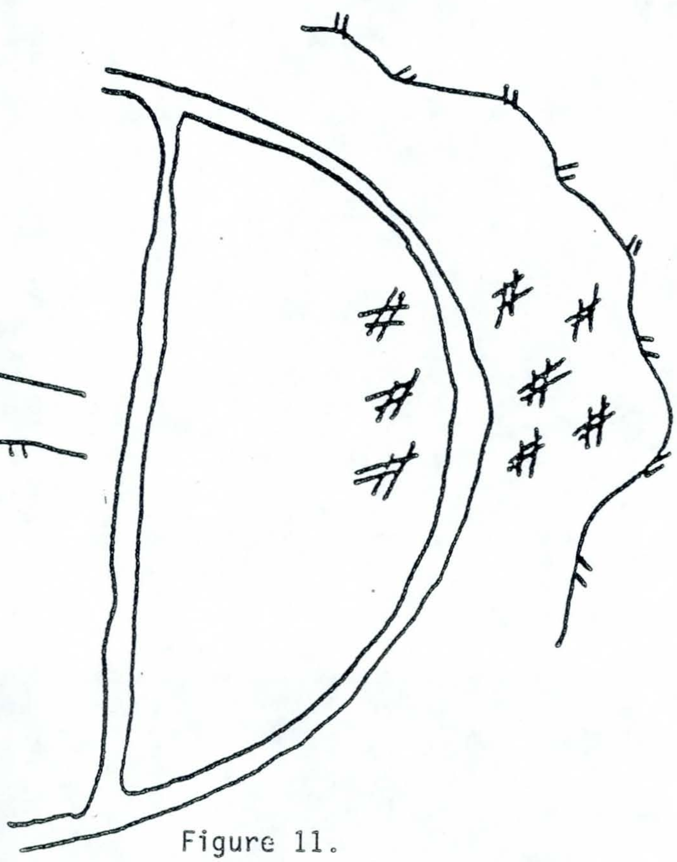


Figure 11.

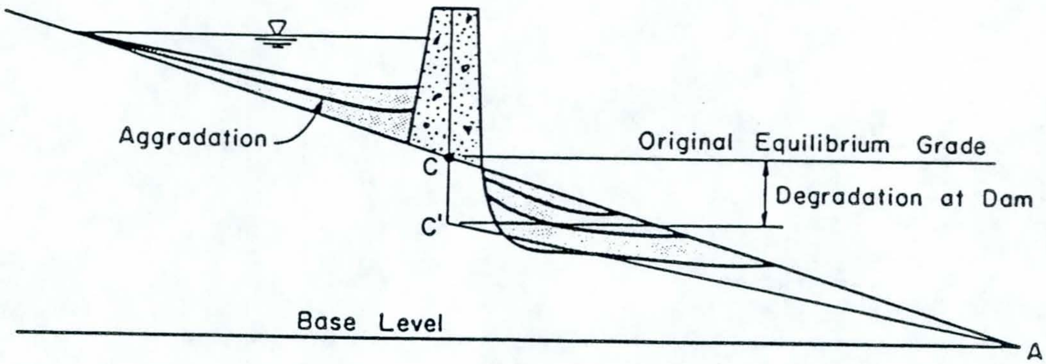


Figure 12a. Channel response above and below a dam.

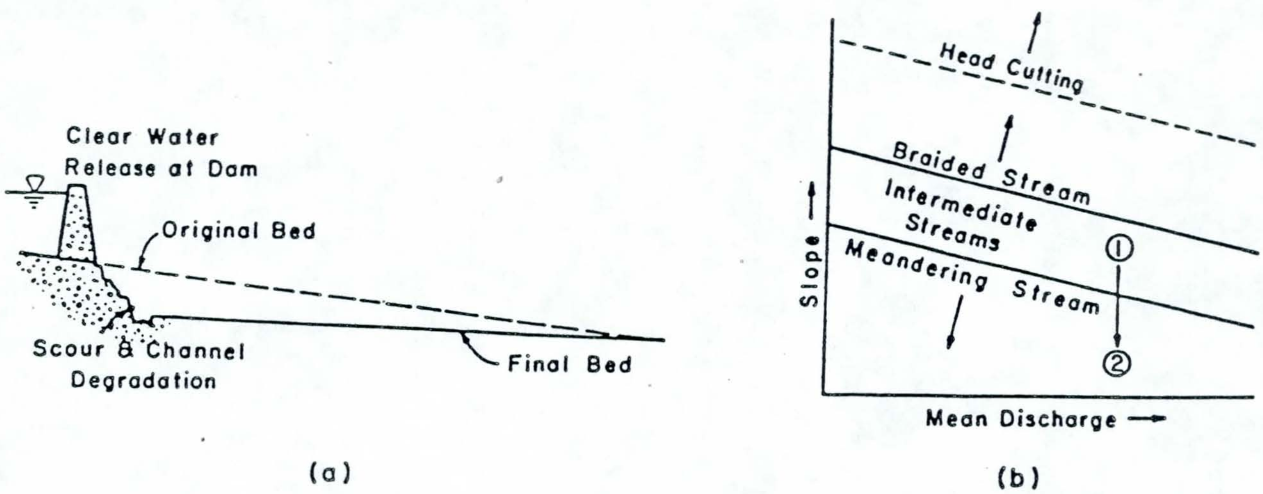


Figure 12b. Clear water release below a dam.

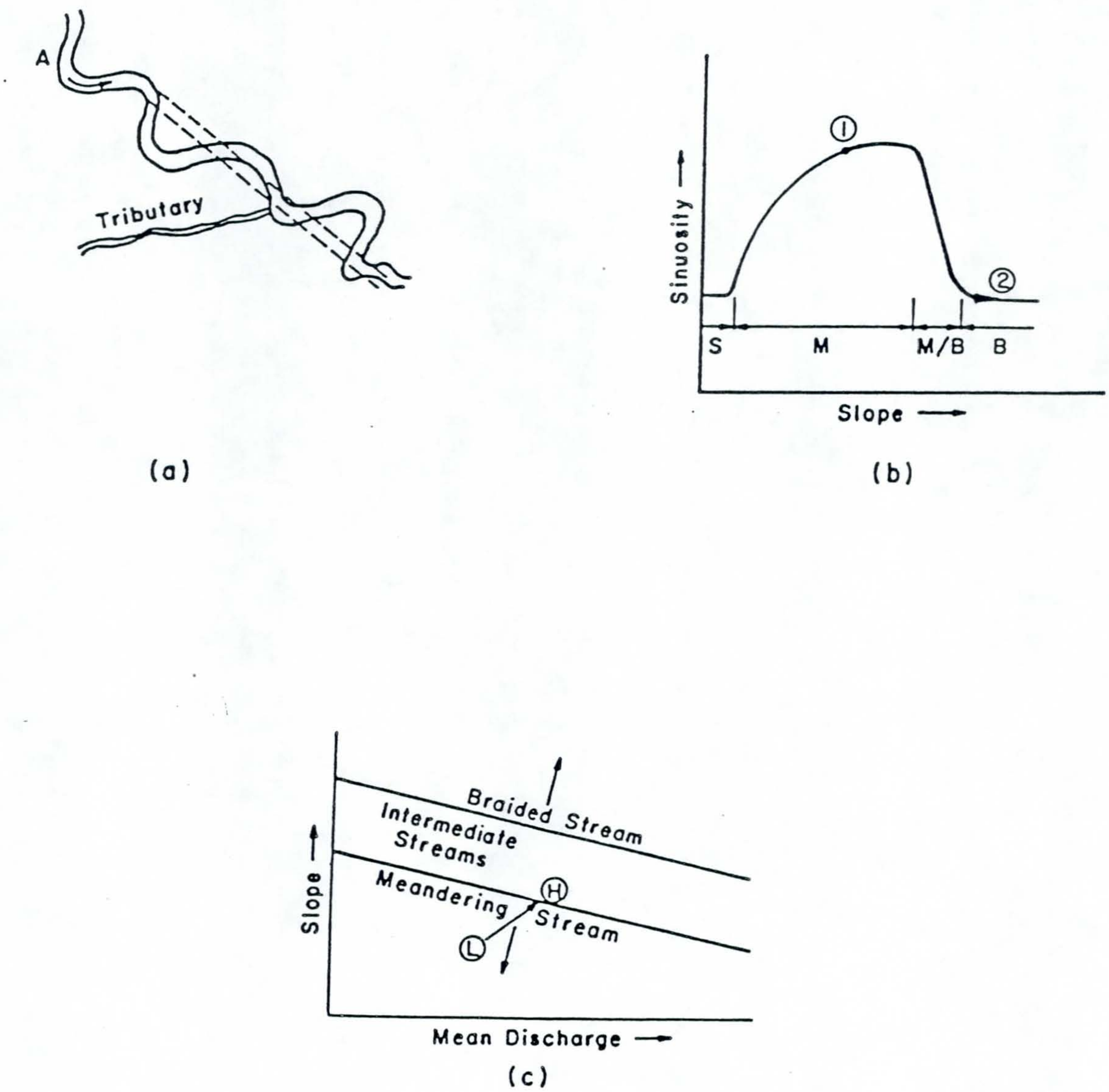


Figure 13. Straightening of a reach by cutoffs.

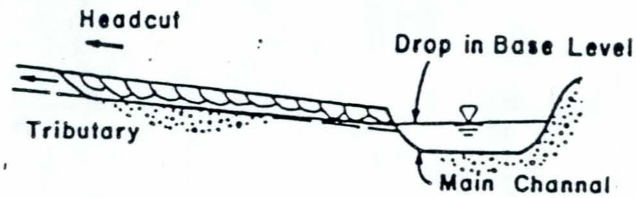


Figure 14. Lowering of base level for tributary stream.

Hydraulic

Slope of thalweg
Right of way
Capacity needed
Basin sediment yield
Topography
Ability to drain adjacent lands

Structural

Costs
Availability of material
Areas for wasting fill

Environmental

Neighborhood character
Neighborhood asthetic requirements
Need for new green areas
Street and traffic pattern
Municipal and county policies

Sociological

Neighborhood social patterns
Neighborhood children population
Pedestrian traffic
Recreational needs

Prior to choosing the channel type, the planner should be sure to consult with experts in related fields in order that the channel chose will create the greatest overall benefits. Whenever practical, the channel should have slow-flow characteristics, be wide and shallow, and be natural in its appearance and functioning.

B. Summary of Design Procedure

1. Estimate the design flow based on hydrologic computations.

2. Determine the channel design slopes based on topographic considerations. The channel design slope should be the uniform slope required to allow the channel to be constructed through slight changes in existing grade.

3. Determine a channel bottom width (approximate prismatic channel shape) based on field observation of the existing channel, flow capacity requirements, engineering judgment, and consideration of sediment-transport-related

issues.

4. Estimate or look up in published tables, an appropriate Manning's n value.

5. Determine the hydraulic regime for the design flow by computing y_c and y_n.

6. Determine channel stability based on an estimate of the bed material size and material type using a critical shear or critical velocity approach.

7. Determine the type of channel bank or toe protection necessary to maintain channel stability (e.g., grass lined, riprap, gabions, soil cement, drop structures, etc.).

8. Assess river bends, channel crossings, local scour, superelevations, wave action, and freeboard problems separately.

9. Recompute the channel hydraulics based on final alignment, bank protection methods, and other design considerations to check the final channel capacity, velocity, and freeboard.

10. Make sure your design will function equally well for low flows and high flows.

WORKSHOP 1

QUALITATIVE ANALYSIS

LECTURE 4

GEORGE K. COTTON, P.E.

Simons, Li & Associates, Inc.
1225 East Broadway Road, Suite 200
Tempe, Arizona 85282

July 1986

LECTURE 4: Workshop 1, Qualitative Analysis

OBJECTIVE: To demonstrate the practical use of basic qualitative analysis tools for plan form and longitudinal profile analysis.

I. GENERAL INTRODUCTION

This workshop consists of a series of exercises that will acquaint the user with various data used in fluvial analysis and methods of qualitative analysis. Reduction of plan form data to determine eroded areas and channel classification variables is demonstrated. Interpretation of bed and bank material samples is presented. Sample calculations using channel classification schemes, basic fluvial geomorphic relationships, and the Lane relationship are conducted.

II. USE OF AERIAL PHOTOGRAPHY

A. Identification of changes in river alignment

- Describe major channel features of the Santa Cruz river in 1941.

- Describe major channel features of the Santa Cruz river in 1985.

- What changed?

- What stayed the same?

- There are a number of instances of property damage and structural failures in the 1985 photo, locate as many as you can find. Can you guess what river processes may have caused the failures?

B. Overlay of historic river alignments

- Delineate the right and left bank for the 1941 and 1985 Santa Cruz aerial photos.

- What are some of the changes that can be found using the photo overlay that are less obvious without the overlay?

- How much judgment did you have to use when developing your overlay?

  - What areas were you uncertain about including?

  - Do you think photo quality or scale is a problem?

- Could you quantify the change in alignment?

III. PLAN FORM VARIABLES

A. Sinuosity

- Ratio of river length to valley length
- Measurement of sinuosity

B. Channel width and depth

- Measurement of channel shape
- Width to depth ratio

IV. BED AND BANK MATERIAL INFORMATION

A. Size

- What are the median particle sizes?
- What are the largest sizes?
  - Are there gravel and cobbles
- What is the smallest size?
  - Are there silt and clay?
- What sizes are present in the banks that are not found in the bed and visa versa?

B. Gradation

- How are the sediment sizes distributed?
- Is the distribution symetric?
- Calculate the gradation coefficient.

C. Classify the bed and bank material

V. CHANNEL CLASSIFICATION

A. Apply Culbertson's and Brice's classification scheme to the Santa Cruz river for the 1941 and 1985 conditions.

B. Note the changes and discuss channel stability.

C. Use slope-discharge relationships to classify the channel. Has channel form changed over time?

VI. LONGITUDINAL PROFILE RESPONSE

A. Measure channel gradient for years of available record.

B. Based on plan form analysis designate a stable upstream reach.

C. Tabulate data on channel width, slope, and mean sediment size.

D. Use the Lane relationship to qualitatively predict relative changes in channel profile stability.

QUANTITATIVE SEDIMENT ANALYSES

(Taken from Chapter VII, "Sediment Transport" in
<u>Engineering Analysis of Fluvial Systems,</u>
<u>by Simons, Li & Associates, Inc., 1982)</u>

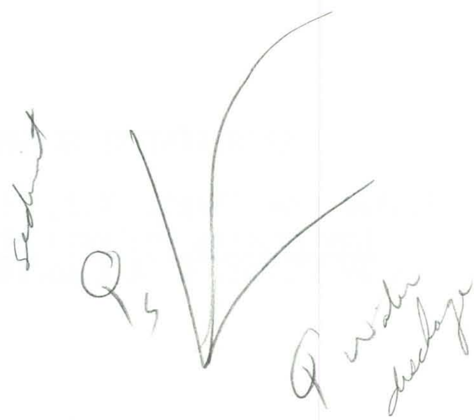
LECTURE 5

ROBERT C. MACARTHUR, Ph.D., P.E.

chapter 5
text book

Simons, Li & Associates, Inc.
3555 Stanford Road
P.O. Box 1816
Fort Collins, Colorado 80522

July 1986



Geometric Data

Sediment Data / Bed

Flow Data

Do Loop

QUANTITATIVE SEDIMENT ANALYSES

Lecture 5

Robert C. MacArthur, Ph.D., P.E.

AUTO = AREA + FORM DRAG RESISTANCE

Lecture Outline

Introduction
Terminology
Sediment-Transport Mechanisms
Factors Affecting Sediment Transport and Deposition
Incipient Motion and the Shield's Relation
Sediment-Transport Equations

Bed Load

Meyer-Peter, Muller Equation
Einstein's Bed-Load Equation
Comparison of Various Bed-Load Equations

Suspended Load
Total Load
Power Relationships
Comparison of Bed-Material Load Equations and Field Applications

Example Bed-Material Discharge Calculation Using Meyer-Peter, Muller Methods

How Sediment-Transport Equations are Used in Numerical Models

Data Requirements
Theoretical Basis
Solution Techniques

Shields Diagram

HOW SEDIMENT-TRANSPORT EQUATIONS ARE USED IN NUMERICAL MODELS

ROBERT C. MACARTHUR, Ph.D., P.E.

July 1986

How Sediment-Transport Equations are Used In Numerical Models

1. Fixed bed versus movable bed models

2. Sediment sources in a river reach

Input Data Requirements for Typical Sediment-Transport Model

1. Geometric data

   A. Cross sections
   B. Distance between cross sections
   C. N-values
   D. Limits of movable bed
   E. Special features

2. Sediment data

   A. Range of particle sizes
   B. Inflowing sediment load
   C. Size of sediment material in stream bed
   D. Tributaries or diversions

3. Hydrologic data

   A. The water discharge hydrograph
   B. The downstream boundary condition
   C. Water temperature

Theoretical Basis

$$S_f = S_o - \frac{\partial y}{\partial x} - \frac{\partial (V^2/2g)}{\partial x}$$ Conservation of Energy (1)

$$Q = VA$$ Continuity of Water Volume (2)

$$\frac{\partial G}{\partial x} + B_o \frac{\partial y_s}{\partial t} = 0$$ Continuity of Sediment Volume (3)

$G = f$ (Q, y, grain size distribution, etc.) Transport Function (4)

where the transport function may be

   A. Toffaleti
   B. Madden modification of Laursen
   C. Yang - stream power
   D. Einstein bed-load function
   E. User supplied

Solution Technique

For a given "event" in the histogram of flows (i.e., a water discharge, Q, associated with a duration, WT)

1. Calculate the water-surface profile by solving equations (1) and (2) using step backwater procedure.

2. Calculate sediment-transport capacity at each section using equation (4).

3. Calculate volume of material scoured and deposited between cross sections from Equation (3).

4. Calculate associated change in bed-surface elevation (Wy) and modify cross-sectional geometry appropriately.

5. Read data for next event.

6. Go to 1.

Model Limitations

1. One-dimensional approximation

2. Quasi-steady

3. Sensitivity to inflowing load data

4. Lack of objective measures of model performance (calibration data)

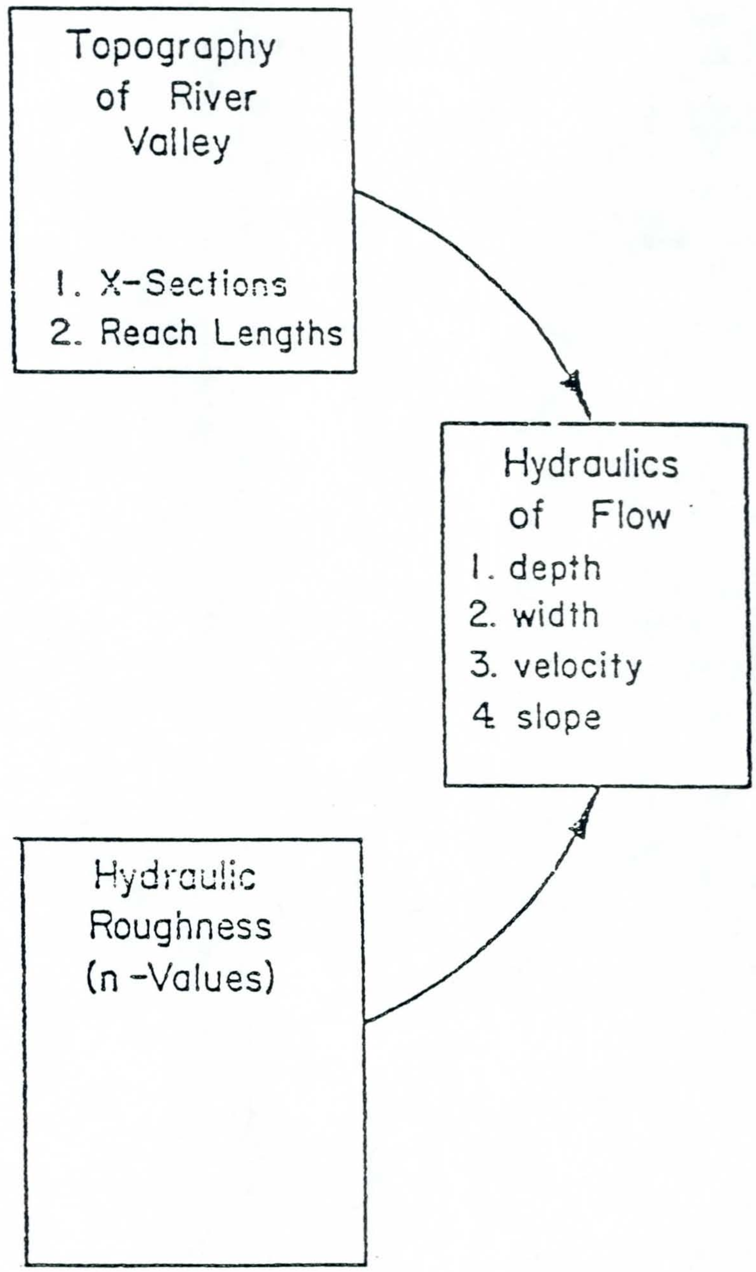


Fig. 6.02. FIXED BED MODEL

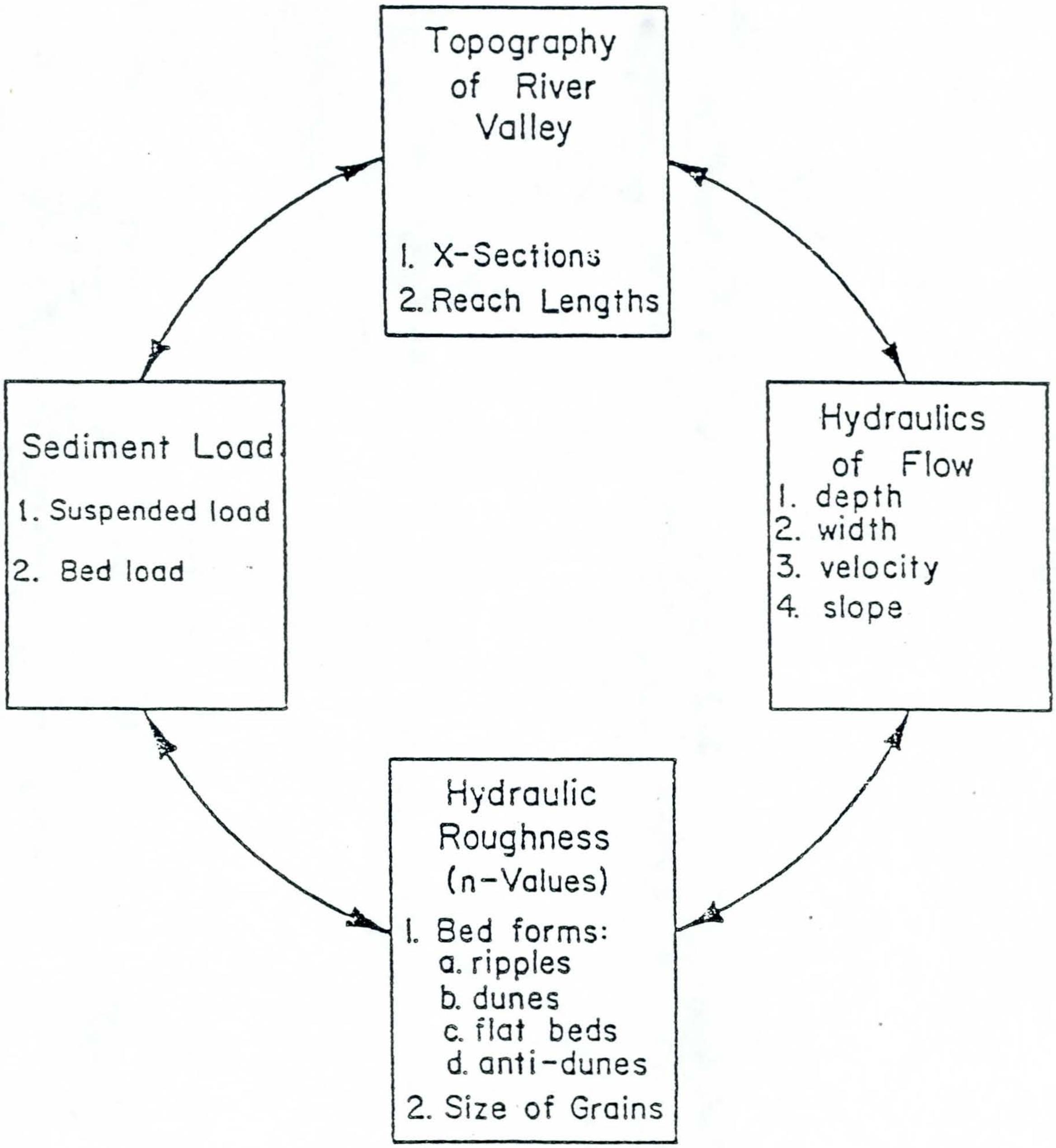
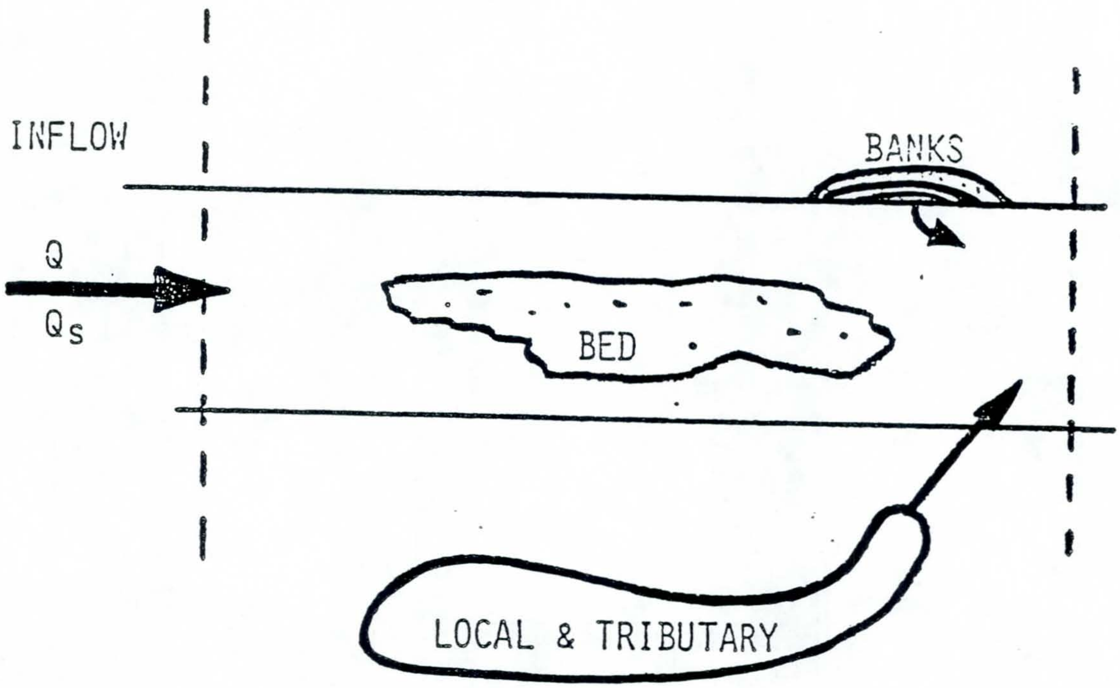


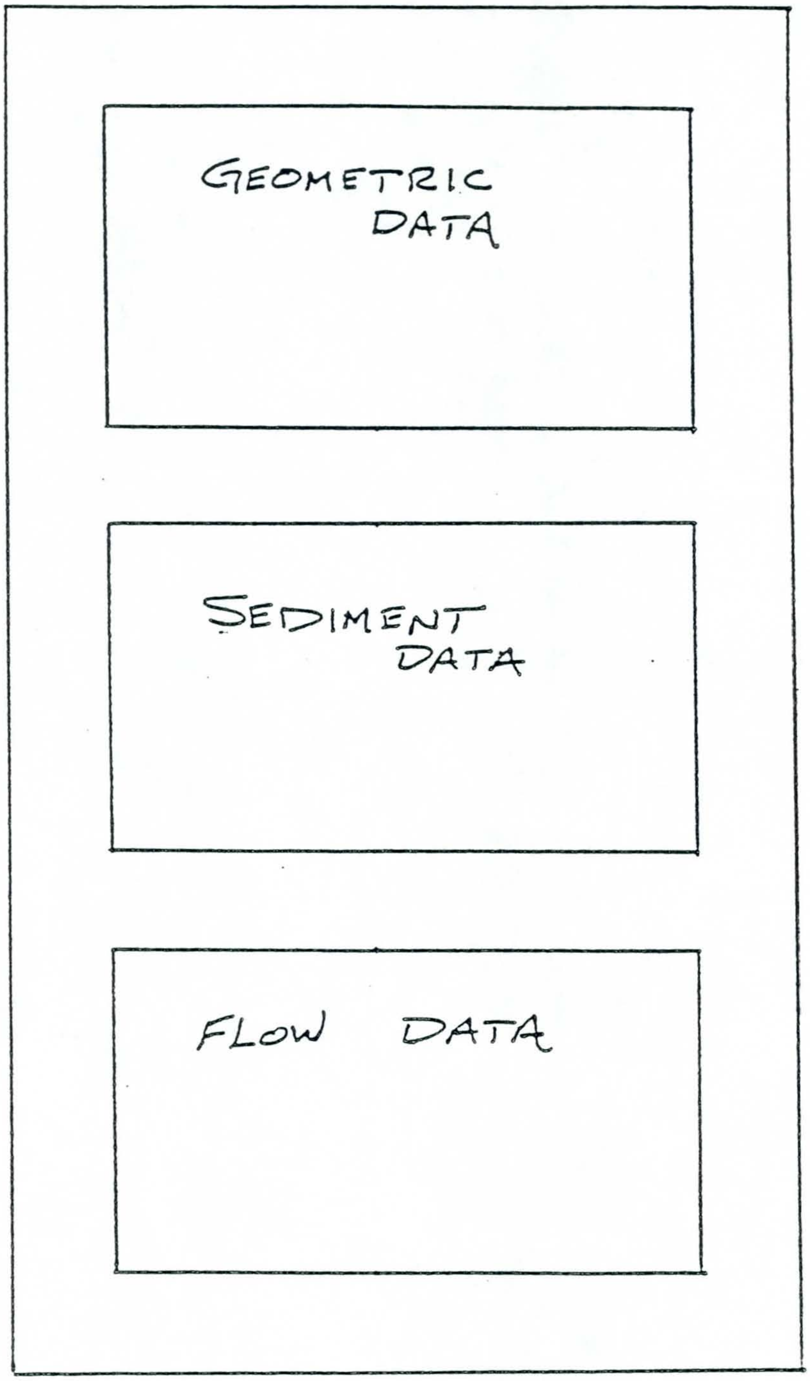
Fig. 6.03. MOVABLE BED MODEL

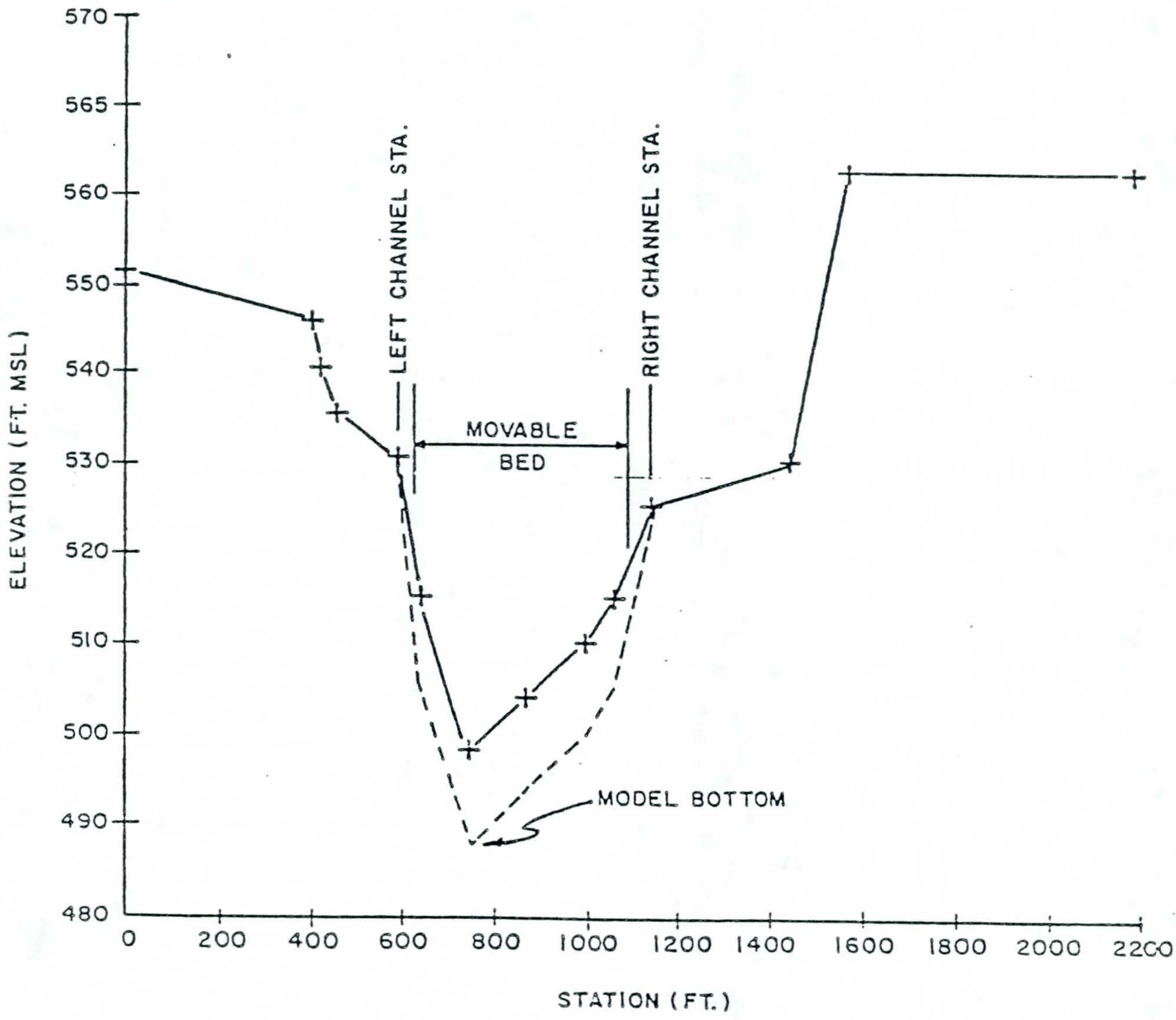


SEDIMENT SOURCES IN A RIVER REACH

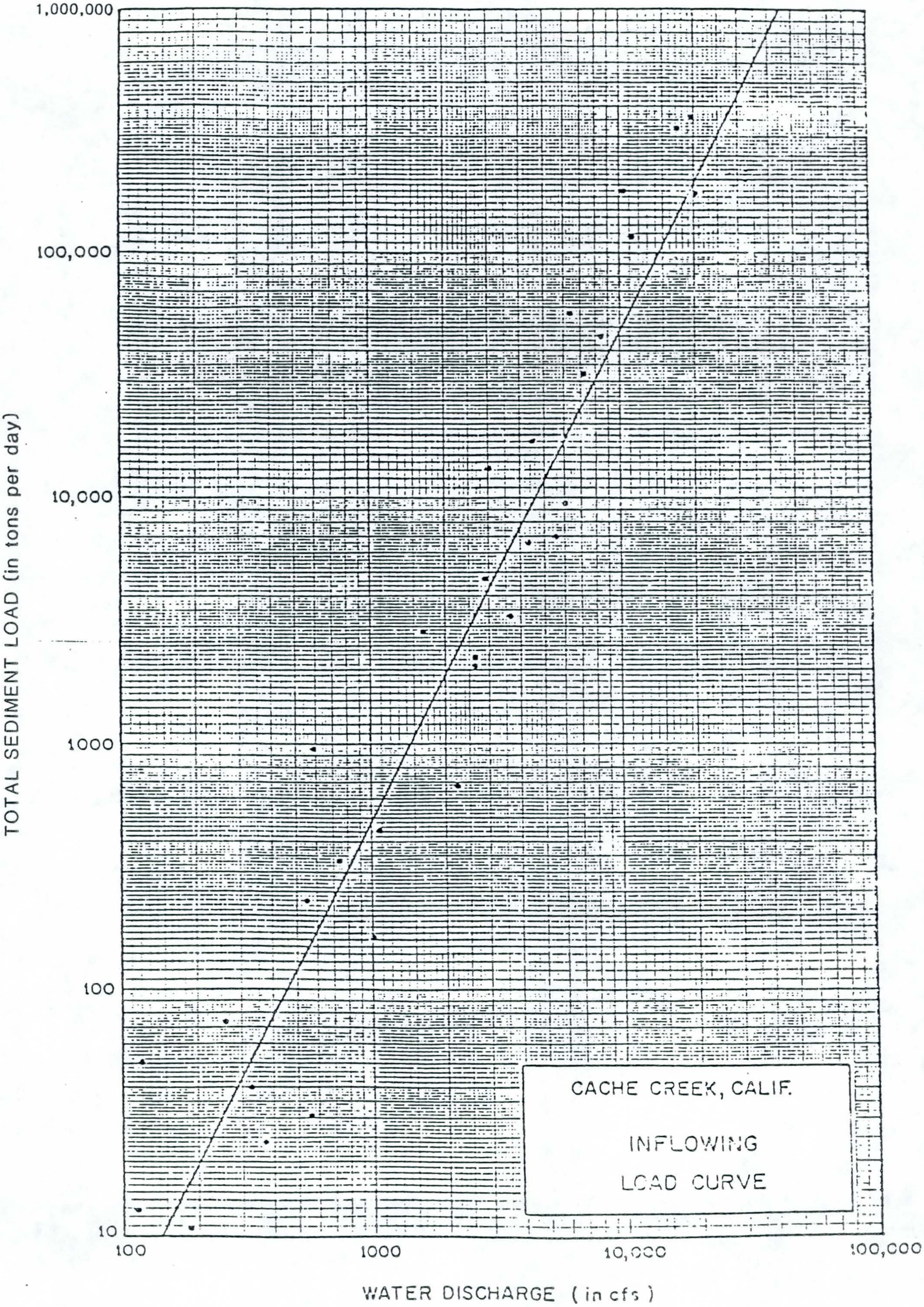
TYPICAL

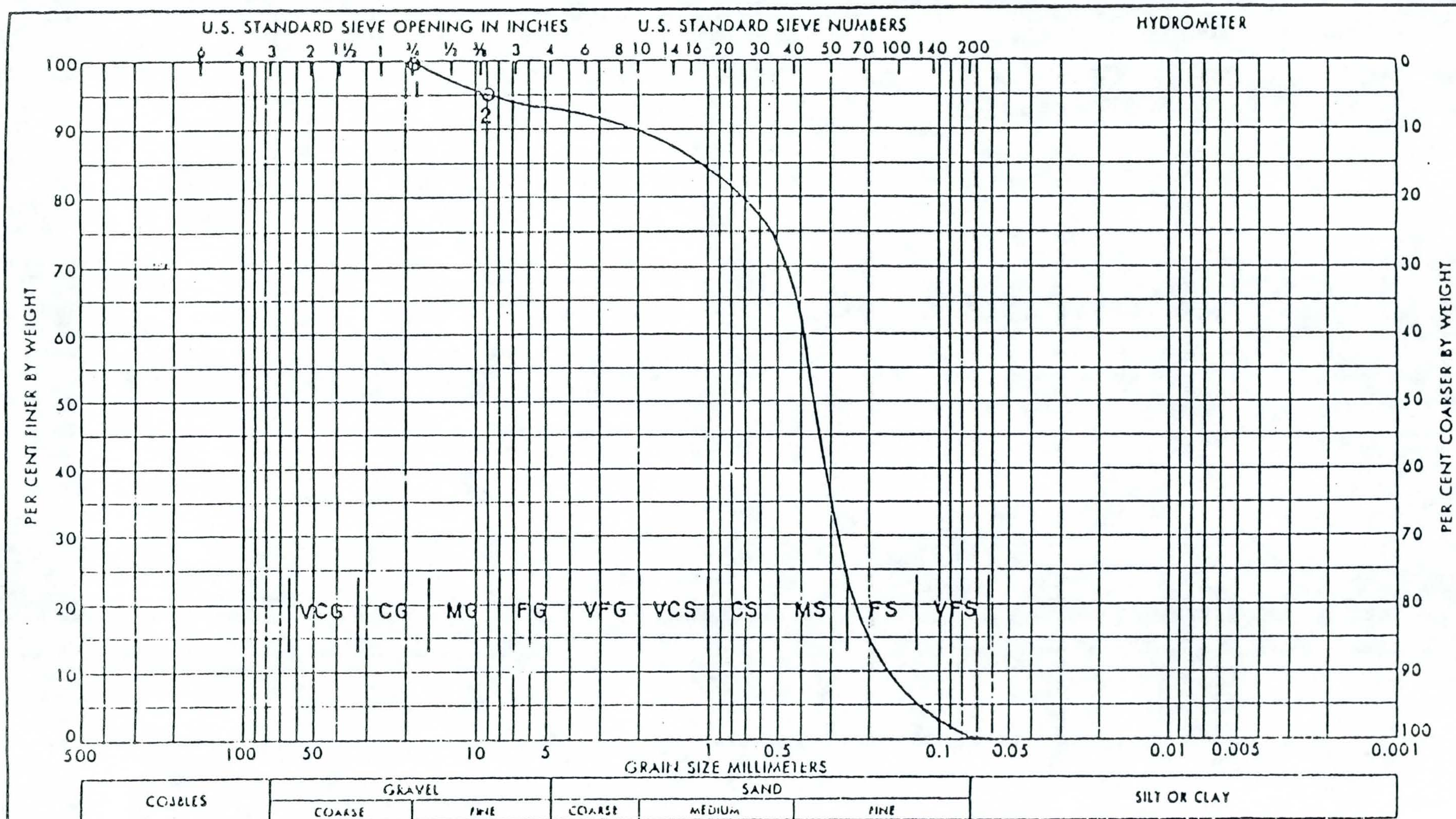
INPUT DATA FILE STRUCTURE





Example Cross-Section


1,000,000
100,000
10,000
1000
100
10
TOTAL SEDIMENT LOAD (in tons per day)
100
1000
10,000
100,000
WATER DISCHARGE (in cfs)
CACHE CREEK, CALIF.
INFLOWING
LOAD CURVE



SAMPLE NO	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI	PROJECT
							AREA
							BORING NO.
GRADATION CURVES							DATE

ENG FORM 2087
1 MAY 63

REPLACES WES FORM NO. 1241, SEP 1962, WHICH IS OBSOLETE.

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<table>
<tr><td colspan="2"></td><td colspan="2">Classification System</td></tr>
<tr><td colspan="2"></td><td>Based on mechanism of transport</td><td>Based on particle size</td></tr>
<tr><td rowspan="3">Total sediment load</td><td>Wash load</td><td rowspan="2">Suspended load</td><td>Wash load</td></tr>
<tr><td>Suspended bed-material load</td><td rowspan="2">Bed-material load</td></tr>
<tr><td>Bed load</td><td>Bed load</td></tr>
</table>

Figure 8. Comparison of location of suspended load, wash load, and bed load carried in channel flow from USDA-SCS, NEH, Section 3 (1971).

TOTAL LOAD

MODE OF TRANSPORT	AVAILABILITY IN STREAMBED	METHOD OF QUANTIFYING
SUSPENDED + BED LOAD TOTAL LOAD	WASH LOAD + BED MATERIAL LOAD TOTAL LOAD	MEASURED LOAD + UNMEASURED LOAD TOTAL LOAD

AN OVERVIEW OF SEDIMENT-TRANSPORT PHENOMENA BASED UPON CASE HISTORIES

(Slide Presentation)

LECTURE 6

MICHAEL E. ZELLER, P.E.

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July 1986

LECTURE 6: An Overview of Sediment-Transport Phenomena Based Upon Arizona Case Histories (Slide Presentation)

OBJECTIVE: This lecture, through the use of 35 mm slides, is intended to provide visual documentation of the manner in which sediment-transport phenomena has, over the past five to ten years, impacted fluvial systems located within the State of Arizona.

I. GENERAL INTRODUCTION

Beginning with the October, 1977 flood in Southeastern Arizona, which primarily affected the Santa Cruz River, and ending with the devastating flood of October, 1983, which impacted all of Southeastern Arizona, no less than five major flood events (e.g., 1977, 1978, 1979, 1980, and 1983) have wrecked havoc upon some of the major watershed and river systems in Arizona. The 35 mm slides selected for showing during this lecture were chosen so as to offer a visual overview of the major classes of sediment-transport phenomena which were observed to have occurred during the recent floods noted herein. These major classes are:

A. Lateral Migration (Bank Erosion)

- Meandering
- Widening

B. Scour (Bed Erosion)

- Local
- General

C. Fill (Sedimentation)

- Braiding
- Avulsion

QUANTITATIVE ENGINEERING ANALYSIS

BED ADJUSTMENT MECHANISMS

LECTURE 7

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1225 East Broadway Road, Suite 200
Tempe, Arizona 85282

July 1986

LECTURE 7: Quantitative Engineering Analysis, Bed Adjustment Mechanisms

OBJECTIVE: The purpose of this lecture will be to identify and examine the various physical processes that are capable of causing vertical movement in the profile of a river-bed.

I. GENERAL INTRODUCTION

The vertical movement that occurs in a river or channel bed during passage of a flood can often create more severe or destructive impacts than just the inundation caused by the floodwaters. Failure to acknowledge and plan for such bed adjustments can lead to catastrophic failures of bridges, highways, or other man-made structures that might be located within the floodplain environment.

There is a continual, dynamic interaction between sediment particles and the transporting medium, water. As water moves sediment through a drainage system, there is a constant struggle to achieve a state of equilibrium or balance between sediment supply and sediment transport capacity. In seeking this balance, the drainage system is in a continual mode of change as both vertical and horizontal adjustments are made to the channel boundaries of the system's watercourses. In natural, undisturbed watersheds, these changes may take place very slowly (hundreds or thousands of years), but when man-made urbanization disrupts such watersheds, large magnitude changes can occur very rapidly.

The vertical changes that occur to the cross-sectional geometry of a river-bed are a complex interaction of several different phenomena. As suggested previously, some of these phenomena occur over long periods of time, while others must be dealt with during every flood. There are at least six known mechanisms that are responsible for causing vertical movement in a river-bed:

1. long-term degradation/aggradation
2. general scour
3. local scour
4. bend scour
5. low-flow incisement
6. influence of bedforms

equilibrium slope analysis

Depending on site specific parameters such as bed-material gradation, channel shape, hydraulic controls (dams, grade controls, etc.), and degree of urbanization or floodplain encroachment, some of these phenomena may be more prominent at one site versus another.

By isolating these phenomena and examining the physical processes responsible for their behavior, it is possible to predict their occurrence and estimate the magnitude of their response within a given manifested in terms of bed slope change

river system.

The remainder of this lecture will focus on an examination of these phenomena and present analytical methods that can be used to quantify their influence on a specific project.

II. LONG-TERM AGGRADATION/DEGRADATION

Long-term aggradation/degradation is normally a response to a disruption within a watershed that either changes the sediment transport capacity of the river system (through alteration of the channel geometry or slope) or changes the sediment supply to the river (such as that occurring downstream of a dam). As the name implies, changes resulting from such watershed disruptions generally occur over a long period of time and are not the result of a single flood. The time period is strongly influenced by the watershed's climatology and hydrology, since runoff must occur in the drainage system for the equilibrium slope changes to take place.

In order to understand and be able to apply the equilibrium slope concept to practical problems, certain principles and constraints must be examined.

A. Sediment Continuity

The concept of equilibrium slope is based on the principle of sediment continuity. Under equilibrim conditions, the sediment continuity equation can be expressed as follows:

$$Q_{s_{in}} = Q_{s_{out}}$$

where $Q_{s_{in}}$ = rate at which sediment is being delivered to a specific reach of river

$Q_{s_{out}}$ = sediment transport capacity of a specific river reach under investigation

For equilibrium to prevail, both terms in this equation must be equal. If they are not, natural adjustments will begin to occur in the bed slope so as to either increase or decrease $Q_{s_{out}}$ to make it equal to $Q_{s_{in}}$.

If the incoming sediment supply exceeds the river's sediment transport capacity, the bed-slope will increase or steepen, which in turn increases the velocity of flow, thus causing an increase in sediment transport capacity, since transport capacity is roughly proportional to the fourth power of the velocity of flow.

Alternately, if the incoming sediment supply is decreased to less than that capable of being transported by the river, the bed-slope will begin to decrease or flatten in order to reduce the velocity of flow and, thus, the sediment transport capacity. Figure 1 presents a graphical illustration of these two cases.

B. Factors to be Evaluated in Determining Equilibrium Slope

An equilibrium slope analysis requires an investigation of many watershed characteristics. The analysis should include an investigation of the watershed history, channel hydraulic and sediment transport charcteristics, and the identification of any natural or man-made controls in the river system. The following list presents an orderly sequence of issues that should be considered during an equilibrium slope analysis.

1. Examine historic bed profiles

   - Determine reasons for observed historic changes

     * Land use change
     * Natural watershed catastrophe
     * Dams

2. Examine historic flood records

   - Correlate with changes in historic bed-profiles

3. Determine dominant channel discharge

   - perennial channels, 2- to 5-year event

   - ephemeral channels, 5- to 10-year event

   - bankfull discharge may be good indicator of dominant flow

$\boxed{Q = 8500 \text{ cfs}}$

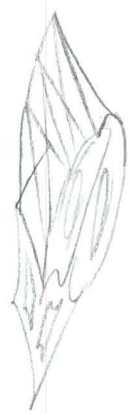
$d = 3.8'$

$V = 5.3 \text{ fps}$

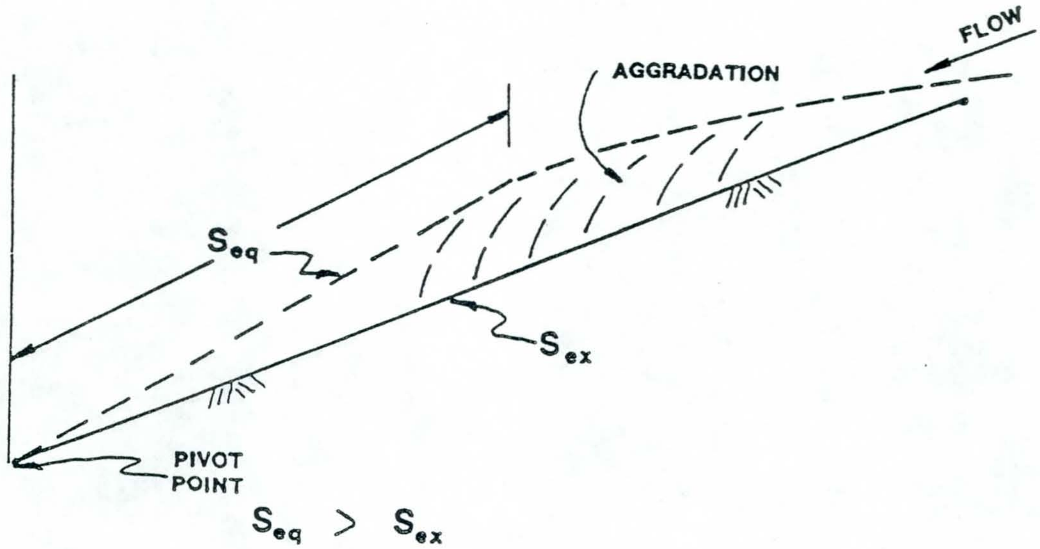
$T = 200$

$q_s = 4.56 \text{ cfs}$

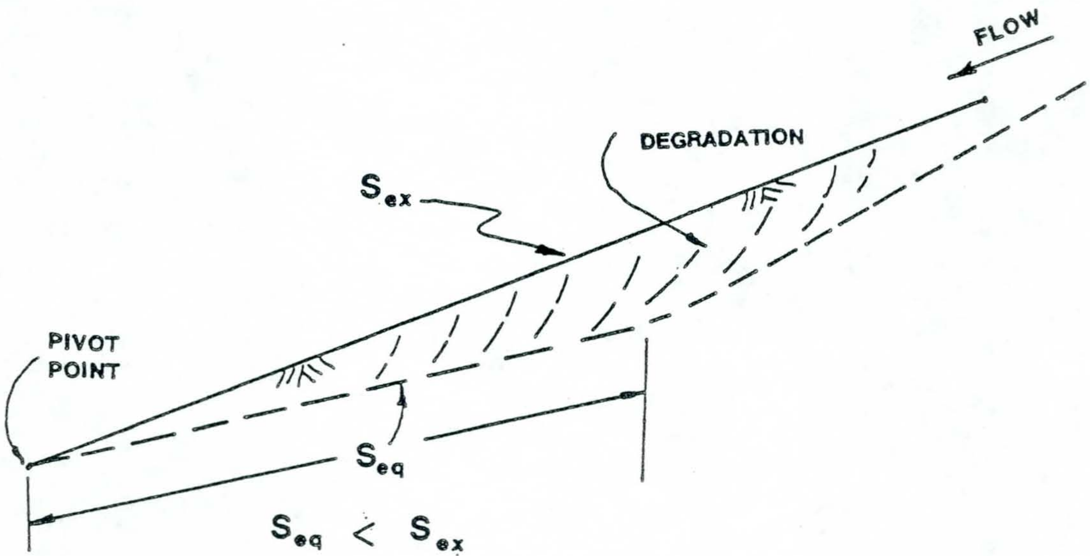
$Q_s = q_s T$



Sediment Transport Capacity $= V^{4-5}$



In this case, the sediment supply exceeds the sediment transport capacity of the reach. Under this condition, the bed slope must increase in order to increase the transport rate to match the supply rate. The initial excess of sediment supply will cause aggradation at the upstream end of the reach until the downstream portion of the bed slope is steep enough to transport all the incoming sediment.



In this case, the incoming sediment supply is less than the sediment transport capacity of the reach. This sediment deficit will be satisfied by a removal of bed material through the reach until the bed slope is flattened enough to reduce the transport capacity to the point that it matches the incoming sediment supply.

Figure 1 - Schematic of long-term degradation/aggradation

4. Sediment supply

   - Locate undisturbed upstream supply reach that exhibits stability, i.e., no recent evidence of large scale erosion. or aggradation Historic photo & profiles are a big help

   - Check for no recent changes in upstream watershed conditions and no major tributary inflows downstream of supply reach.

   - Supply will be equal to transport capacity of supply section.

5. Sediment transport capacity

   - Can be computed using applicable sediment transport equations for all downstream reaches

   [Chapter 5]

6. Calculation procedure is usually iterative.

   - Slopes are adjusted until transport capacity equals supply rate.

     . do hydraulic rules first to get depth + velocity
     . use Y + V to compute transport rate.

7. Locate control points for slope pivot.

   - rock outcroppings

   - man-made controls

8. Engineering judgement must be used to interpret results.

   - Some sites may only be evaluated from a qualitative standpoint rather than quantitative.

   - Projection of equilibrium slope (from a pivot point) for long distances upstream will yield unrealistic bed elevations.

III. GENERAL SCOUR/DEPOSITION

General scour occurs in response to a contraction in channel geometry, while general deposition is induced by an expansion of channel geometry. Channel contractions and expansions cause an increase or decrease, respectively, in channel velocity, which creates a similar change in sediment transport capacity.

Such changes in channel geometry create a localized imbalance between sediment supply and sediment transport capacity. When upstream supply exceeds downstream transport capacity, deposition results; the reverse (scour) occurs when downstream transport capacity exceeds upstream supply. The mechanics of this phenomenon are discussed in the following paragraphs.

A. Sediment continuity

General scour/deposition is similar to long-term aggradation/degradation in that it can be analyzed through application of the sediment continuity principle. However, unlike its long-term counterpart, general scour/deposition examines bed-profile changes during short time intervals of a specific flood hydrograph. Although such intervals are too short for equilibrium to be established, they are used to analyze the transitory bed changes that may be encountered simultaneously with, and beyond the point of, achieving long-term equilibrium. By applying this concept to a specific river reach, the bed profile changes can be related to different segments of the hydrograph. Such an analysis may show a given reach of the river to be depositional during one time interval of the hydrograph and scouring during a later interval.

Since equilibrium is generally not established during the short time intervals used in the general scour/deposition analysis, the sediment continuity equation must include a term for the sediment volume changes that occur during each time interval. Mathematically, this is expressed as:

$$Q_{s_{in}} - Q_{s_{out}} = \frac{\Delta Vol}{\Delta Time}$$

$Q_{s_{in}}$ = incoming sediment supply rate

$Q_{s_{out}}$ = sediment transport capacity through a specific channel reach

$\frac{\Delta Vol}{\Delta Time}$ = change in sediment volume that occurs during a given time interval

This is the same equation previously used to define the equilibrium slope concept, with the stipulation that the $\frac{\Delta Vol}{\Delta Time}$ term be zero for equilibrium slope conditions.

Figure 2 graphically illustrates the basic difference between an equilibrium slope calculation and a general scour/deposition calculation. This figure shows that bed elevation changes due to general scour/deposition occur simultaneously with equilibrium slope adjustments and continue, indefinately, beyond the point at which an equilibrium slope is achieved.

B. Procedures for Evaluating General Scour/Deposition

An analysis of general scour/deposition for a project site is normally pursued by a sediment routing procedure which applies the sediment continuity principle to adjacent segments of the river reach under investigation. Basic assumptions of the procedure are outlined as follows:

1. Discretization of flood hydrograph
   The hydrograph for which the analysis will be performed must be subdivided into discrete time intervals. The average discharge for each time interval is used to compute hydraulic and sediment transport properties for each river segment. Figure 3 illustrates a discretized hydrograph.

2. Subdivision of river reach
   The river reach under investigation should be divided into segments of equal length with relatively uniform hydraulic and cross-sectional characteristics. The selection of subdivided reach limits can often be made from a review of a HEC-2 analysis for the site.

3. Sediment supply
   Since the sediment routing procedure proceeds in a downstream direction, the sediment supply to any given subreach is equal to the sediment transport capacity of the adjacent upstream reach. These supply rates will

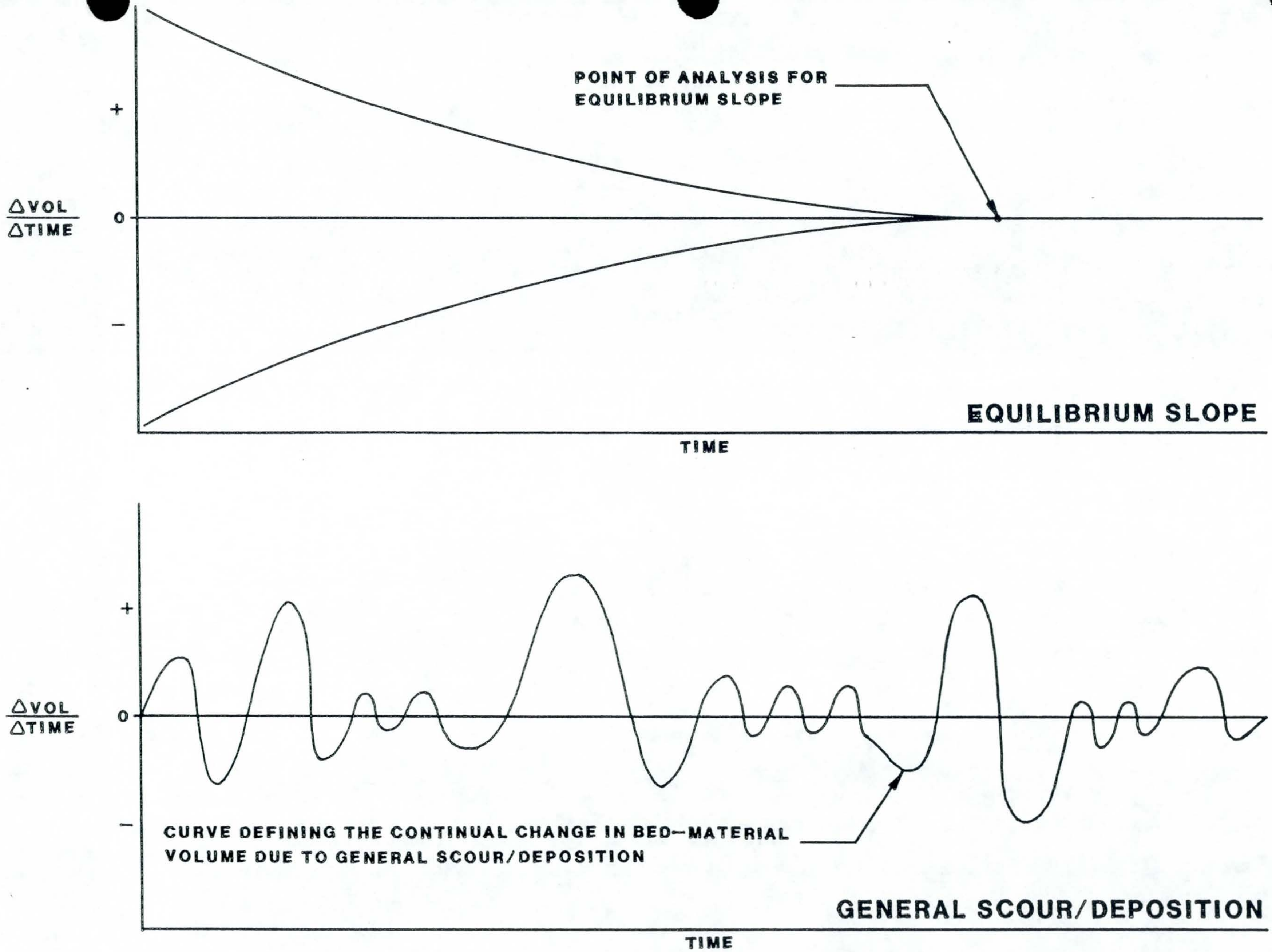


FIGURE 2 COMPARISON OF ΔVOL/ΔTIME FOR EQUILIBRIUM SLOPE & GENERAL SCOUR/DEPOSITION

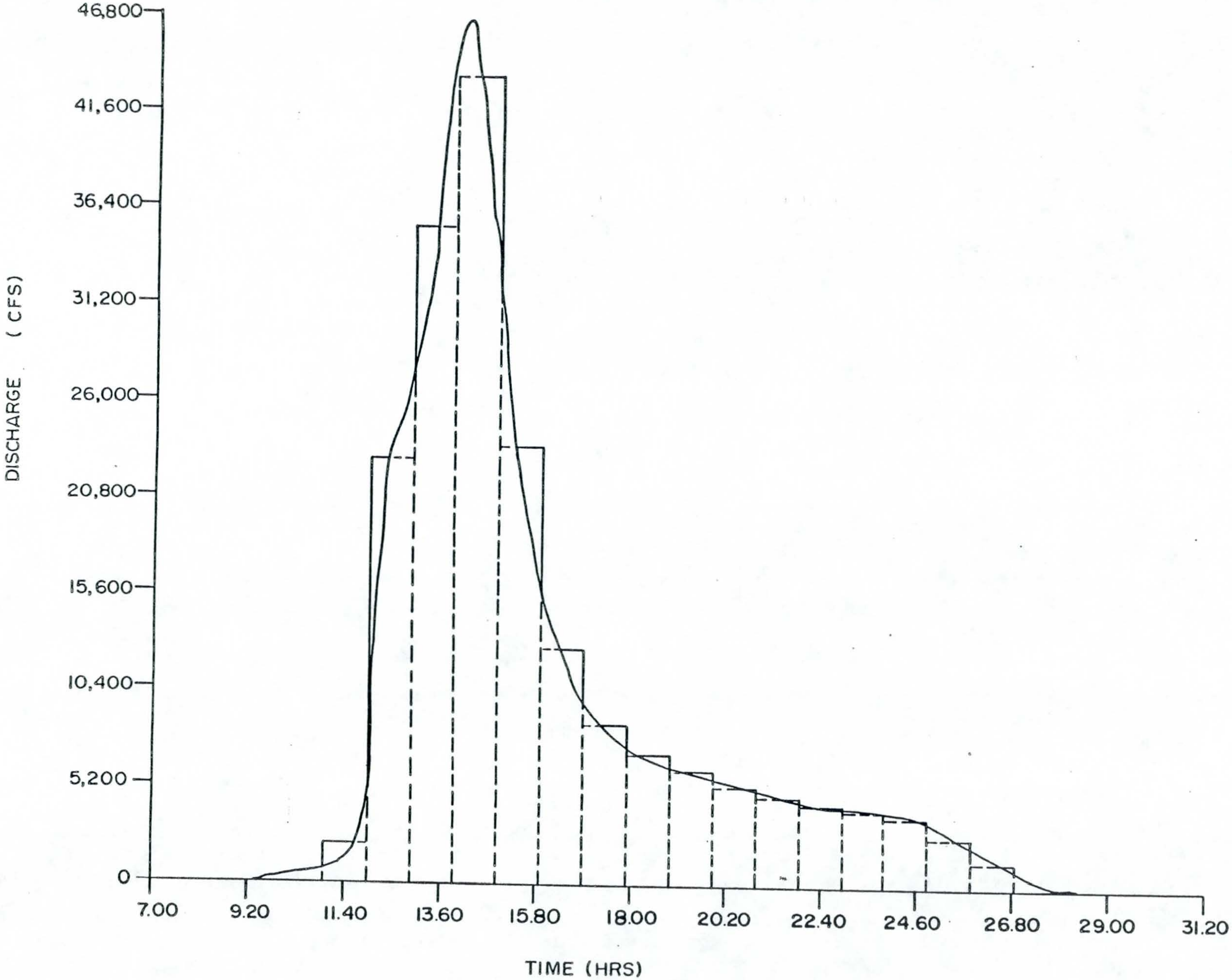


Figure 3 - Discretized 100-year flood hydrograph.

Chapter 5

Power Relationship

unit/weight

must be multiplied by entire reach

$q_s = a \psi^b V^c$

$Q_s = q_s W$

vary with each discretized time interval of the hydrograph. It should be emphasized that this is a major departure from the sediment supply assumption for equilibrium slope. For equilibrium slope, a constant sediment supply rate is used throughout the calculation, since all downstream subdivisions of a given reach are assumed to have achieved a state of equilibrium with each other.

4. Sediment transport capacity
Transport capacity can be computed by any applicable methods that have been discussed in previous lectures. As with the sediment supply, the transport capacity must be recomputed for each time interval of the hydrograph.

5. Comparison of sediment supply with transport capacity
A reach by reach comparison of sediment supply to transport capacity will yield a value of $\frac{\Delta Vol}{\Delta Time}$ in the sediment continuity equation. Knowing the time interval (ΔTime) and the length and width of a given subreach, the $\frac{\Delta Vol}{\Delta Time}$ term can be converted to a vertical dimension which represents the vertical adjustment to the river-bed for a specific time interval of the hydrograph. This comparison is made for all adjacent subdivisions of the river reach under investigation and for all time intervals of the hydrograph. The results of this comparison provide a mathematical description of the general scour/deposition patterns that will occur during a specific flood event.

must correct for porosity

6. Correction of sediment volumes for porosity
Most sediment transport equations yield sediment transport rates in terms of unbulked volume per unit time. However, in its natural state, sediment has a certain degree of porosity. Accordingly, the sediment volumes derived from the transport equations must be corrected for porosity prior to using these volumes to determine changes to bed elevations. This correction can be made with the following equation:

$$V_t = \frac{V_s}{1-n}$$

where V_t = bulked sediment volume

V_s = sediment volume computed by transport equations

n = porosity of sediment

.40

100 lbs/ft^3

7. Interpretation of results

The analysis of general scour/deposition can be conducted at different levels of accuracy. A qualitative assessment of this phenomenon can be made by simply comparing the sediment transport rates of adjacent river sections. Using the principle of sediment continuity, this comparison will indicate which sections would be expected to scour versus those expected to aggrade.

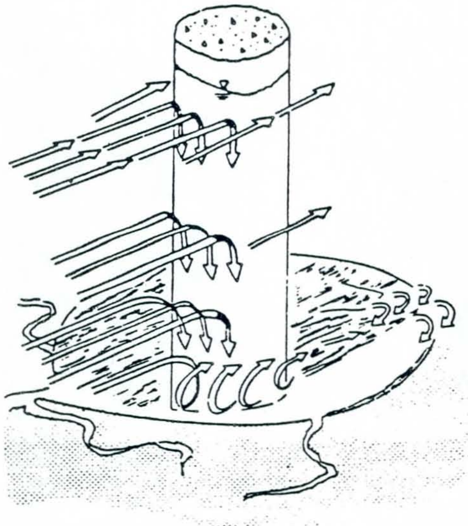
The analysis can be carried a step further by computing the $\frac{\Delta Vol}{\Delta Time}$ term in the continuity equation and applying this term to subdivided river reaches to estimate the magnitude of the vertical bed movement that might be expected in each reach. Using rigid-bed assumptions to compute the hydraulic parameters for use in the sediment transport equations, this procedure can easily be performed using a hand-held calculator.

With present state-of-the art technology, the analysis can be carried to a third level using a computer to perform quasi-dynamic sediment routing. Unlike the rigid boundary method discussed above, quasi-dynamic sediment routing updates the cross-sectional geometry at the end of each time interval and computes a new backwater profile, and associated hydraulic parameters, for use in the sediment transport calculations for the next time step.

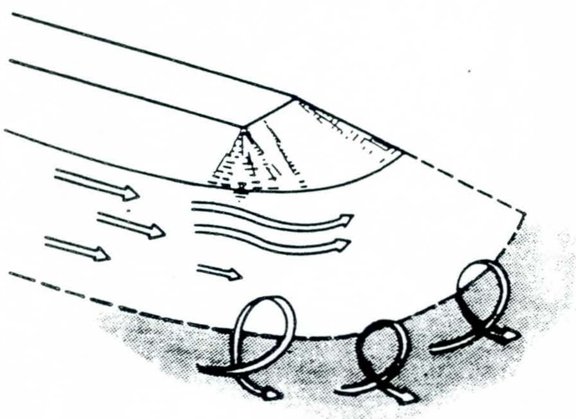
IV. LOCAL SCOUR

Local scour is a common phenomenon that most engineers are aware of, especially those involved in bridge design. Local scour is observed whenever an abrupt change in the direction of flow occurs, such as at bridge piers or embankments. Local scour at bridge piers is a result of high velocity vortex systems developed at the pier. Figure 4 illustrates this vortex action at both a bridge pier and an embankment. Local scour results from these vortex currents when they increase the capacity of the flow to remove or transport the bed-material in greater quantities than which replacement bed-material is being supplied. In essence, this is another application of the sediment continuity principle.

Local scour is difficult to model because of the complex three-dimensional flow patterns that are set into motion. For the same reason, accurate field measurements have also been difficult to obtain. Nevertheless, numerous formulas have been developed for predicting local scour. Most of these formulas have been based on flume experiments with little or no field verification.



Schematic representation of scour at a cylindrical pier.



Schematic representation of scour at an embankment.

Figure 4 - Illustration of local scour

A. Pier Scour

The following factors should be considered in the analysis of pier scour.

1. Change in scour depth with time
   Figure 5 presents a typical relationship between local scour depth and passage of a flood hydrograph. As would be expected, the maximum scour depth usually occurs during the peak discharge of the flood, which is normally when the highest velocities exist. As the scouring action decreases on the falling limb of the hydrograph, the scour holes are usually filled, thus leaving little to no evidence of the maximum scour that may have occurred during the peak of the flood.

2. Factors affecting pier scour
   Local scour at piers is strongly influenced by the pier shape (blunt nose, circular, etc.) and width, the velocity of flow, depth of flow, and sediment size. The vertical angle of the pier, with respect to the approaching flow, will also influence the scour depth. An illustration of the vertical angle of attack is illustrated in Figure 6.

3. Pier scour equations
   There are at least 10 prediction equations for bridge pier scour. An excellent summary of these equations is presented in a paper entitled Comparison of Prediction Equations for Bridge Pier and Abutment Scour authored by J. Sterling Jones. A copy of this paper is included in the Appendix to these lecture notes.

   Engineers are continually faced with a decision as to which equation would be most appropriate for a specific project application. Jones recommends that the following two approaches be pursued in selecting the appropriate equation for a bridge design:

   (1) The available equations should be compared with site specific field data to determine which ones best duplicate actual field measurements of scour.

   Use of this method can introduce errors in measurements because of the influence of other bed adjustment mechanisms that may be occurring simultaneously with pier scour. If the pier scour component cannot be isolated, errors will result in attempts to match an equation to the field measurements.

   (2) If sufficient data is not available under Approach

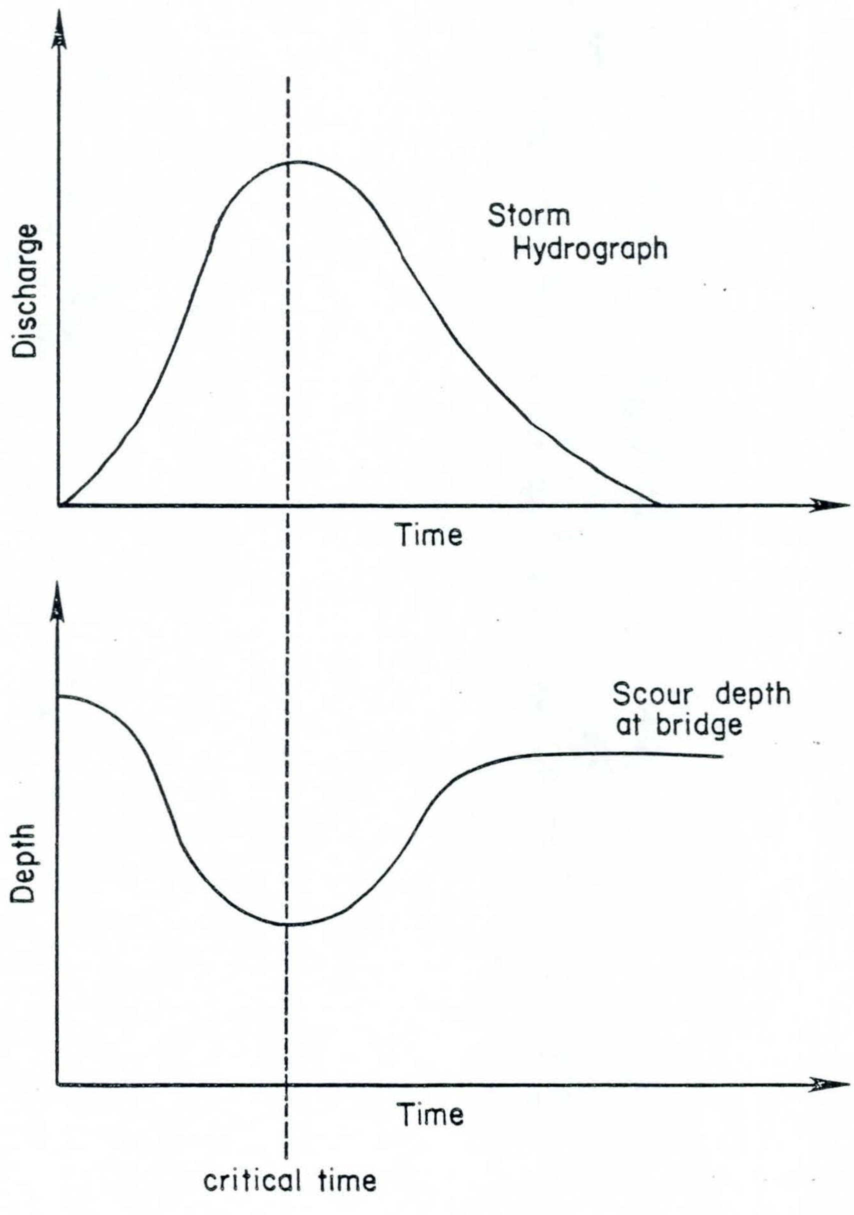


Figure 5 - Temporal change of scour hole depth during a storm (typical).

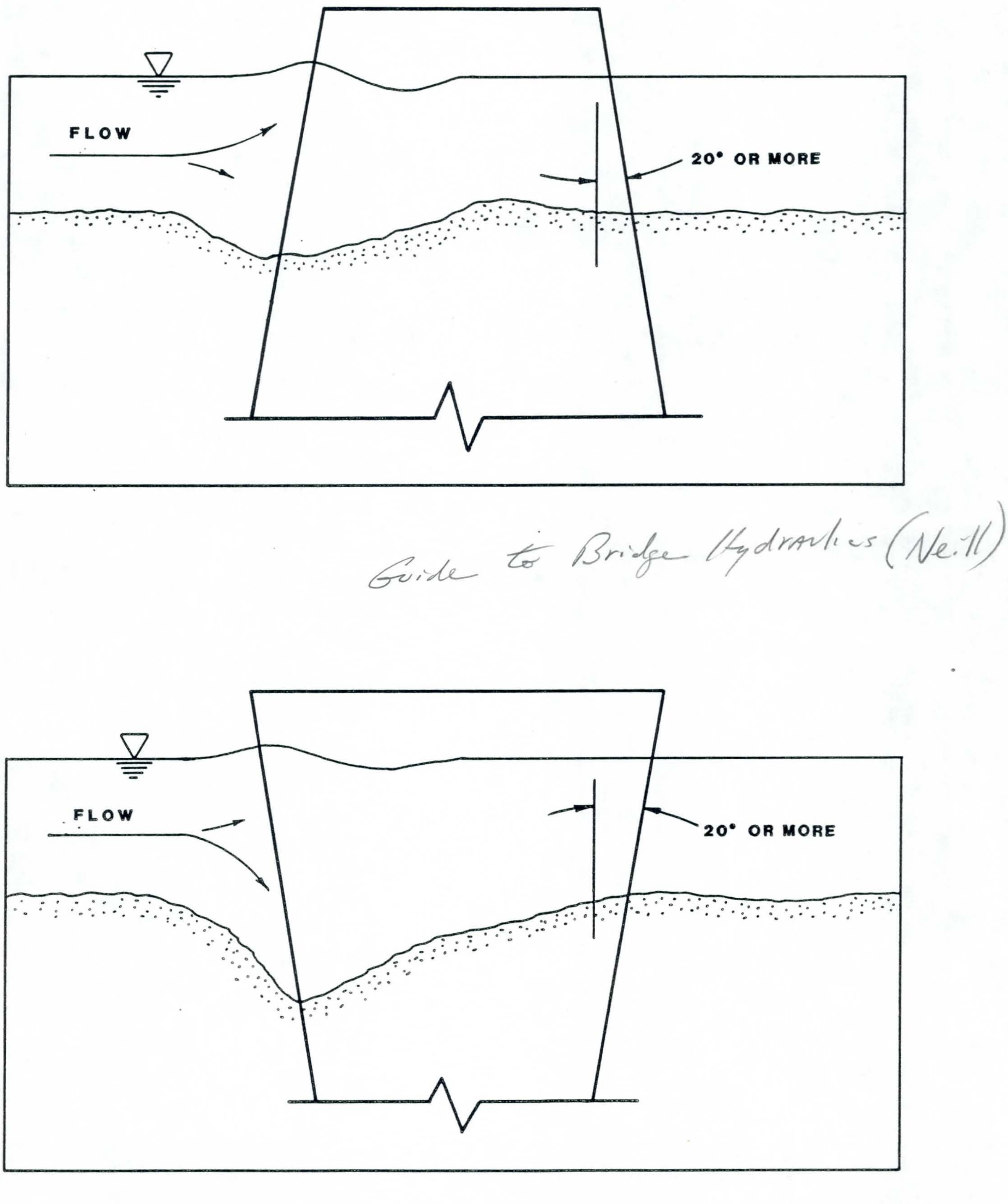


FIGURE 6 EFFECT OF VERTICAL PIER ANGLE ON SCOUR DEPTH

No. 1, the conditions under which the equations were derived should be evaluated and the one that best matches the design conditions should be used. Figure 4 of Jones' paper provides a concise summary of the data envelope used to develop pier scour equations.

B. Abutment scour

Figure 7 presents an illustration of scour pockets that can form near the nose of abutments that extend into a floodplain or main channel. Such abutments essentially act as constrictions to flow and create a localized increase in velocity as the water abruptly transitions from a wide cross-section to a narrow cross-section. This abrupt transition of flow also creates vortices which, when combined with the velocity increase, cause scour at the nose of the abutment.

Very little research has been done on abutment scour. Laursen employed the sediment continuity equation, along with an assumption of clear water in the overbank area, to develop an equation relating scour depth to the horizontal distribution and depth of flow, as well as to a velocity parameter.

Scale model flume experiments conducted by Liu resulted in an abutment scour equation relating scour depth to depth of flow, Froude Number, and embankment length.

These model studies employed simple, rectangular channel configurations, the results of which may not totally be transferrable to the more irregular channel geometry encountered under actual field conditions. Data from Liu's experiments, along with field data collected along the Mississippi River, suggests the following equations for computing abutment scour with subcritical flow:

$$\frac{\Delta Z_{1s}}{Y} = 1.1\left(\frac{a}{Y}\right)^{0.4} Fr^{0.33} \qquad \left(\frac{a}{Y} < 25\right)$$

and

$$\frac{\Delta Z_{1s}}{Y} = 4\,F_r^{0.33} \qquad \left(\frac{a}{Y} \geq 25\right)$$

where ΔZ_{1s} = scour depth measured from the mean bed level to the bottom of the scour hole

Y = upstream depth of flow

a = embankment length

F_r = upstream Froude Number

Application of these equations to field situations can be complicated when trying to define the location at which the variables should be measured. Suggested guidelines for

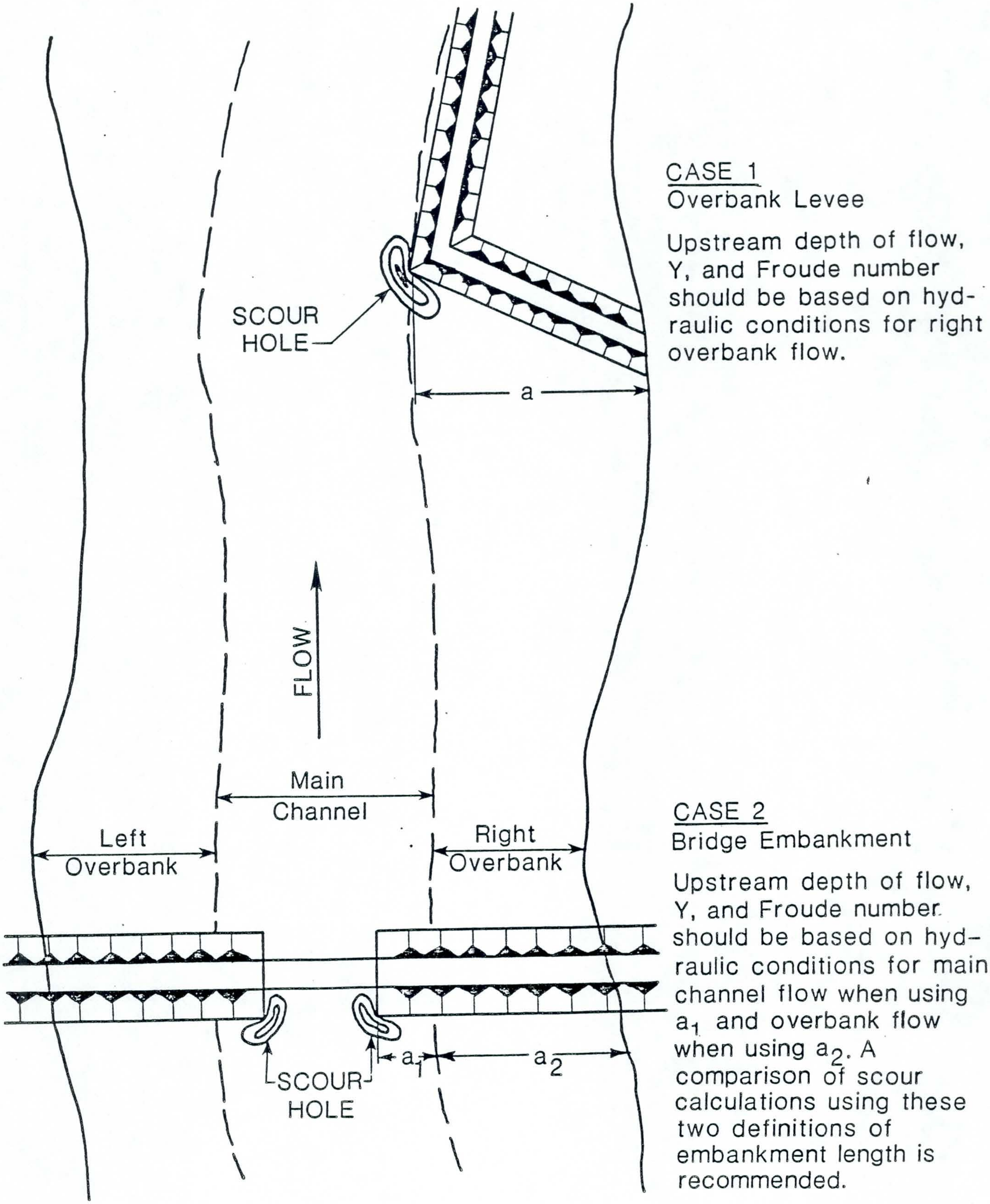


Figure 7 - Definition sketch of embankment length "a".

measuring these parameters are presented on Figure 7. These are only suggestions and should be tempered with engineering judgement, based on specific site conditions.

V. BEND SCOUR

Bend scour, as the name implies, occurs in channel bends which induce transverse or "secondary" currents which, in turn, scour sediment from the outside of a bend and cause it to be deposited along the inside of the bend. It is important to note that this scouring mechanism is caused by the spiral pattern of secondary flow, and is not due to a shift of the maximum longitudinal velocity filament against the outer bank. Channel bends will cause a shift in this velocity filament, but through the bend the maximum longitudinal velocity is normally moved nearer to the inside bank, whereas the shift to the outer bank occurs downstream of the bend. It is at these downstream locations that the shift in longitudinal velocity patterns will most likely cause lateral erosion of a channel bank. The mechanics of this phenomenon are illustrated in Figure 8.

Figure 9 presents topography and cross-sections of measured scour in a channel bend that was investigated through laboratory experiments conducted by I. L. Rozovskii (Flow Of Water In Bends Of Open Channels, 1957).

The purpose of this lecture will be to present a method for estimating the vertical magnitude of bend scour as well as the distance downstream of the bend that bend scour might be propagated.

A. Depth of bend scour

The following expression was developed by Zeller to estimate the maximum depth of bend scour:

$$\Delta Z_{bs} = \frac{0.0685\ YV^{0.8}}{Y_h^{0.4}\ S_e^{0.3}} \left[2.1 \left(\frac{\sin^2 \frac{\alpha}{2}}{\cos \alpha} \right)^{0.2} - 1 \right]$$

where Z_{bs} = bend scour component of total scour depth (feet)
V = mean velocity of upstream flow (fps)
Y = <u>maximum</u> depth of upstream flow (feet)
Y_h = <u>hydraulic</u> depth of upstream flow (feet)
S_e = upstream energy slope (bed slope for uniform flow conditions, feet/feet)
α = angle formed by the projection of the channel centerline from the point which meets a line tangent to the outer bank of the channel (degrees, see Figure 10).

Mathematically, it can be shown that, for a simple circular curve, the

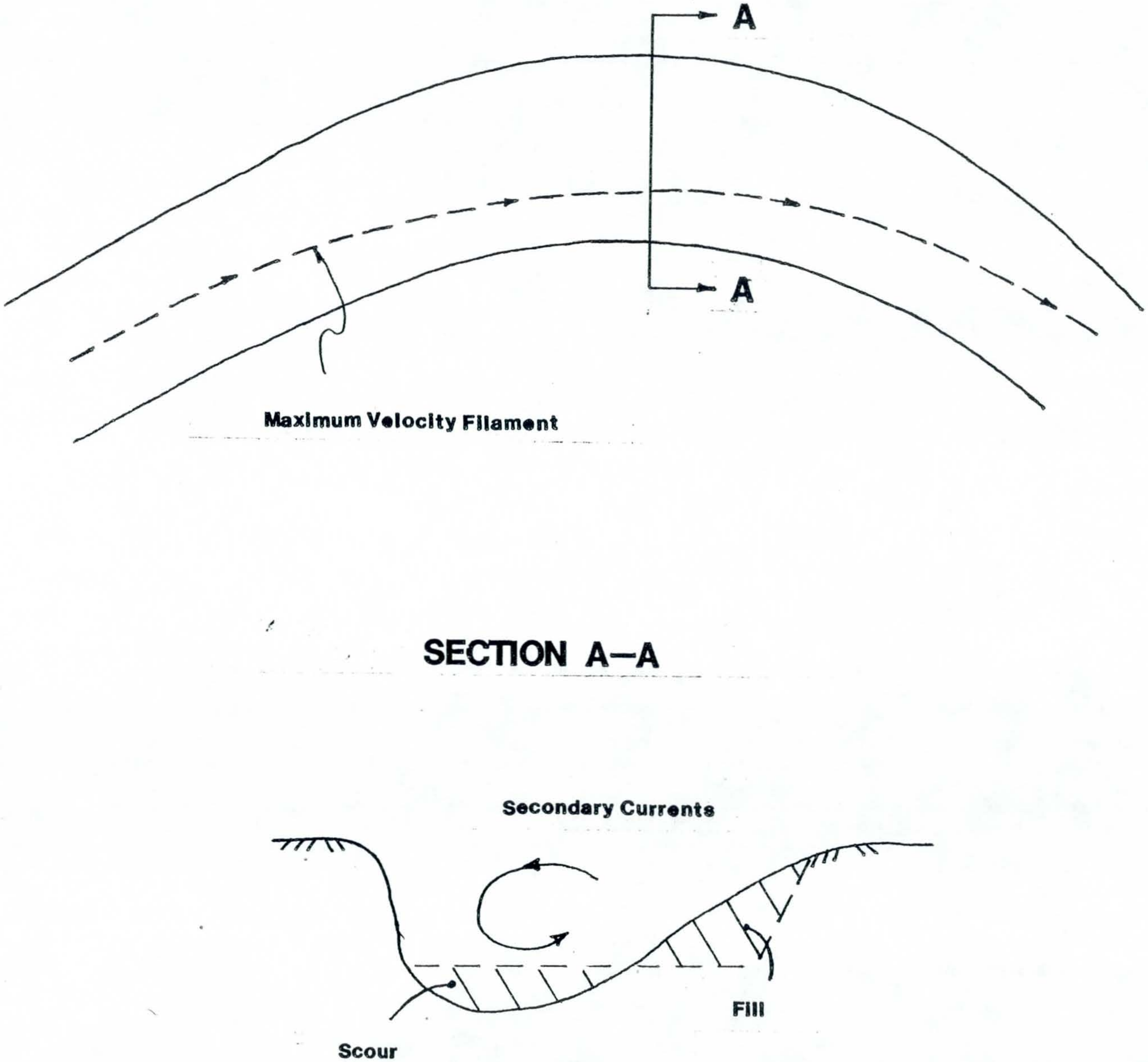
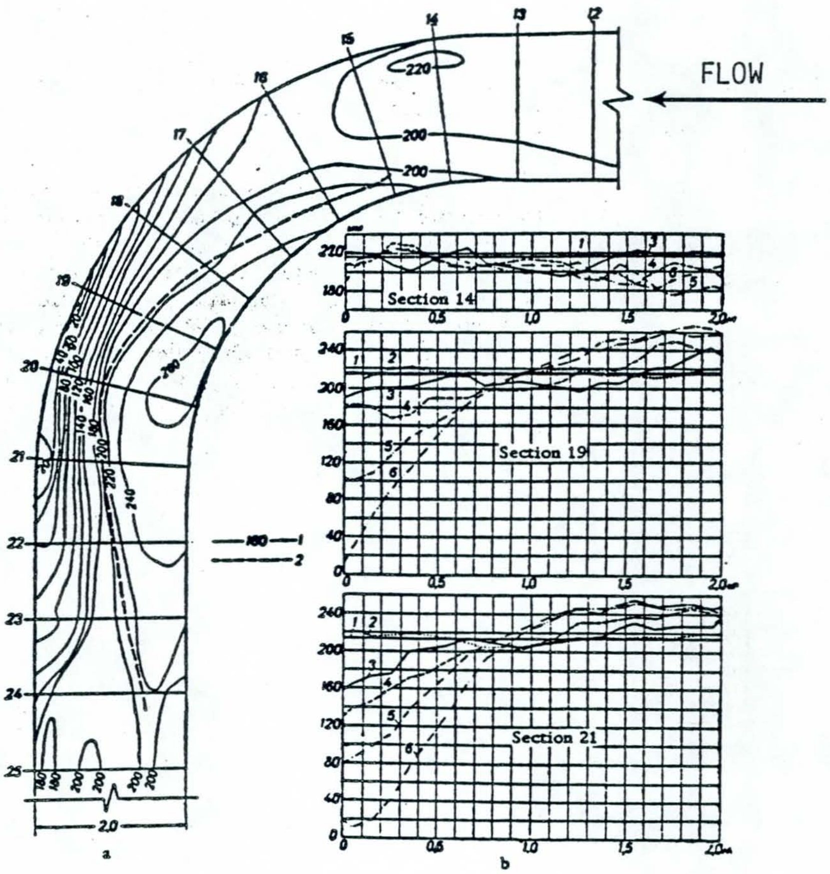


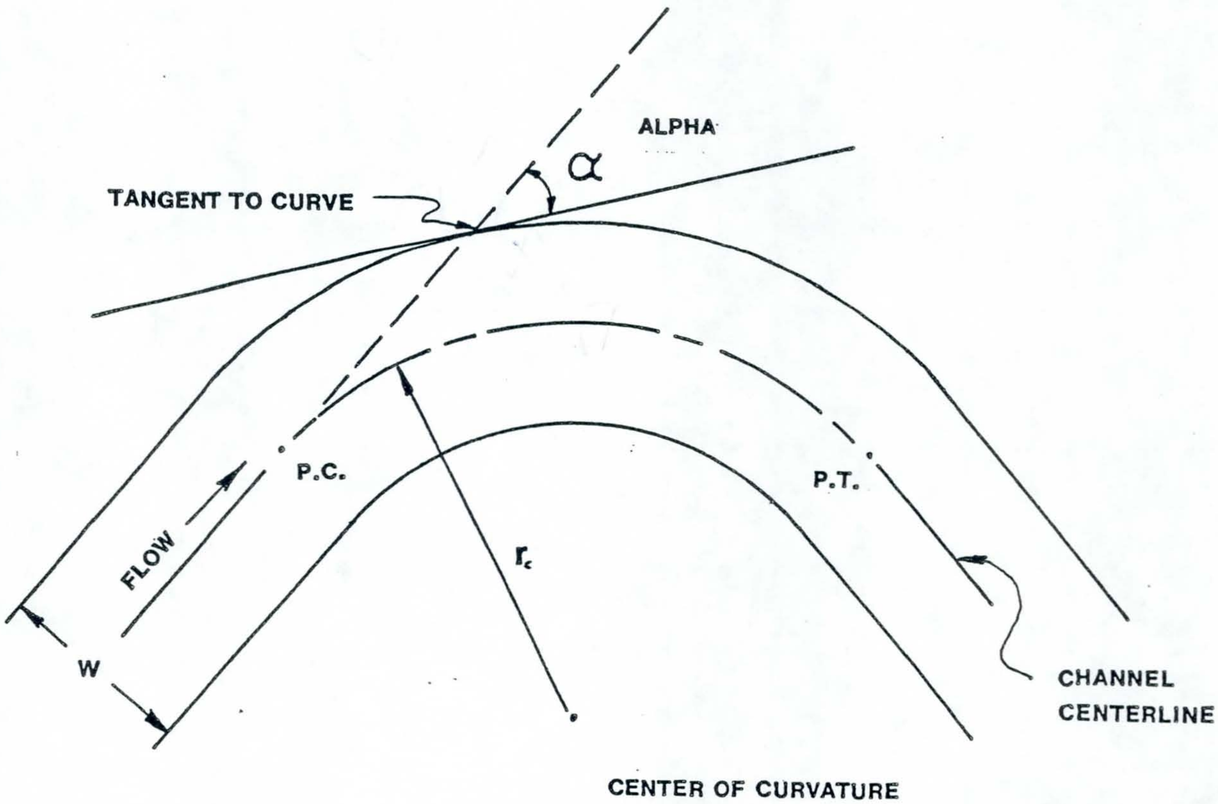
Figure 8 - Mechanics of bend scour



a–topography of bottom in a model of channel bend; 1–contour lines; 2–boundary between erosion and deposits; b–consecutive bottom deformation stages in channel bend

Figure 9 - Laboratory Measurements of Bend Scour

Reference: Flow Of Water In Bends Of Open Channels, by I. L. Rozokskii



NOTE: α MUST BE $\geq 18^\circ$ WHEN USING THE BEND SCOUR
$\alpha \leq 60^\circ$
EQUATION PRESENTED IN THIS SECTION

Figure 10 - Illustration of terminology for bend scour calculations.

$$D + Z + \frac{V^2}{2g} = K$$

Pier Scour — CSU equation

$$\frac{\Delta z}{y} = 2.2\left(\frac{b_r}{y}\right)^{.65} Fr^{.43}$$

Shen's equation

$$\Delta z_{eq} = 7.3 \times 10^{4} R_p^{.619}$$

$$\frac{\Delta z_{ds}}{b_p} = 11.\, F_{Rp} \text{ for } F < .2$$

following relationship exists between α and the ratio of radius of curvature to channel topwidth.

$$\frac{r_c}{W} = \frac{\cos \alpha}{4 \sin^2(\frac{\alpha}{2})}$$

where r_c = radius of curvature to centerline of channel (feet)
W = channel topwidth (feet)

If the bend under evaluation deviates significantly from a simple circular curve, the engineer should consider dividing the bend into a series of circular curves and analyzing the bend as a compound curve. This concept is illustrated in Figure 11. Under this procedure, there would be a different value of α determined for each segment of the compound curve. A scour depth would then be computed for each segment of the curve using the α determined for that segment.

B. Downstream extent of bend scour

An approximation of the longitudinal extent of bend scour can be based on the downstream distance required for the dissipation of the secondary currents that are responsible for bend scour. Rozovskii developed an expression for predicting the distance from the end of a bend at which the secondary currents will have decayed to a negligible magnitude. This relationship is:

$$X = 2.3 \left(\frac{C}{\sqrt{g}}\right) Y$$

$C = \frac{1.49}{n} R^{1/6}$

where X = distance from the end of channel curvature (point of P.T.) to the downstream point at which secondary currents have dissipated (feet)

C = chezy coefficient
g = gravitational acceleration (32.2 feet/second^2)
Y = depth of flow (to be conservative, use maximum depth of flow, exclusive of scour, within the bend) (feet)

This expression should only be used as a guide in determining the distance downstream of a curve that secondary currents will continue to be effective in producing bend scour. As a conservative estimate of the longitudinal extent of bend scour, both through and downstream of the curve, the engineer would be advised to consider bend scour commencing at the upstream point of curvature (P.C.) and extending a distance X downstream of the point of tangency (P.T.). Engineering judgement should be used in electing to deviate from this generalized recommendation.

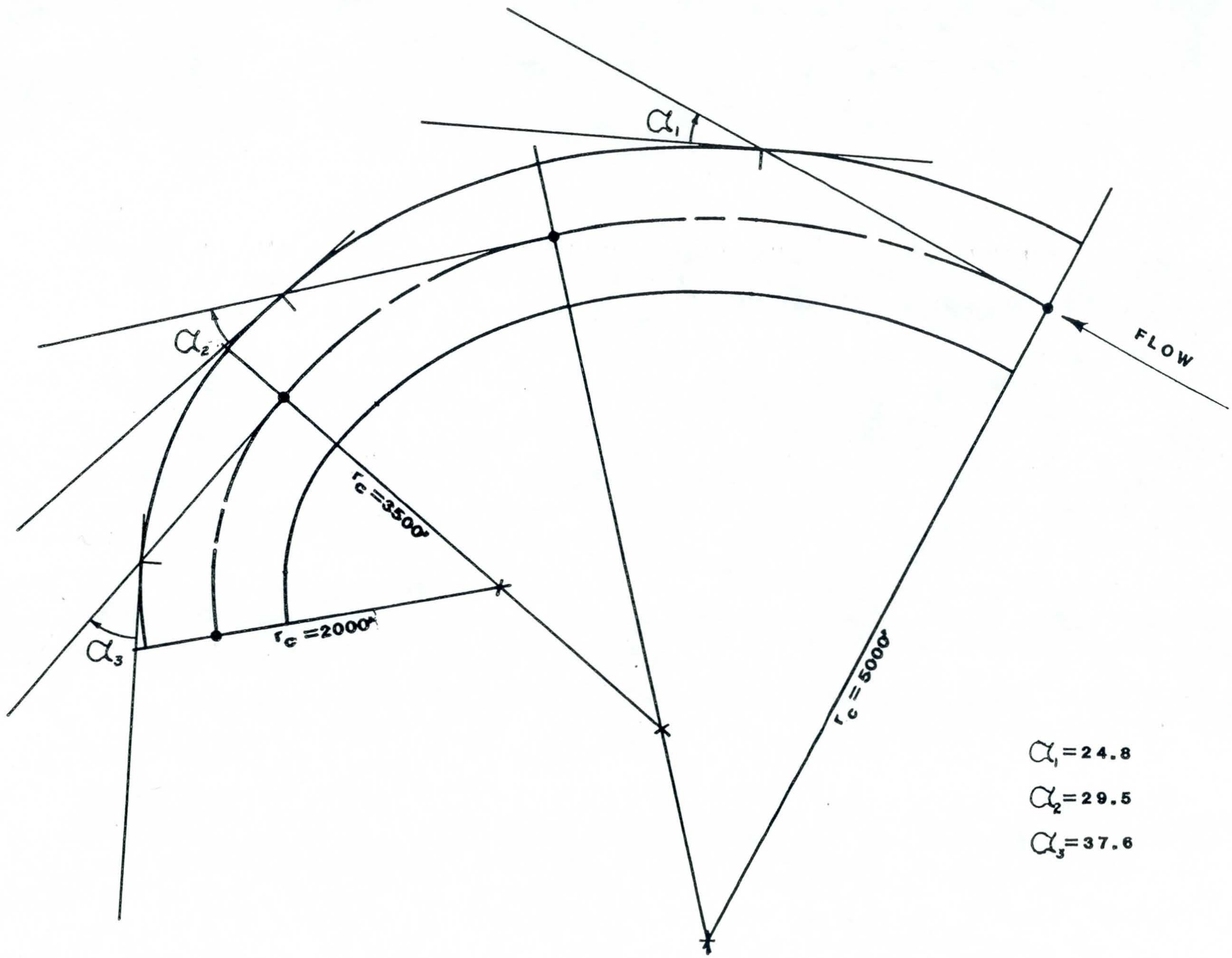


FIGURE 11 USE OF COMPOUND CURVES FOR BEND SCOUR ANALYSIS

VI. LOW-FLOW INCISEMENT

Low-flow incisement is an attempt by nature to transport water in the most efficient manner possible, i.e., with a minimum expenditure of energy. Low-flow incisement is normally in response to small, frequent flows in a channel. Rather than carrying this water in a uniform, shallow depth, occupying the entire channel bottom and creating a substantial amount of frictional resistance due to the large wetted-channel perimeter, the stream will incise a more efficient, compact channel cross-section which is capable of carrying the amount of water with much less frictional resistance (due to the smaller wetted-channel perimeter). Figure 12 illustrates this concept with some calculated channel parameters showing the differences in cross-section characteristics. Notice the large decrease in wetted perimeter that accompanys the incised channel.

There are no rigorous methodologies for the prediction of low-flow channel incisement. A field inspection of the study area is probably the best method to determine the potential for this phenomenon. If the existing channel does not have low-flow incisement, but proposed channelization or other changes result in conditions favorable for low-flow channel development (i.e., a wide, flat bed with no downstream grade control), then as a rule of thumb, a reasonable incisement depth is one to two feet.

Since low-flow channels are capable of meandering throughout the main channel, the low-flow incisement depth should be added to all other vertical channel adjustments that may be used to determine the total scour depth for bridge piers, pipelines, bank stabilization, etc. If no channel improvements (grading or shaping) are proposed, and a low-flow channel already forms the invert of the channel cross-section, it is appropriate to reference all other scour calculations to this existing low-flow invert. Obviously, under this condition an additional allowance for low-flow incisement need not be added to the existing invert elevation.

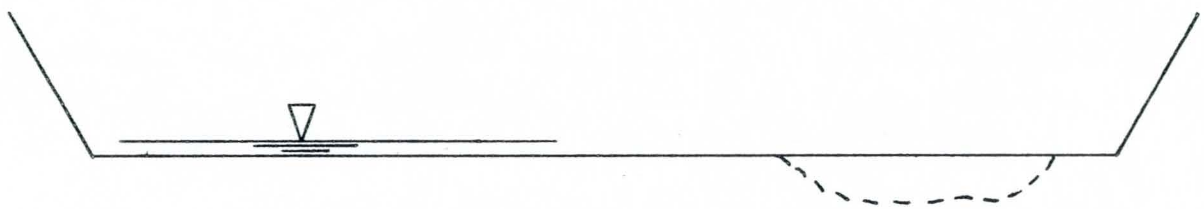
VII. INFLUENCE OF BED FORMS

The movement of flowing water over the natural earth bed of a river or channel can cause irregularities in the bed profile. These irregularities are often referred to as bed forms or sand waves. Sand waves may be somewhat of a misnomer since the bed forms can occur in channels not having sand beds.

The actual formation and behavior of bed forms have been observed and studied in laboratory flumes. This research has shown that there is an orderly progression of changes to bed forms as velocity of flow increases. Research performed by Simons and Richardson resulted in the following classification of bed forms according to flow regime:

LEGEND

Main Channel	————
Incised Low-Flow Channel	- - - - -



Wide Low Flow	Incised Low Flow
BW = 100 ft	BW = 20 ft
D = 0.4 ft	D = 1.05 ft
n = 0.025	n = 0.025
S = 0.002	S = 0.002
A = 40 ft^2	A = 22 ft^2
P = 101 ft	P = 23 ft
R = 0.4	R = 0.96
V = 1.44 fps	V = 2.62 fps
Q = 58 cfs	Q = 58 cfs

Figure 12 - Illustration of low-flow channel incisement and comparison of hydraulic calculations

Lower flow regime

1. Ripples

2. Dunes

Transition zone: bed configurations range from dunes to plane bed or to antidunes.

Upper flow regime:

1. Plane bed with sediment movement

2. Antidunes

3. Chutes and pools

For practical purposes, the lower flow regime can be considered subcritical flow; the upper flow regime is supercritical; and the transition zone encompasses the flow characteristics that occur during passage from the lower regime to the upper regime.

The bed forms that are of primary interest in designing structures for bed elevation changes are dunes and antidunes.

A. Antidunes

Antidunes can form in either the transition zone (between lower and upper regime) or upper flow regime (Simons and Senturk, 1977). Kennedy (1963) made a detailed study of antidune flow. He suggested that the wave length is generally given by $2\ V^2/g$ (g is the gravitational acceleration) and two-dimensional waves break when the ratio of wave height to wave length reaches a value of approximately 0.14. This theory assumes that the depth of flow is roughly equal to the maximum height of the antidune. Thus, the antidune height h_a from crest to trough (see Figure 13) can be estimated utilizing the relation

$$h_a = 0.14 \frac{2\pi V^2}{g} = 0.027\ V^2$$

for $h_a < y$; assume $h_a = y_o$ when the calculated value of $h_a > y_o$, since h_a can never be greater than y_o.

B. Dunes

Lower regime flow also produces bed forms which should be considered in designing levee, channel, or bridge projects. Based on data collected from flume experiments (Simons and Richardson, 1960), dune formations have been observed at Froude numbers ranging from 0.38 to 0.60. The ratio of depth of flow to dune need to consider change in "n" due to bedforms

$h_a = .0027 V^2$

h_a cannot exceed y / flow depth

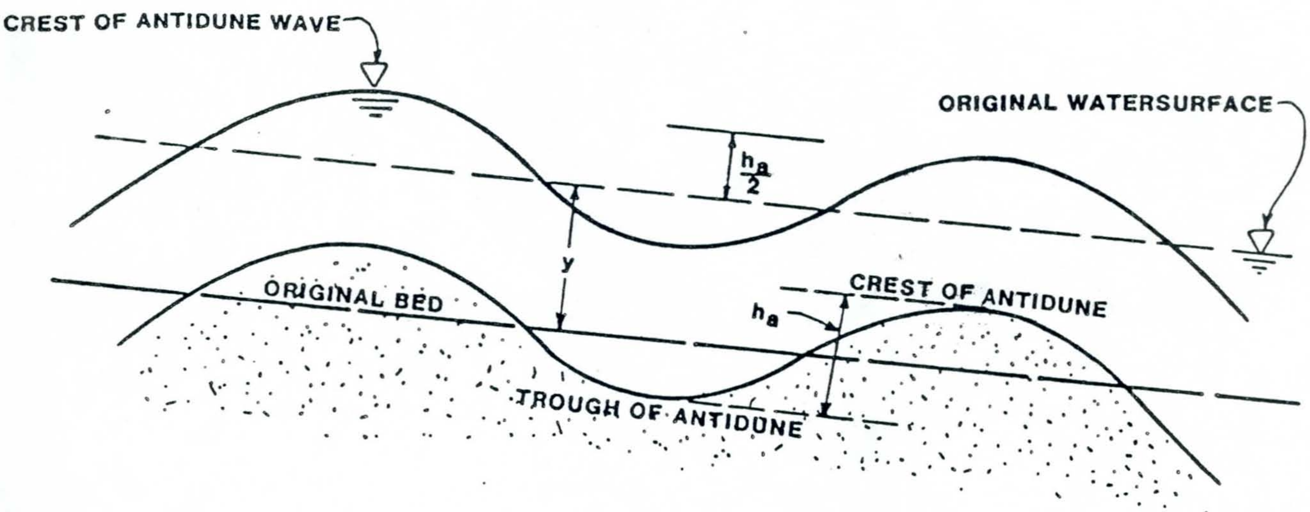


Figure 13 - Illustration of antidune bed-forms

height (d/h) ranged from 1 to 5. When this ratio is 1.0, the dune troughs could be depressed below the natural channel bed a distance equal to one-half the depth of flow. As a conservative guideline, this value (one-half the depth of flow) may be used to account for dune troughs forming adjacent to a structure.

C. Limitations

The experimental data used to describe the behavior of bed forms is based on sand-bed laboratory flumes. The potential for bed forms to develop will depend upon both velocity and bed-material size. As long as sand-bed channels are being analyzed, the quantitative guidelines presented above should provide reasonable estimates of bed form depths. However, applying these guidelines to cobble bed streams may result in overly conservative estimates. Engineering judgement must be used in such cases. Figure 14 presents experimental data, developed by Simons and Richardson, showing the relationship among stream power, median fall diameter, and bed form. For practical purposes, the median fall diameter can be approximated as the D_{50} size of the bed material.

VIII. ARMORING POTENTIAL

As the name suggests, channel armoring occurs when a layer of non-moving particles form on the bed of a channel and prevent further downward deformation of the channel bed. Armoring is a function of velocity (shear stress) and particle size. As flowing water initiates bed-material movement, the small, fine-grain particles, that are unable to withstand the force of the flowing water, are swept away, leaving the larger, non-moving particles on the channel bed. This process continues until only those particles large enough to resist movement (due to the flowing water) are left on the surface of the channel bed. Unless the velocity is increased beyond that required to initiate motion of these larger particles, further lowering of the bed will not occur. At this point, "armoring" is said to have occurred.

The formation of an armor layer does not mean that all particles beneath the armor layer are large enough to resist movement at the velocity for which the channel is armored. The accumulation of large non-moving particles in the armor layer serves as a shield to prevent leaching of the finer (smaller) particles that may still exist below the armor layer. If the velocity of flow were increased beyond that for which the channel is armored, the armor layer would be disrupted (broken) as some of the surface particles would now be swept away. This would expose the fine-grain materials below the old armor layer to the flowing water and more material would be swept until a new, larger-particle, armor layer is formed.

As the smaller particles are swept away during the armoring process, a bad scan

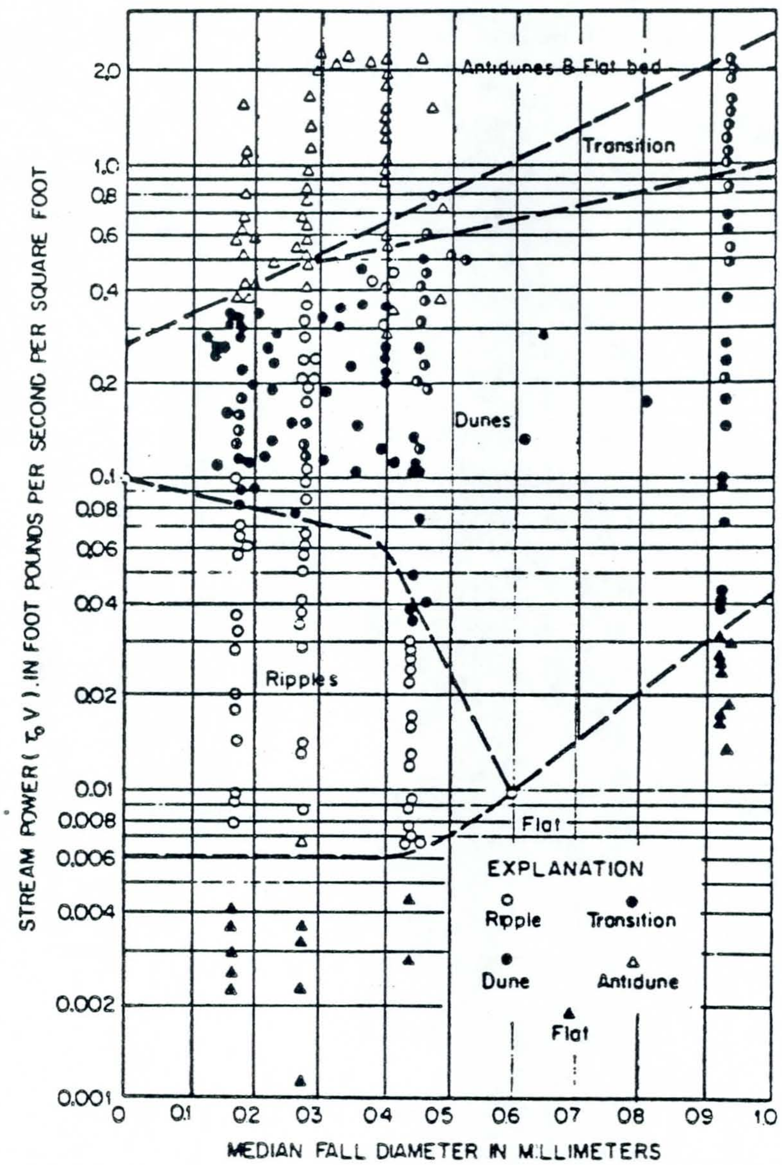


Figure 14 - Relation of bed form to stream power and median fall diameter of bed sediment (after Simons and Richardson, 1966).

volume of material is obviously being removed from the channel bed. The removal of this material causes degradation of the bed. When designing structural improvements in a river environment, it is important to how much degradation may occur during the armoring process, or perhaps even more importantly, whether there are even particle sizes existing at a project site capable of producing an armor layer under the anticipated design conditions.

A procedure has been developed by the United States Bureau of Reclamation (Design of Small Dams) which predicts the depth of degrdation required to establish an armor layer. This relationship is:

$$\Delta Z_a = Y_a \left(\frac{1}{P_c} - 1\right)$$

where ΔZ = degradation depth required to establish an armor layer

Y_a = thickness of the armor layer

P_c = decimal fraction of material courser than the armoring size (D_a)

Figure 15 illustrates the relationship between the degradation process and the physical parameters used in this procedure.

Values for Y_a will range from one to three times the armor particle size (D_a). Field observations suggest that a relatively stable armor layer requires a minimum thickness of two layers of armoring size particles (2 D_a). D_a can be computed from incipient motion criteria using the relationship:

$$D_a = \frac{\tau}{0.047\ (\gamma_s - \gamma)}$$

greater Da would cause armoring

where D_a = diameter of particle size for the condition of incipient motion (on the verge of movement)

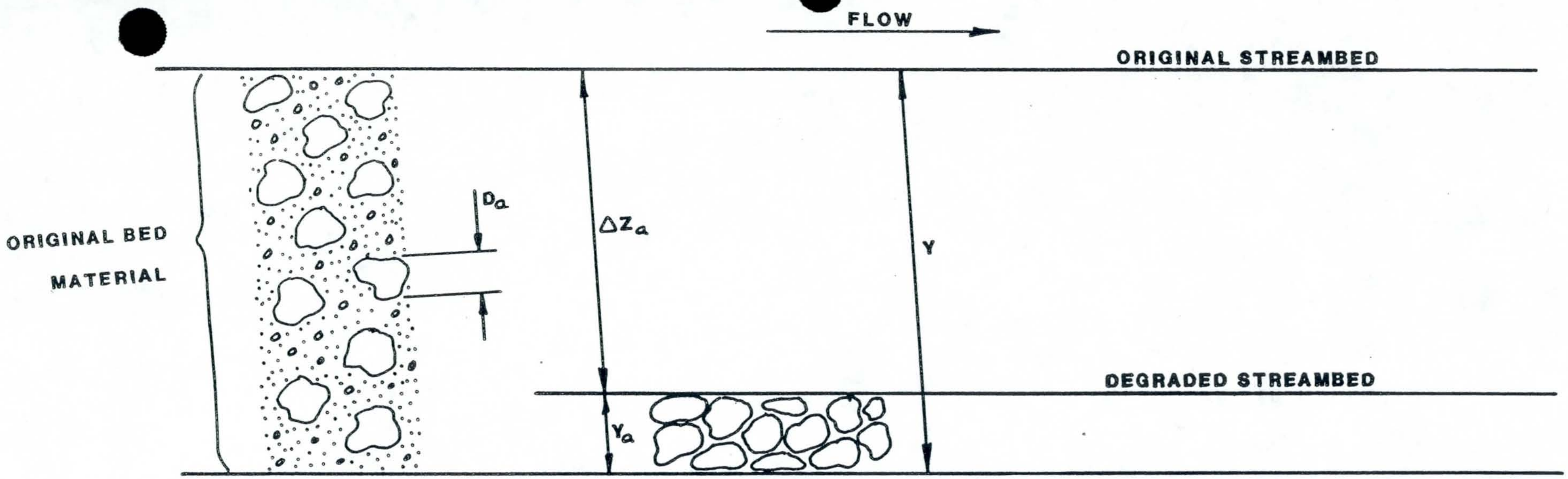
τ = boundary shear stress

γ_s = specific weight of sediment

γ = specific weight of water

IX. SUMMARY OF BED ADJUSTMENT MECHANISMS

From the preceding lectures, it has been shown that there are at least six phenomena capable of producing vertical movement in a river or channel bed. These phenomena must be considered in the design of any man-made improvements to be located in a floodplain environment. Failure to do so may lead to the pre-nature destruction of such impro-



$$\Delta Z_a = Y_a \left(\frac{1}{P_c} - 1\right)$$

Y = DEPTH TO BOTTOM OF THE ARMORING LAYER

ΔZ_a = DEPTH OF DEGRADATION

Y_a = ARMORING LAYER

D_a = DIAMETER OF ARMOR MATERIAL

P_c = DECIMAL PERCENTAGE OF ORIGINAL BED MATERIAL LARGER THAN D_a

FIGURE 15 ILLUSTRATION OF ARMORING PROCESS vements.

The interaction of these six bed adjustment mechanisms is not completely known. Until additional research provides a more detailed description of the interaction between these mechanisms, it is recommended that a conservative approach be pursued in analyzing the total vertical bed movement attributed to such phenomena. Such an approach would be to assume that each of the six mechanisms acts independently of the other five. Under this concept, the total vertical bed movement would be equal to the sum of the six bed-adjustment components.

Mathematically, this is expressed as follows:

$$\Delta Z_{tot} = \Delta Z_{agg/deg} + \Delta Z_{gs/d} + \Delta Z_{ls} + \Delta Z_{bs} + \Delta Z_{i} + \tfrac{1}{2} h_a$$

where ΔZ_{tot} = total vertical adjustment in bed elevation

$\Delta Z_{agg/deg}$ = adjustment due to long-term aggradation/degradation

$\Delta Z_{gs/d}$ = adjustment due to general scour/deposition

ΔZ_{ls} = adjustment due to local scour

ΔZ_{bs} = adjustment due to bend scour

ΔZ_{i} = adjustment due to low-flow incisement

$\frac{1}{2} h_a$ = adjustment due to bed-form troughs (dunes, antidunes)

If site specific calculations indicate aggradation or deposition will accompany the $\Delta Z_{agg/deg}$ or $\Delta Z_{gs/d}$ terms, it is recommended, as a conservative practice, that these terms be ignored in computing the total downward bed movement.

Due to the complex interaction that will occur among these six phenomena, it is perhaps impossible to accurately predict the total cumulative bed adjustment that might occur at a given location. The hydraulic parameters (velocity, depth, top width, etc.) that are used to compute the dimension of each phenomenon will constantly change as this interaction proceeds; however, the parameters that are used in the calculations are normally based on rigid-bed conditions which give no consideration to channel geometry changes that may be initiated as a result of the simultaneous occurrence of all or part of the six phenomena. Accordingly, the application of a factor of safety to the total computed vertical adjustment (ΔZ_{tot}) is very judgmental, i.e., no firm value can be recommended. In deciding to apply a factor of safety to the computed result, the engineer should consider the magnitude of damage that might accompany a design failure, the probability or risk that such an event might occur, the construction cost associated with applying a safety factor, and the reliability of the data that were used in the channel adjustment calculations. Depending upon the answer to such questions, typical safety factors will probably range from 1.0 to 1.5.

QUANTITATIVE ANALYSIS

LECTURE 8

GEORGE K. COTTON, P.E.

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Fort Collins, Colorado 80522

July 1986

LECTURE 8: Quantitative Analysis

OBJECTIVE: To demonstrate the practical use of several quantitative tools for analysis of short and long-term river response.

I. GENERAL INTRODUCTION

This workshop consists of exercises that will acquaint the user with design procedures for the quantitative analysis of aggradation and degradation. Computation of short-term aggradation and degradation is conducted using simplified sediment transport equations and the sediment continuity equation. Computation of long-term channel profile response is conducted next using the equilibrium slope method. The workshop is based on data collected on the Santa Cruz River near Tucson, Arizona. This is the same area that was covered in the previous workshop on qualitative response.

II. SHORT-TERM AGGRADATION/DEGRADATION ANALYSIS

A. Sediment Continuity Principle

$$Qs_{in} - Qs_{out} = dVol/dt$$

where $Qs = a\ Y^b\ V^c$

B. Review of Study Area

- Upstream sediment supply reach

C. Hydraulic Conditions

D. Computation of Aggradation and Degradation

E. Comparison to Level I Analysis

III. EQUILIBRIUM SLOPE ANALYSIS

A. The Equilibrium Concept

$$Qs_{in} = Qs_{out}$$

B. Computation of Equilibrium Slope

C. Sensitivity Analysis

D. Comparison to Level I Analysis

DESIGN CRITERIA

LECTURE 9

RUH-MING LI, Ph.D., P.E.

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Fort Collins, Colorado 80522

July 1986

LECTURE 9: Design Criteria

OBJECTIVE: This lecture demonstrates the combined application of quantitative analysis techniques for the design of various hydraulic structures. A detailed case study is presented for the Agua Fria River that covers the design of three major types of hydraulic structures: bridge crossing, channelization, and grade control. Background on the initial qualitative analysis phase of the project is also given.

I. GENERAL INTRODUCTION

Project objectives and scope determine the type of analysis and level of effort necessary. Through proper selection and application of the methodologies presented, the engineer or designer can complete a logical sequence of analysis that provides a comprehensive understanding of the fluvial system and its response mechanisms. In the design phase of the project, such knowledge establishes the design criteria for hydraulic structures in the fluvial environment.

Design criteria for fluvial structures are based on the potential for either loss of life or significant property damage. A major structure is one where if failure happens, loss of life is likely and/or property damage is significant. For a minor structure, loss of life is unlikely and property damage is small if the structure fails. Major structures require a rigorous analysis in order to reduce the uncertainty about the performance characteristics of the structure under severe conditions. It may be preferable with major structures to establish risk based criteria where the objective is to minimize the combined cost associated with construction of the project and the associated risk cost. The uncertainty associated with minor structures, can often be addressed using standard criteria and simplified design methods.

II. BRIDGE DESIGN

A. Hazards at Bridge Waterways

- Lateral migration
  - Long-term migration process
  - Abrupt changes during high flow

- Bed Elevation Changes at Abutments and Piers
  - Cumulative long-term changes
  - Changes due to severe events
  - Local scour

B. Protection Criteria

- Depth criteria for abutments
  - Toe down to total scour
- Depth criteria for bridge piers
  - Placed to depth of total scour
  - Limited research on preventative measures
- Control of lateral migration
  - Guide banks, spur dikes, jette fields, etc.
- Low chord criteria
  - Freeboard sufficient to pass debris
  - Allowance for potential aggradation
  - Allowance for waves due to antidunes

III. CHANNELIZATION

A. Hazards for Channelization

- Bank protection failure
- Excessive scour
- Inadequate freeboard
- Lateral migration

B. Protection Criteria

- Toe down to scour depth
- Bank/levee height

° Lateral migration

- Bank protection methods

- Buffer Zone

IV. GRADE CONTROL STRUCTURES

A. Use of Grade Control Structures

° Stabilization of river profile

° Reduce the need for other types of bank stabilization

° Assist in the development of armoring

B. Types of Grade Control Structures

° Small riprap drops

° Sheet piling

° Soil-cement

° Concrete with baffled apron and stilling basin

C. Spacing of Grade Controls

° Function of drop height and existing and final design channel gradients

° Number of structures to control a total reach

D. Hazards at Grade Controls

° Lateral migration

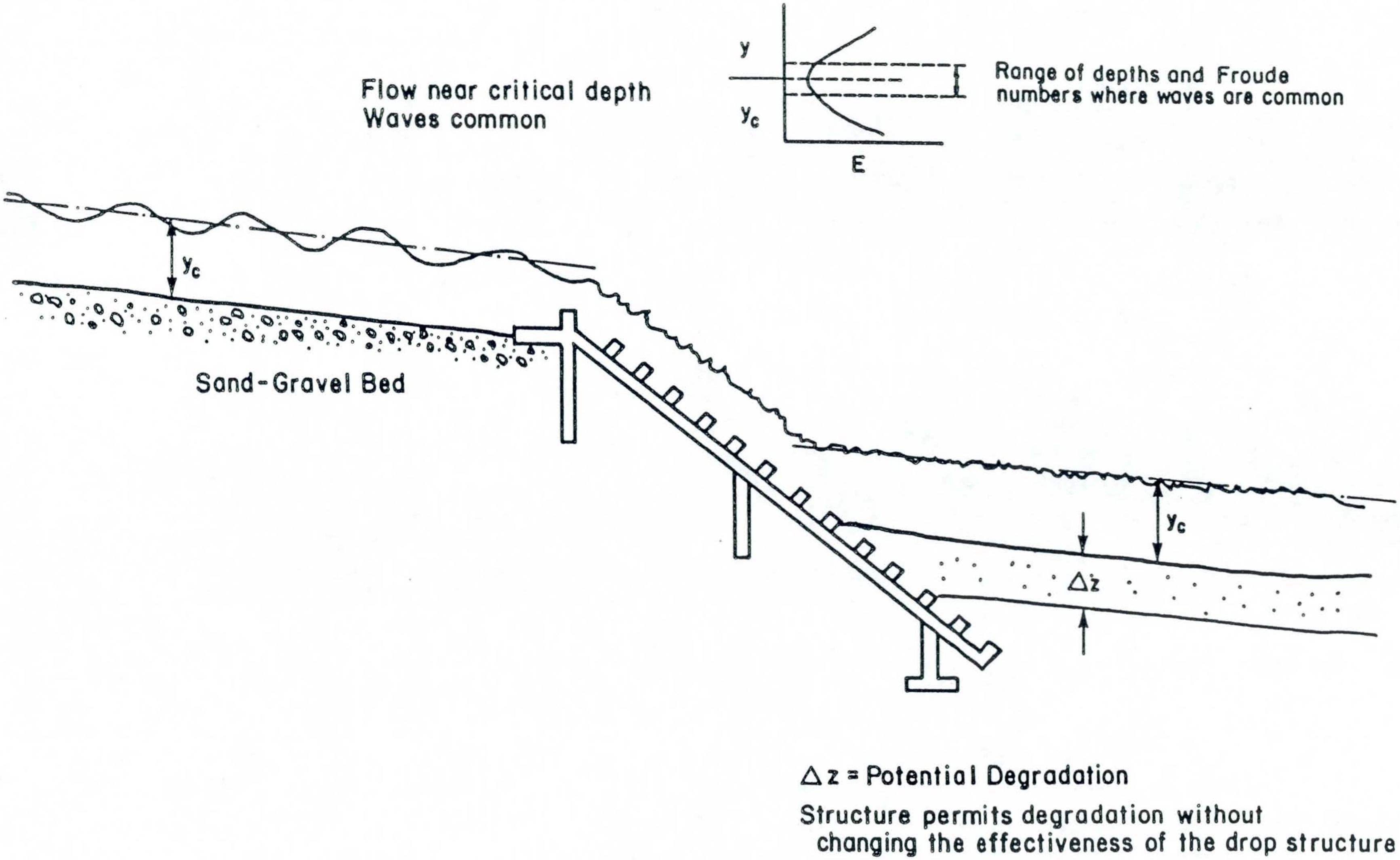
° Geotechnical failure (soil piping, uplift)

° Scour

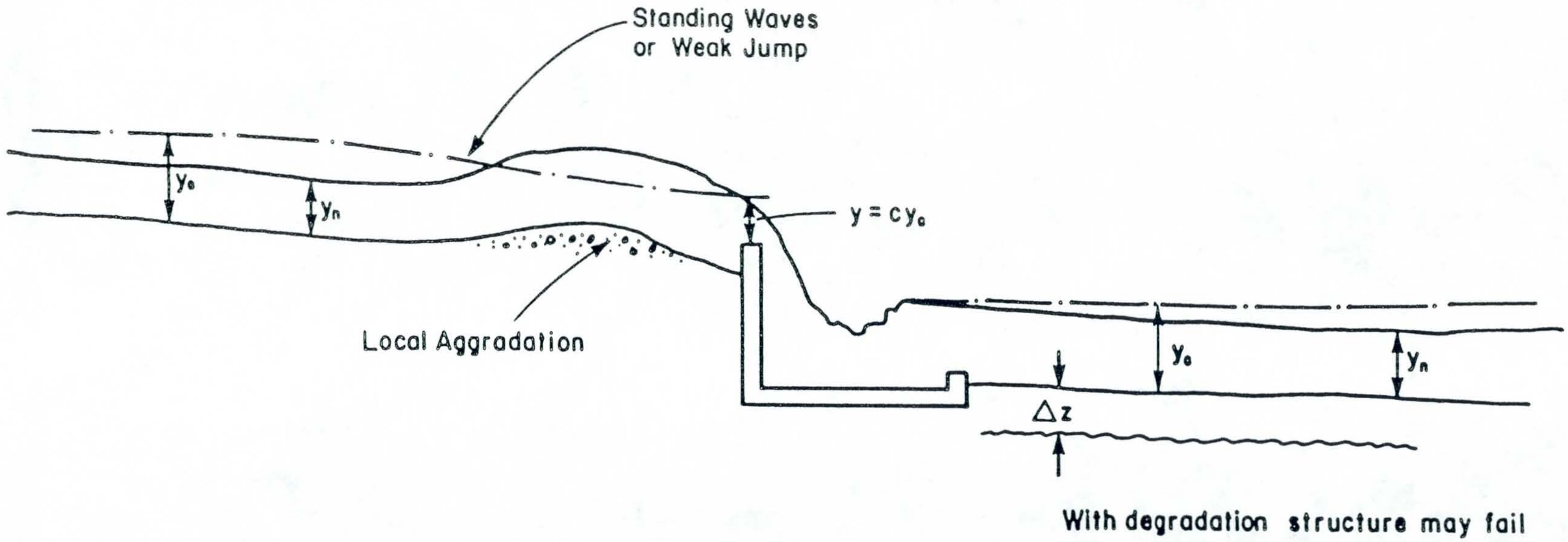
V. MAJOR STRUCTURE DESIGN ON THE AGUA FRIA RIVER

A. General Introduction

SLA has conducted a number of design projects on the Agua Fria River that have involved most types of major structures. This case study is a good example of the application of multi-level analysis and formulation of design criteria. This case study



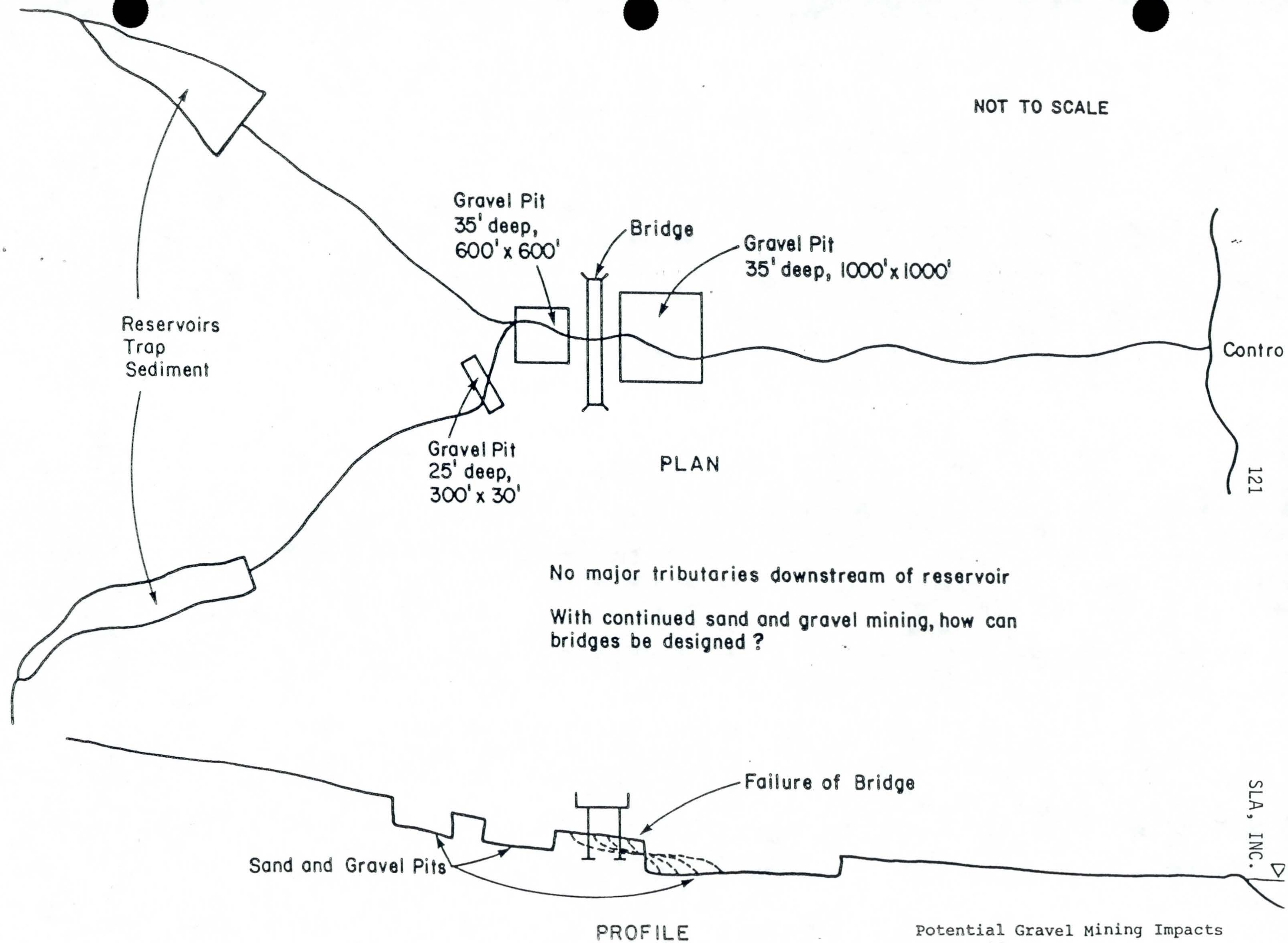
Drop Structure Design That Allows for Degradation



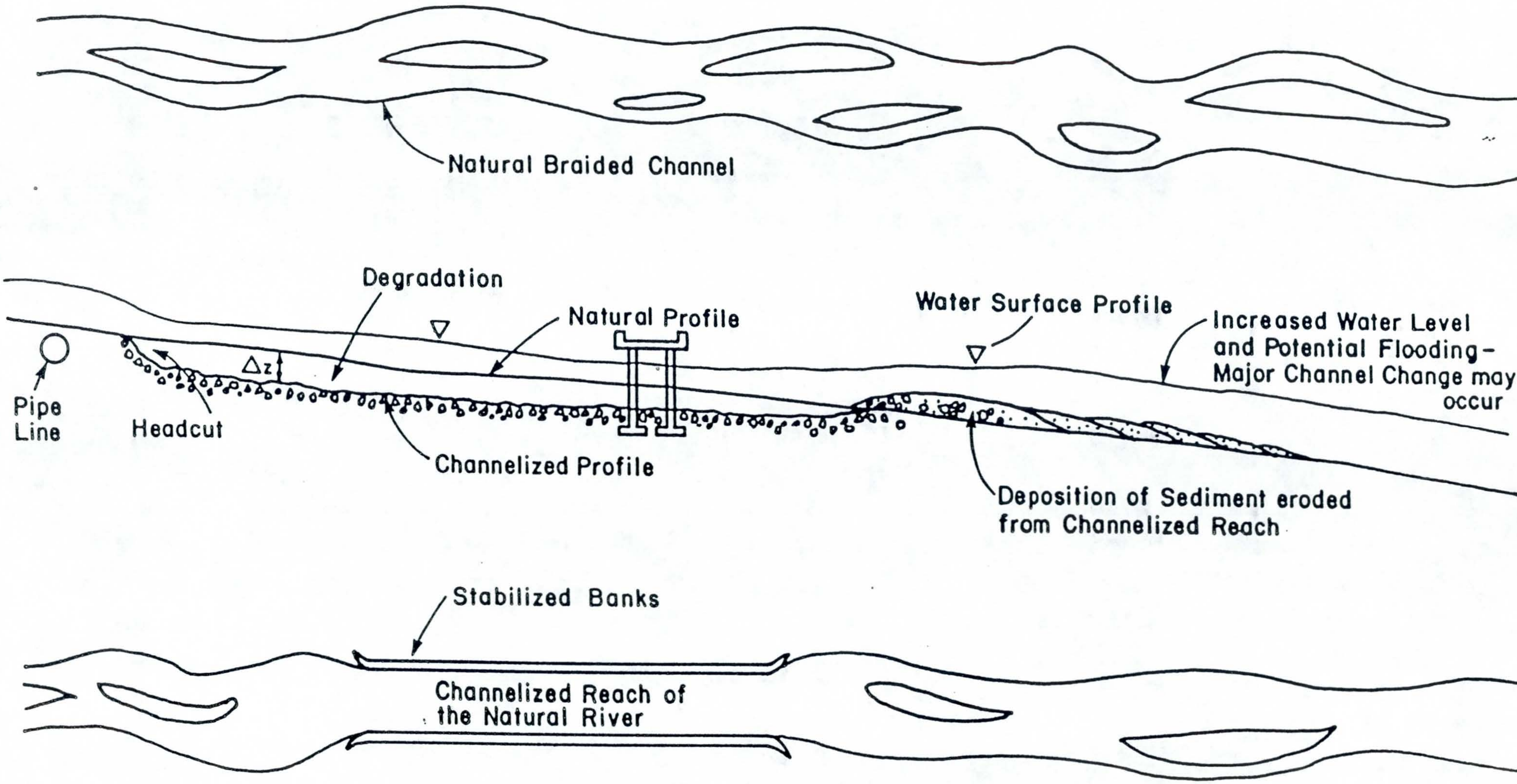
FOR STABILITY

1) Must stabilize upstream of drop structure
2) Transport approaching drop structure must equal transport downstream of drop — difficult!

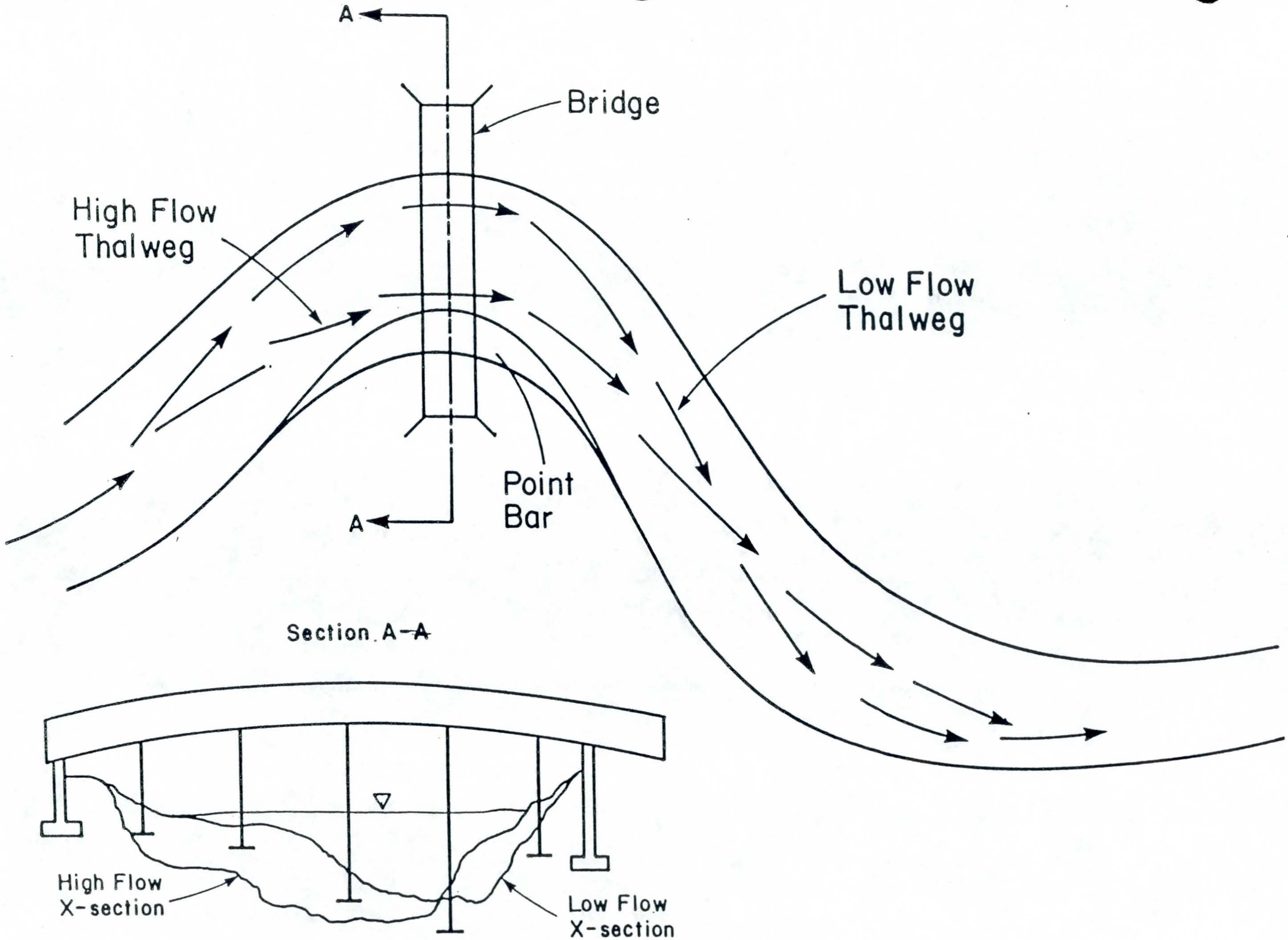
Definition Sketch of Drop Structure Flow Conditions



Potential Gravel Mining Impacts on a Bridge



Definition Sketch of the Flow Conditions Through a Channelized Reach



Definition Sketch of the Low and High Flow Channels
at a Bridge Crossing.

also is a good example of the effect of man's activities on a fluvial system. It is interesting to note that SLA's initial involvement with the Agua Fria came about because of litigation over the loss of the Indian School Road Bridge during the floods in 1980. After this disagreement was settled, we were contracted by Maricopa County to help solve a number of the river stability problems in this area of the Agua Fria. We have also had the opportunity to do a system analysis for the Los Angeles District of the Corps of Engineers on the Agua Fria that extended from Waddell Dam to the confluence with the Gila River.

B. Background Information

- Agua Fria watershed
- Hydrology
  - Flood history
  - Design floods
- Hydraulic characteristics
- Geomorphic characteristics
- Man's influence on the Agua Fria River
- River stability problems near Indian School Road

C. Analysis of Bridge Crossings

- Factors considered in low chord evaluation
  - Aggradation, wave height, superelevation, debris blockage
- Factors considered in pier and abutment evaluation
  - Degradation, local scour, sand wave depth, bend scour, contraction scour

D. Design of Bridge Protection Measures

- Guide bank and spur dike system
- Riprap protection of bridge piers
- Burial depths of piers

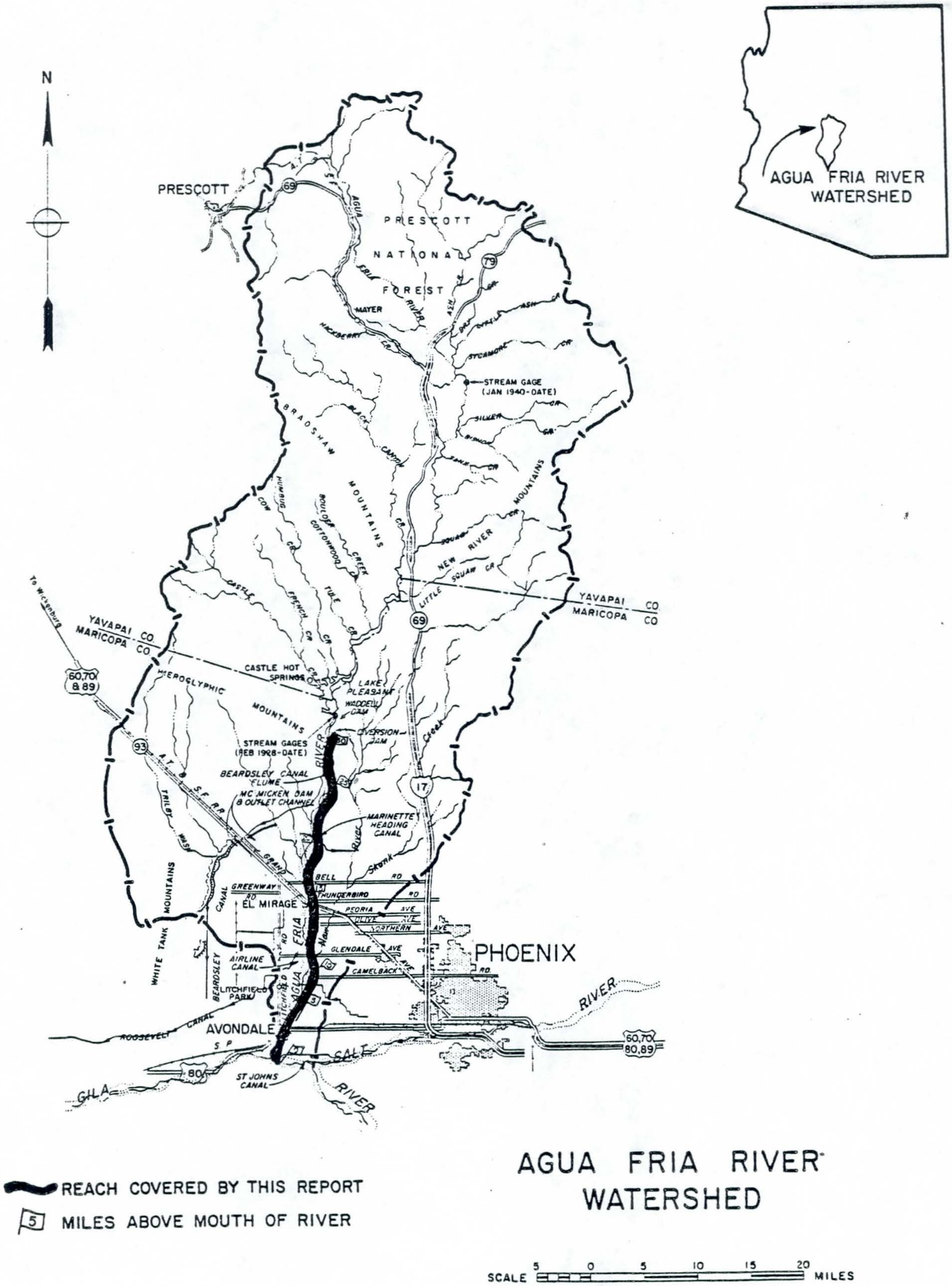
E. Design of Agua Fria Channelization

- Equilibrium slope analysis for long-term channel response

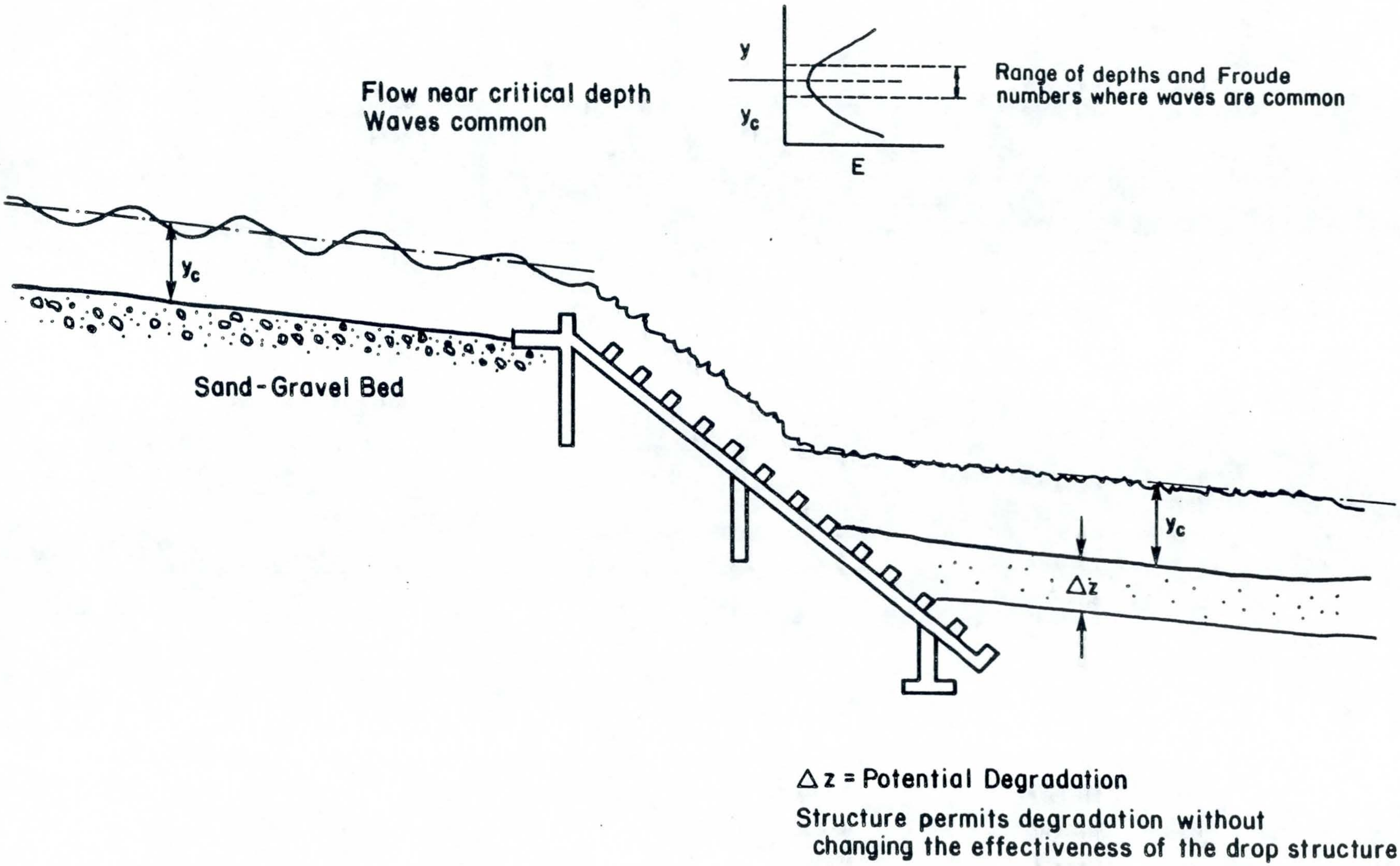
- Short-term channel response
- Toe-down requirements for protection
- Selection of bank protection method

F. Design of Agua Fria grade control

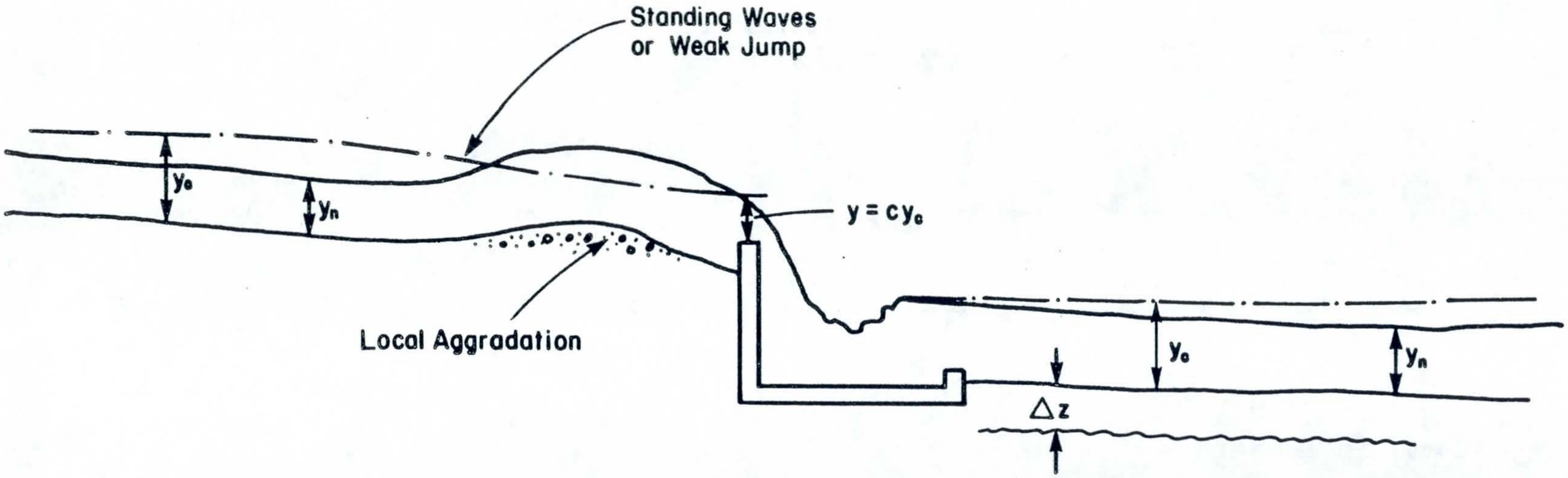
- Selection of grade control type
- Spacing of grade control
- Scour protection requirements



Agua Fria River Watershed.



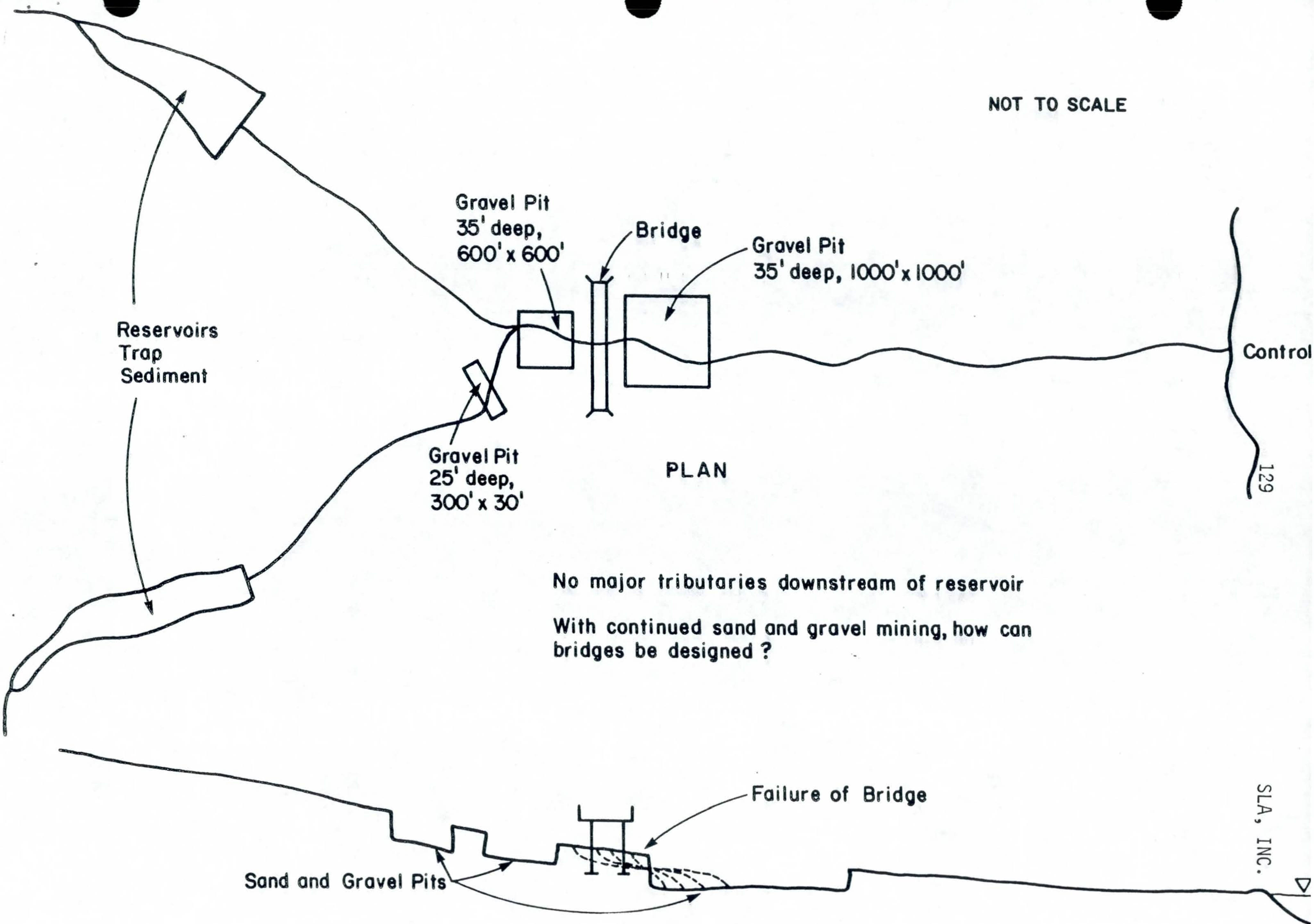
Drop Structure Design That Allows for Degradation



FOR STABILITY

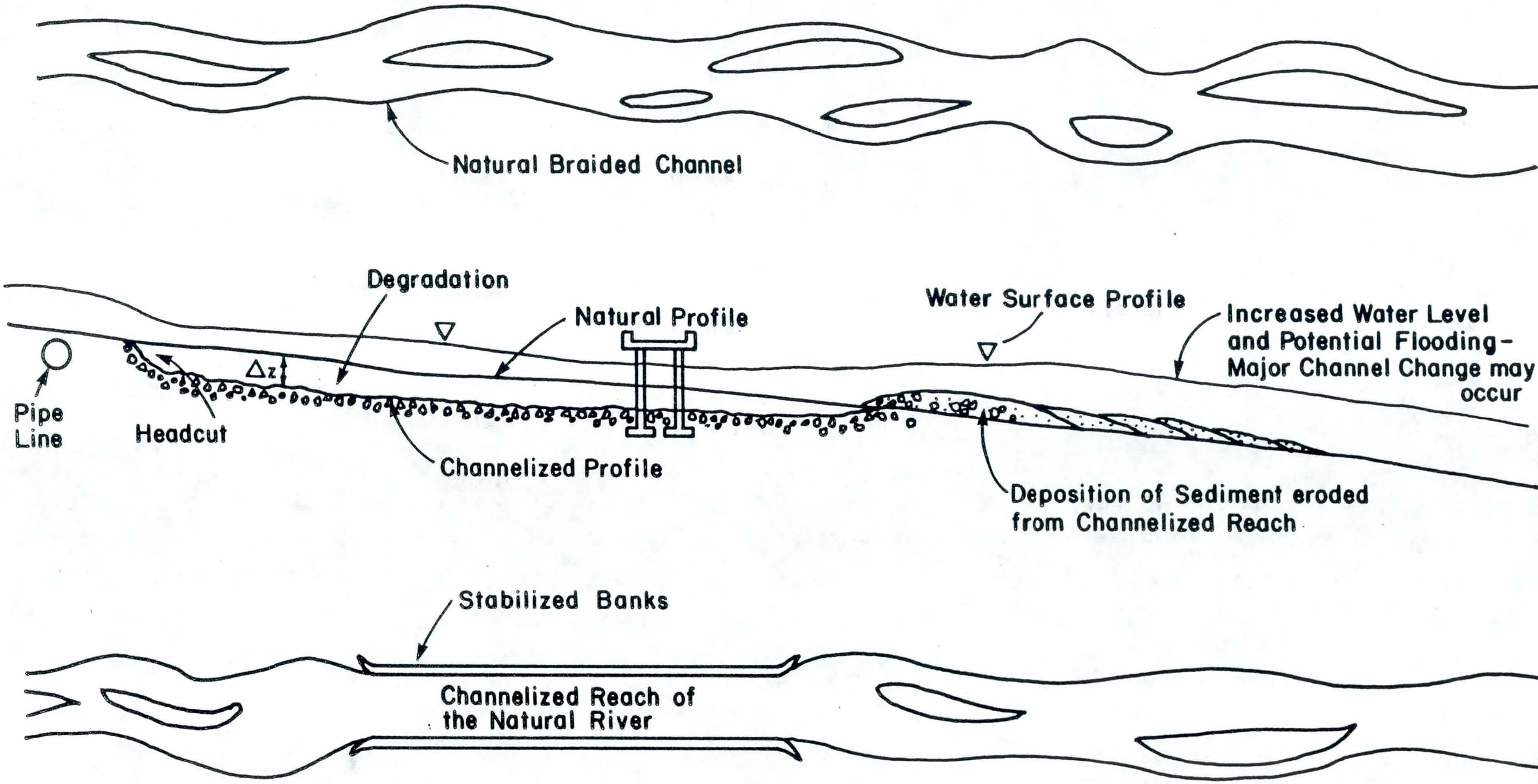
1) Must stabilize upstream of drop structure
2) Transport approaching drop structure must equal transport downstream of drop — difficult!

Definition Sketch of Drop Structure Flow Conditions

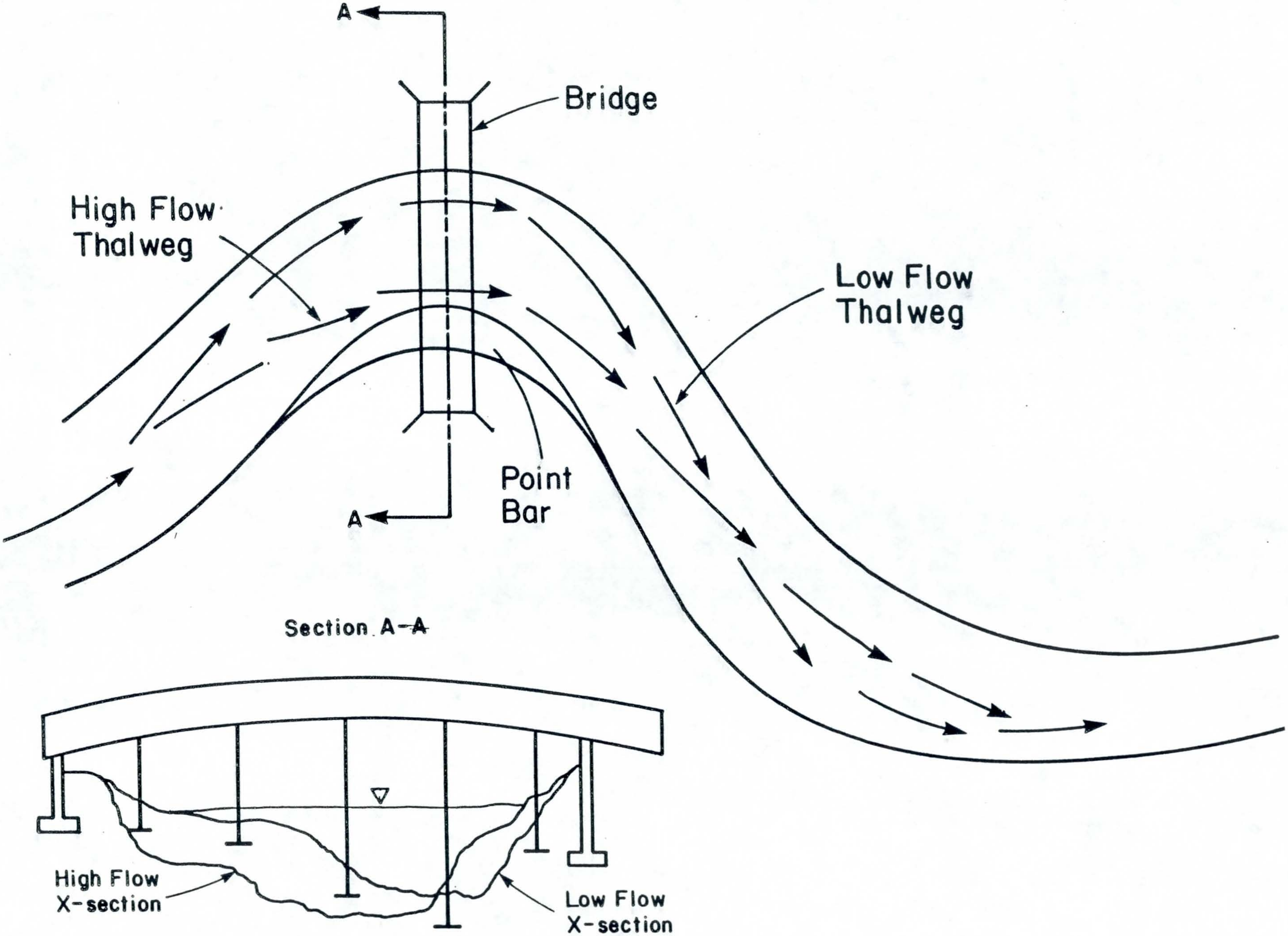


Potential Gravel Mining Impacts on a Bridge

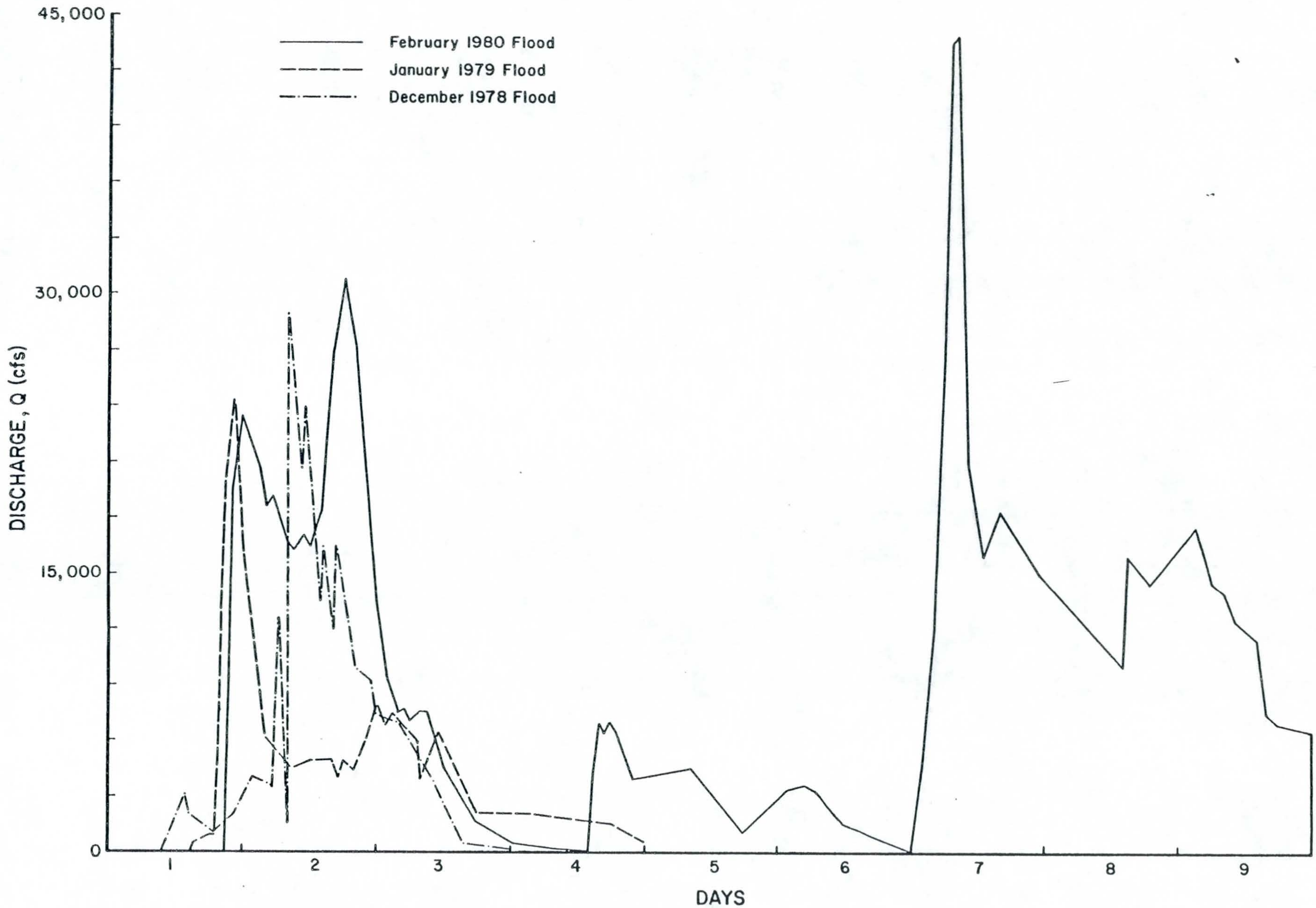
SLA, INC.



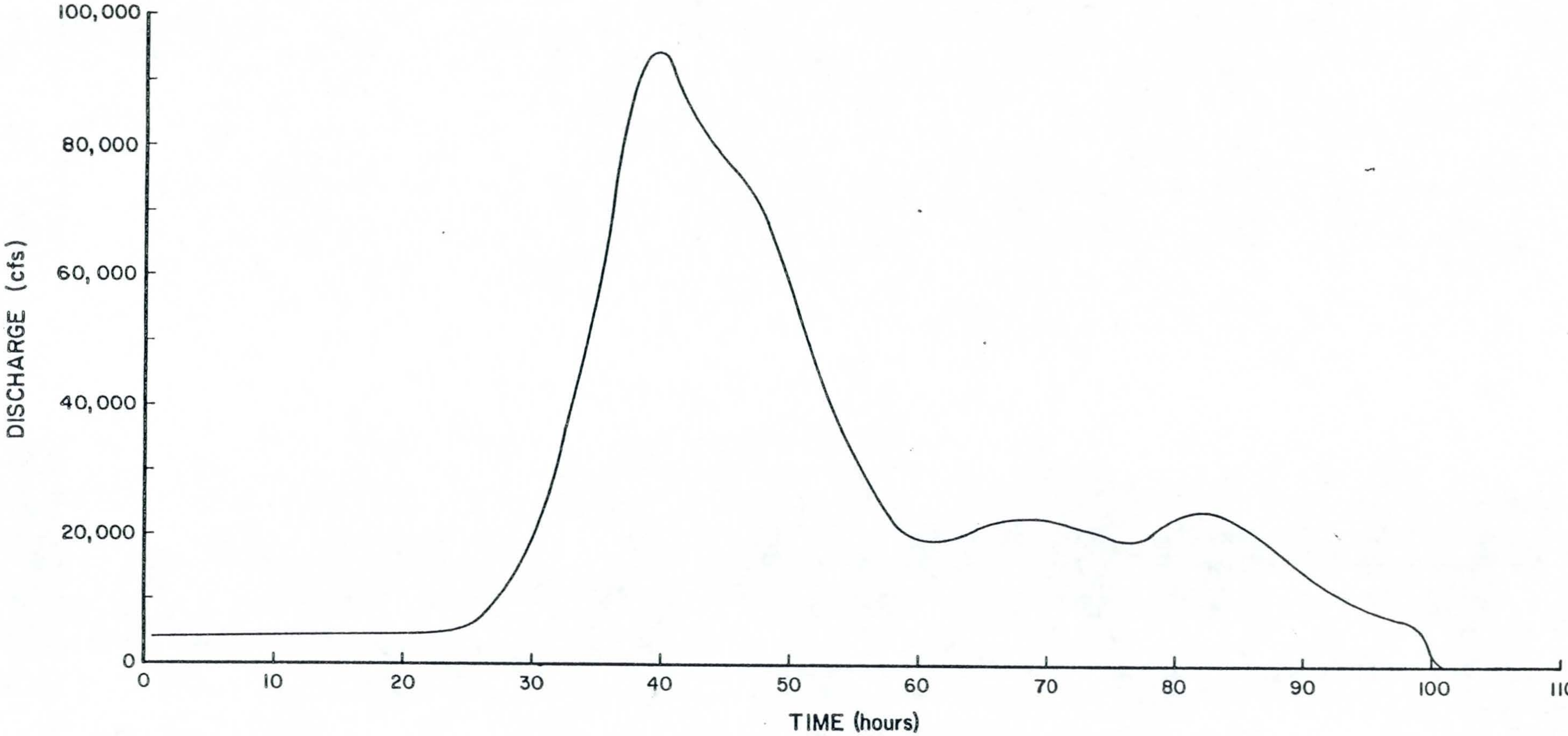
Definition Sketch of the Flow Conditions Through a Channelized Reach



Definition Sketch of the Low and High Flow Channels
at a Bridge Crossing.



Hydrographs for 1978, 1979 and 1980 flood events on the Agua Fria.



100-year flood on the Agua Fria below the New River confluence
as computed by Los Angeles Corps of Engineers, 3/2/81.

Pertinent Data of Existing Bridges.

	Camelback Road	ISRB	RID Flume	I-10 Bridge	SPRR Bridge	Buckeye Road Bridge
Pier width or diameter	4'	1' 8"	4'	3'4"	6'8"	3'
Pier length	*	60'	15'	*	27'	70'
Bottom of pier footing	947.4'	983'	990.5'	945.0'	914.3' to 922.2'	947.2'
Thalweg elevation	1,017.4'	1,000'	993.6'	966.0'	952.0'	952.0'
Skew of bridge piers to flow direction	5°	30°	0°	5°	10°	10°
Low chord	1,031.7'	1,014.4'	1,008.7'	991.0'	965.7'	968.1'

*Circular piers.

Average Flow Velocity, Hydraulic Depth, Effective Width, and Discharge for the 10-Year Flood Event.

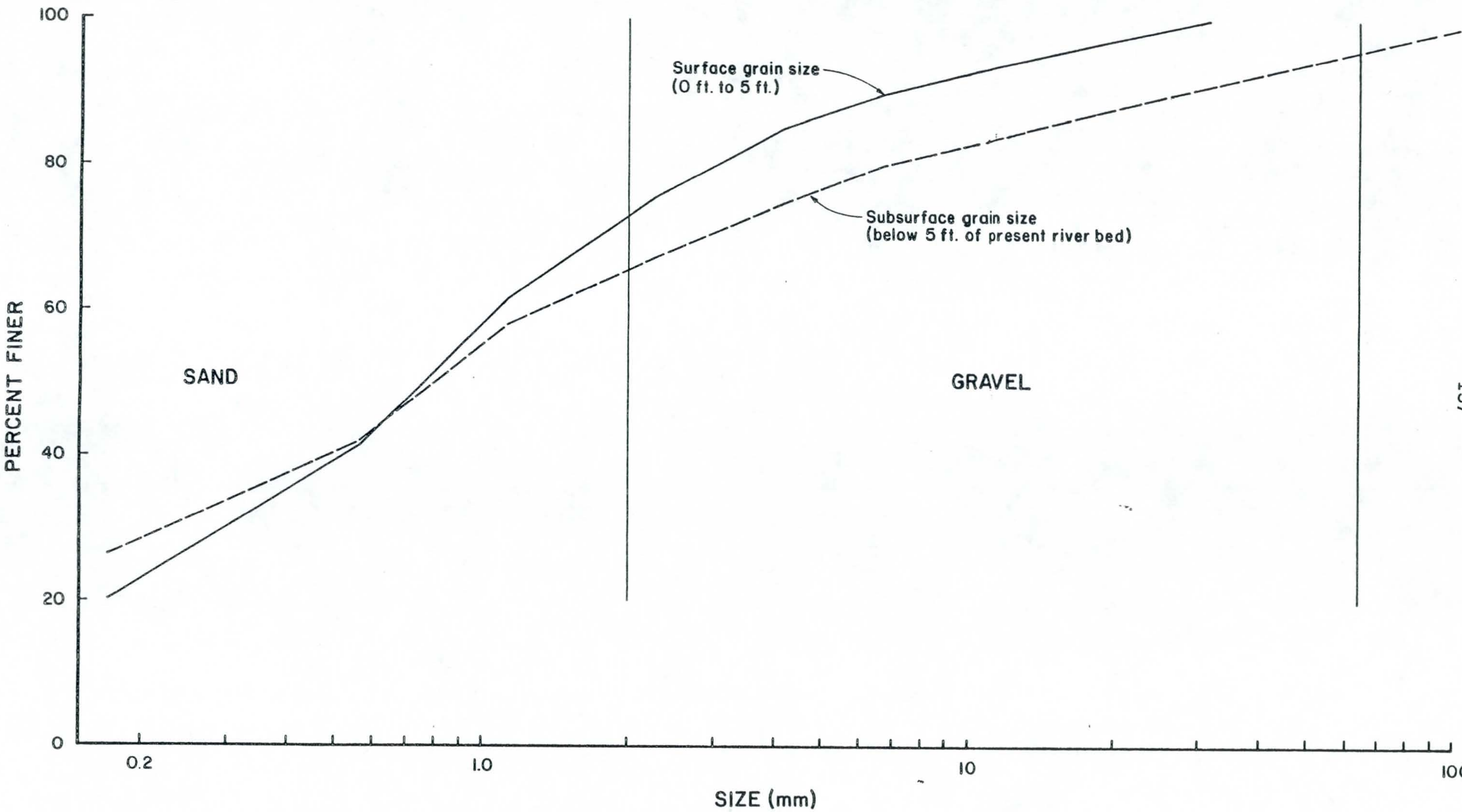
Reach	Left Flood Plain				Main Channel				Right Flood Plain			
	Velocity (fps)	Hydraulic Depth (ft)	Effective Width (ft)	Water Discharge (cfs)	Velocity (fps)	Hydraulic Depth (ft)	Effective Width (ft)	Water Discharge (cfs)	Velocity (fps)	Hydraulic Depth (ft)	Effective Width (ft)	Water Discharge (cfs)
1	1.09	0.65	554	396	4.63	3.29	1,905	28,981	1.03	0.75	534	410
2	0.68	0.37	367	91	4.66	3.45	1,913	30,804	0.72	0.32	224	51
3	0.33	0.16	494	27	7.06	6.28	670	29,669	0.76	0.41	1,240	390
4	1.89	1.55	455	1,327	5.75	5.99	833	28,668	0.14	0.03	930	4
5	0.53	0.40	265	56	6.35	5.34	840	28,463	1.24	0.51	1,428	907
6	0.26	0.20	123	7	5.56	4.54	1,125	28,377	0	0	0	0
7	0	0	0	0	5.63	4.78	1,041	28,000	0	0	0	0
8	0.92	0.64	527	311	5.05	4.06	1,239	25,390	2.18	1.26	759	2,085
9	1.64	1.11	1,248	2,267	5.04	3.86	1,271	24,731	0.36	0.31	15	2
10	1.16	0.77	847	756	5.26	4.07	1,225	26,244	0	0	0	0

Reach 1 Glendale Avenue to Confluence with New River.
Reach 2 Confluence with New River to Indian School Road Bridge.
Reach 3 Indian School Road Bridge to the Roosevelt Irrigation District Flume.
Reach 4 Roosevelt Irrigation District Flume to Thomas Road.
Reach 5 Thomas Road to 1,500 feet upstream of I-10.
Reach 6 1,500 feet upstream of I-10 to Van Buren.
Reach 7 Van Buren to Buckeye Road.
Reach 8 Buckeye Road to Lower Buckeye Road.
Reach 9 Lower Buckeye Road to Broadway Road.
Reach 10 Broadway Road to the Confluence with the Gila River.

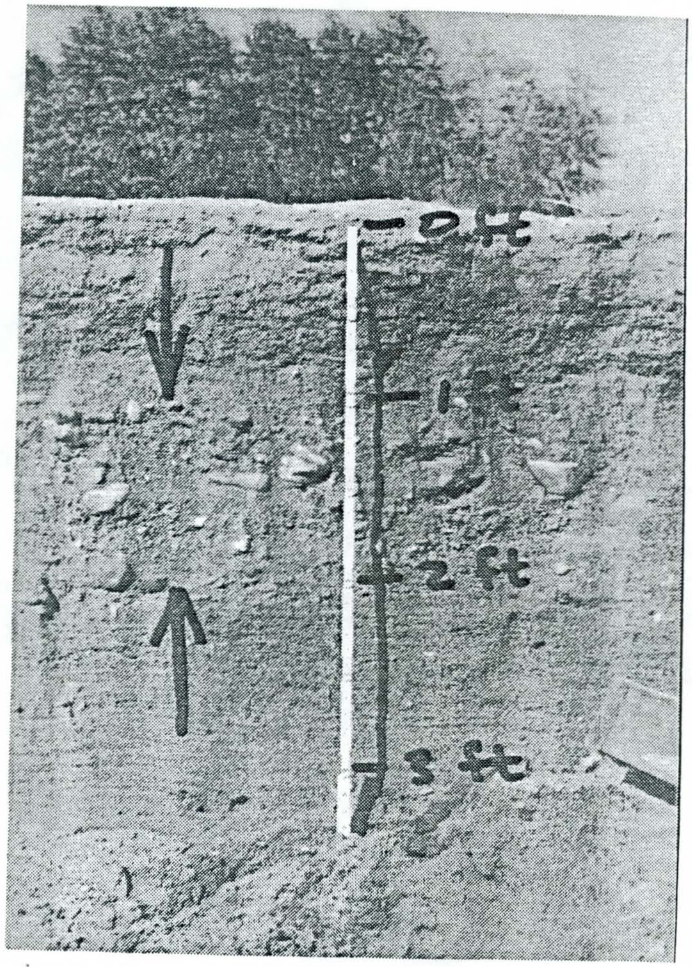
Average Flow Velocity, Hydraulic Depth, Effective Width, and Discharge for the 100-Year Flood Event.

	Left Flood Plain				Main Channel				Right Flood Plain			
Reach	Velocity (fps)	Hydraulic Depth (ft)	Effective Width (ft)	Water Discharge (cfs)	Velocity (fps)	Hydraulic Depth (ft)	Effective Width (ft)	Water Discharge (cfs)	Velocity (fps)	Hydraulic Depth (ft)	Effective Width (ft)	Water Discharge (cfs)
1	2.18	1.58	987	3,399	5.52	4.55	2,135	53,630	3.51	3.45	2,723	32,970
2	2.53	2.40	896	5,442	6.36	6.44	2,016	82,587	2.52	2.39	956	5,758
3	2.68	2.23	1,349	8,065	9.56	10.20	687	67,031	3.57	3.41	1,478	17,989
4	3.48	2.96	1,618	16,671	8.03	9.61	835	64,417	2.31	2.09	2,468	11,913
5	2.33	2.44	659	3,749	7.24	8.27	907	54,324	3.15	3.03	3,539	33,779
6	2.78	3.12	1,167	10,119	8.15	8.23	1,192	79,952	1.52	1.38	149	313
7	2.36	2.58	2,167	13,193	7.60	9.25	1,073	75,448	1.86	2.02	362	1,360
8	2.75	3.02	1,045	8,675	7.50	7.18	1,244	66,982	3.99	3.88	913	14,129
9	3.81	3.56	2,053	27,846	7.20	6.60	1,284	61,029	1.02	1.09	113	126
10	3.29	2.39	2,606	20,491	7.37	6.87	1,254	63,488	2.52	1.04	1,916	5,021

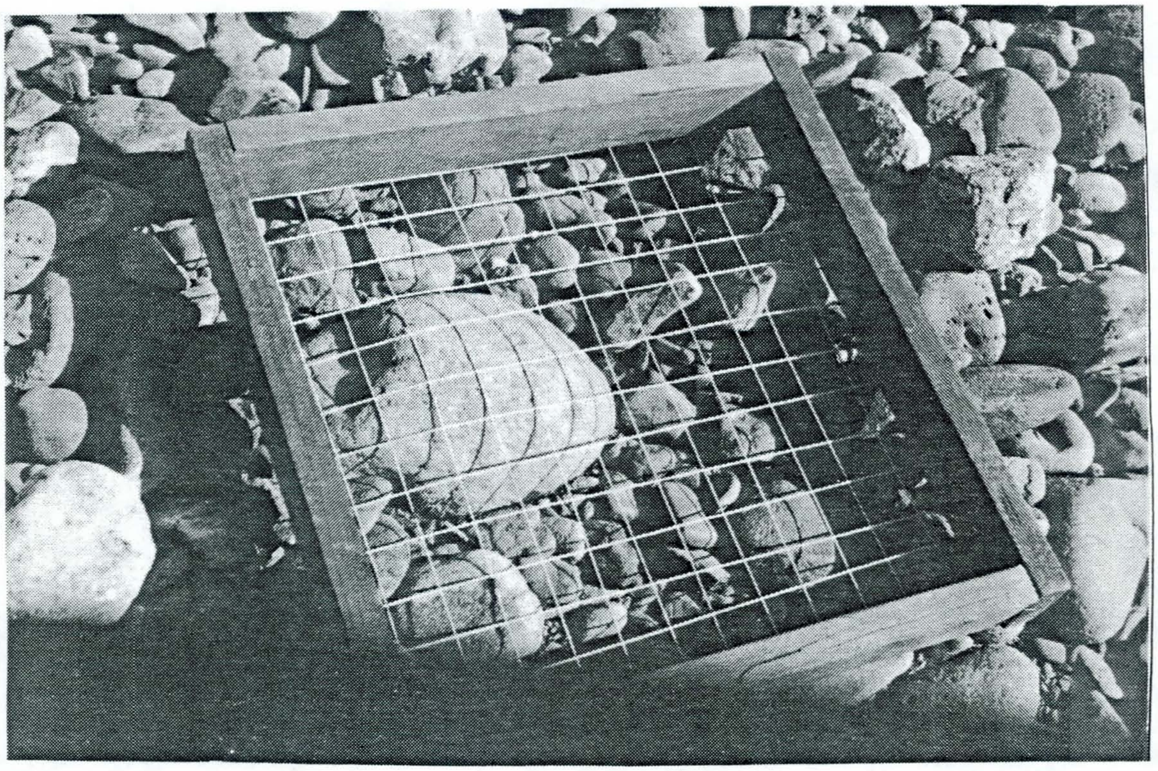
Reach 1 Glendale Avenue to Confluence with New River.
Reach 2 Confluence with New River to Indian School Road Bridge.
Reach 3 Indian School Road Bridge to the Roosevelt Irrigation District Flume.
Reach 4 Roosevelt Irrigation District Flume to Thomas Road.
Reach 5 Thomas Road to 1,500 feet upstream of I-10.
Reach 6 1,500 feet upstream of I-10 to Van Buren.
Reach 7 Van Buren to Buckeye Road.
Reach 8 Buckeye Road to Lower Buckeye Road.
Reach 9 Lower Buckeye Road to Broadway Road.
Reach 10 Broadway Road to the Confluence with the Gila River.



Surface and subsurface grain size distributions of the Agua Fria near Indian School Road Bridge.

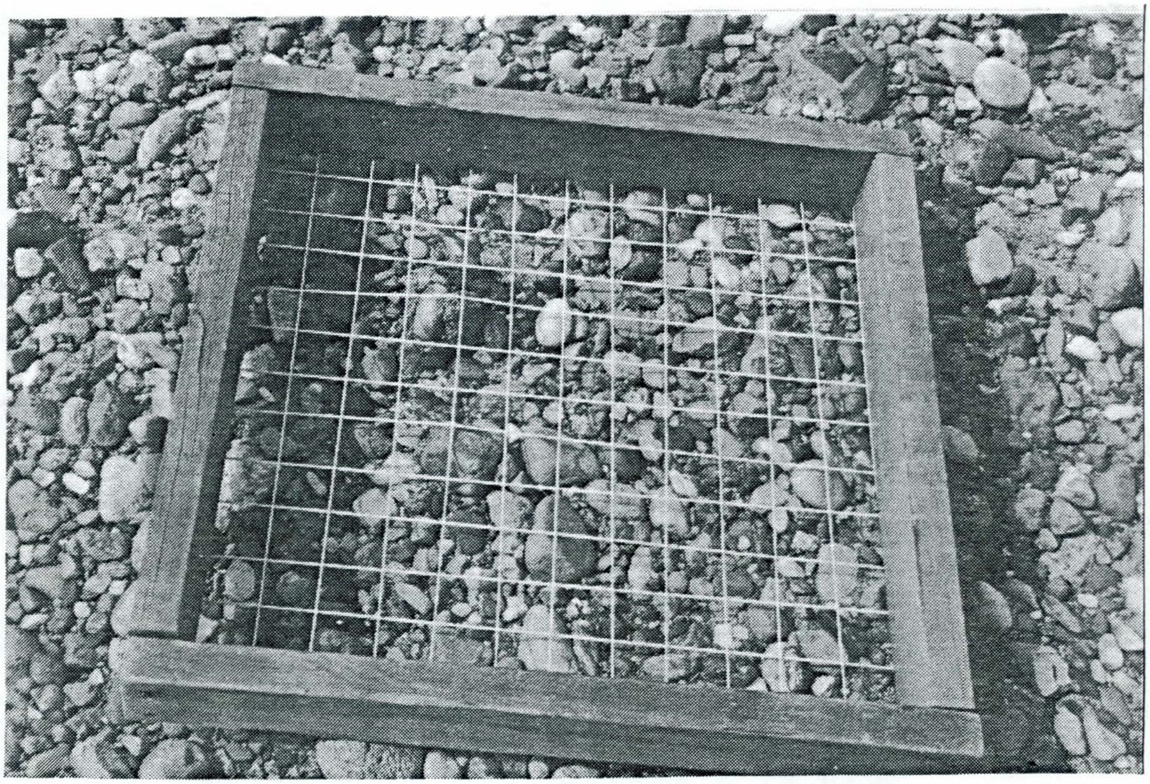


Gravel layer below the river bed of Agua Fria River approximately 800 feet downstream of Indian School Road Bridge.

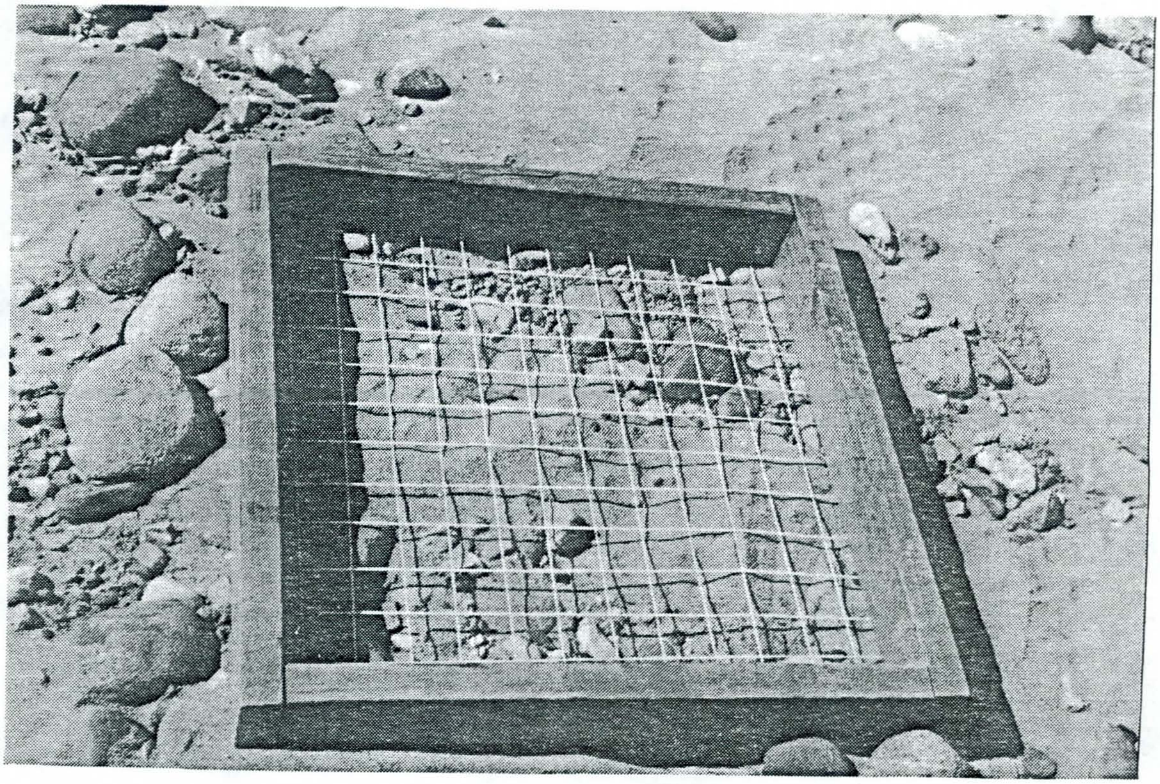


* The square is two inches on each side.

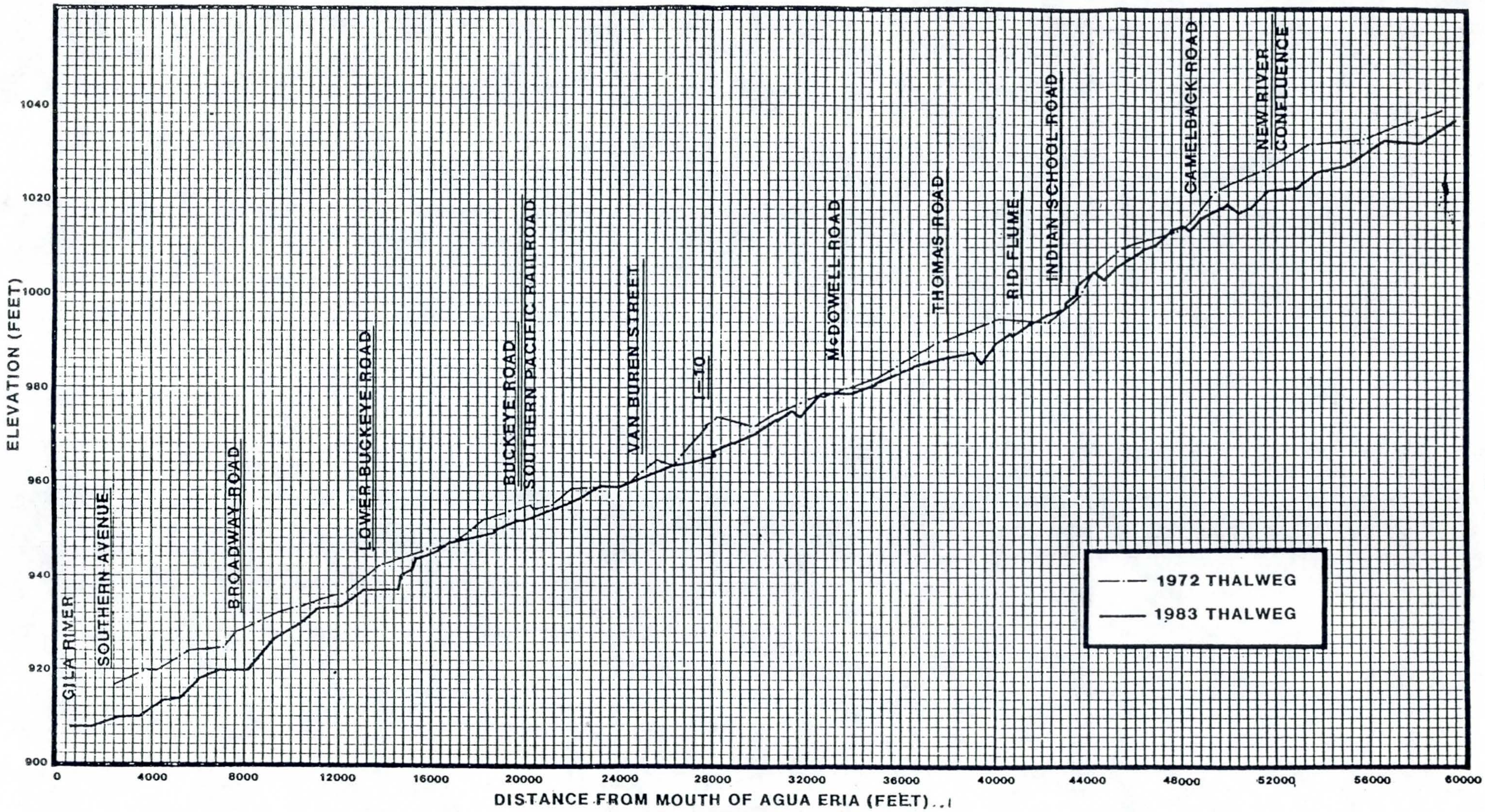
Bed material of the Agua Fria River
near Waddell Dam.



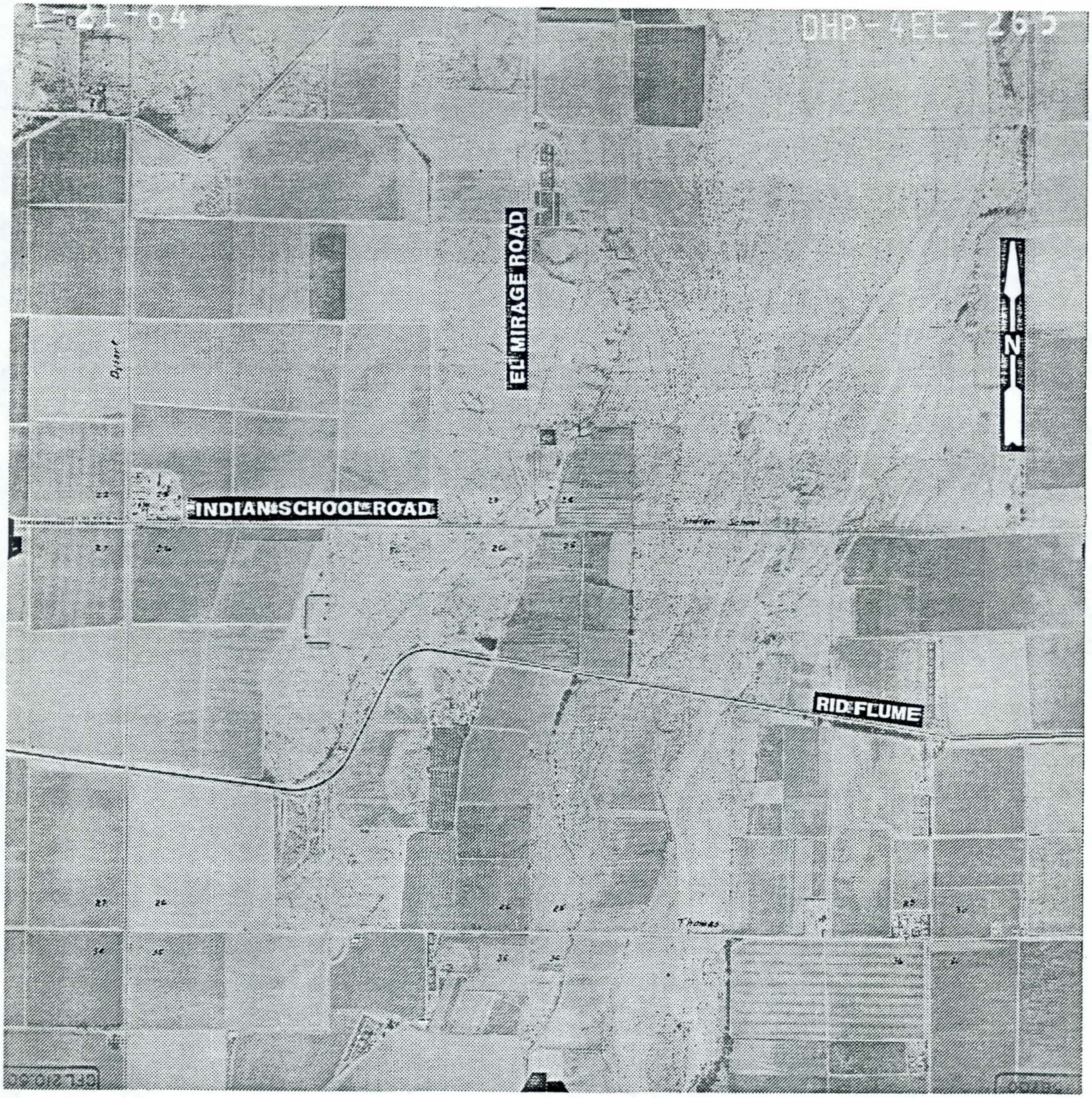
Bed material of the Agua Fria River
near Grand Avenue.



River bed materials of the Agua Fria River upstream of the confluence with New River.



COMPARISON OF 1972 AND 1983 THALWEG ELEVATIONS IN THE LOWER AGUA FRIA



1964 aerial photo of the Agua Fria River near Indian School Road.

1980 aerial photo of the Agua Fria River near Indian School Road.

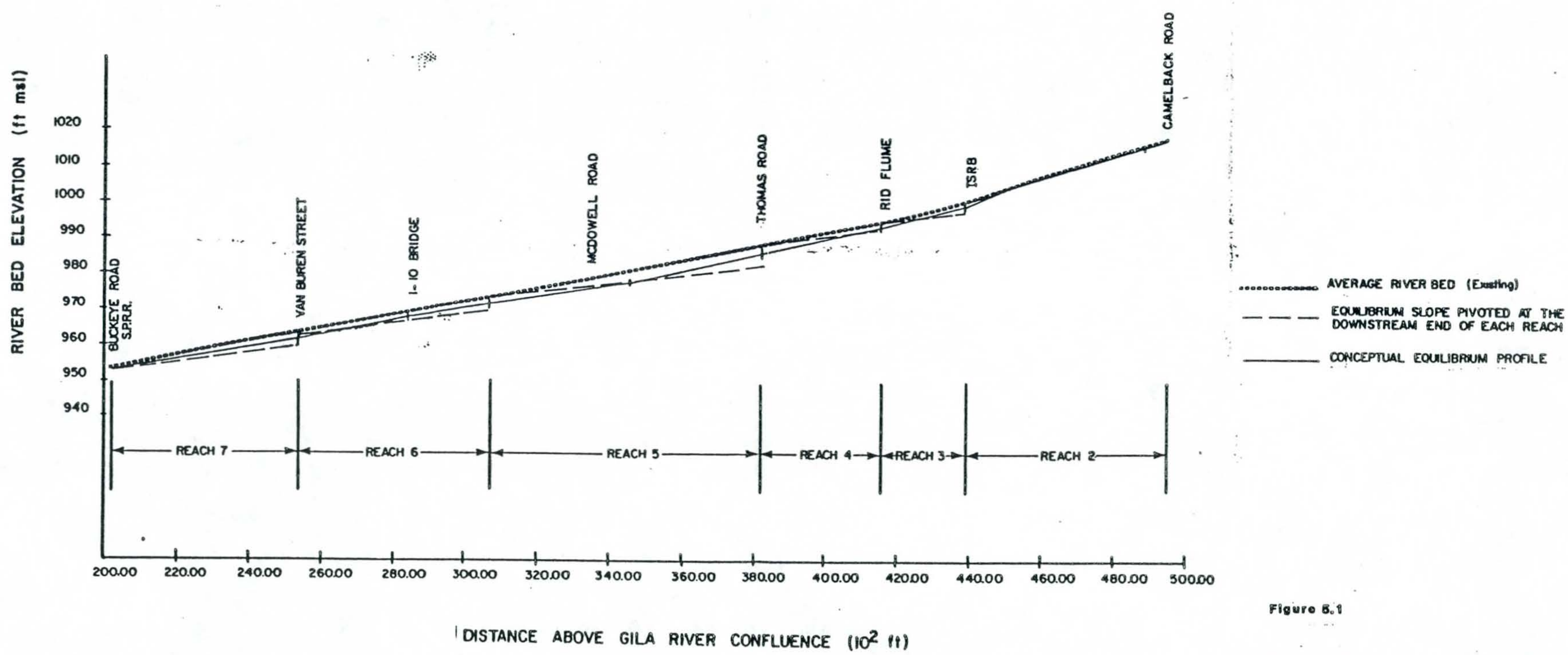


Figure 5.1

Comparison of existing and equilibrium slope profiles between Buckeye Road and Camelback Road

APPLICATION OF THE MULTI-LEVEL ANALYSIS APPROACH

(Case History)

LECTURE 10

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July 1986

LECTURE 10: Application of the Multi-Level Analysis Approach (Case History)

OBJECTIVE: This lecture is intended to provide an example application of the multi-level analysis approach. The Canada del Oro Flood-Control Project, an actual case history has been selected for presentation purposes.

I. GENERAL INFORMATION

The Canada del Oro Flood-Control Project is situated along a reach of the Canada del Oro Wash that lies both within and adjacent to the Town of Oro Valley, which is located within Pima County, Arizona, approximately 10 miles north of the City of Tucson (see Figure). The feasibility studies for the project began in late 1980, and construction of the project was completed around the middle of 1984. The necessity for such a project was the direct result of man's encroachment into a fluvial environment that conveys large quantities of sediment and that is highly susceptible to rapid changes in cross-sectional geometry. If nothing were done to mitigate potential flood and erosion hazards, highly unstable and hazardous conditions would remain a constant threat to many of the residents of the Town of Oro Valley. The flood-control project included mitigating the impacts of tributary flows, as well as the impacts from floods along the Canada del Oro Wash itself.

II. PRINCIPLE GOALS OF STUDY

A. Prepare Report Addressing Flood and Erosion Problems

B. Provide Concept Plans Which Offer Solutions to Existing Flood and Erosion Problems

III. PRIMARY SCOPE OF STUDY

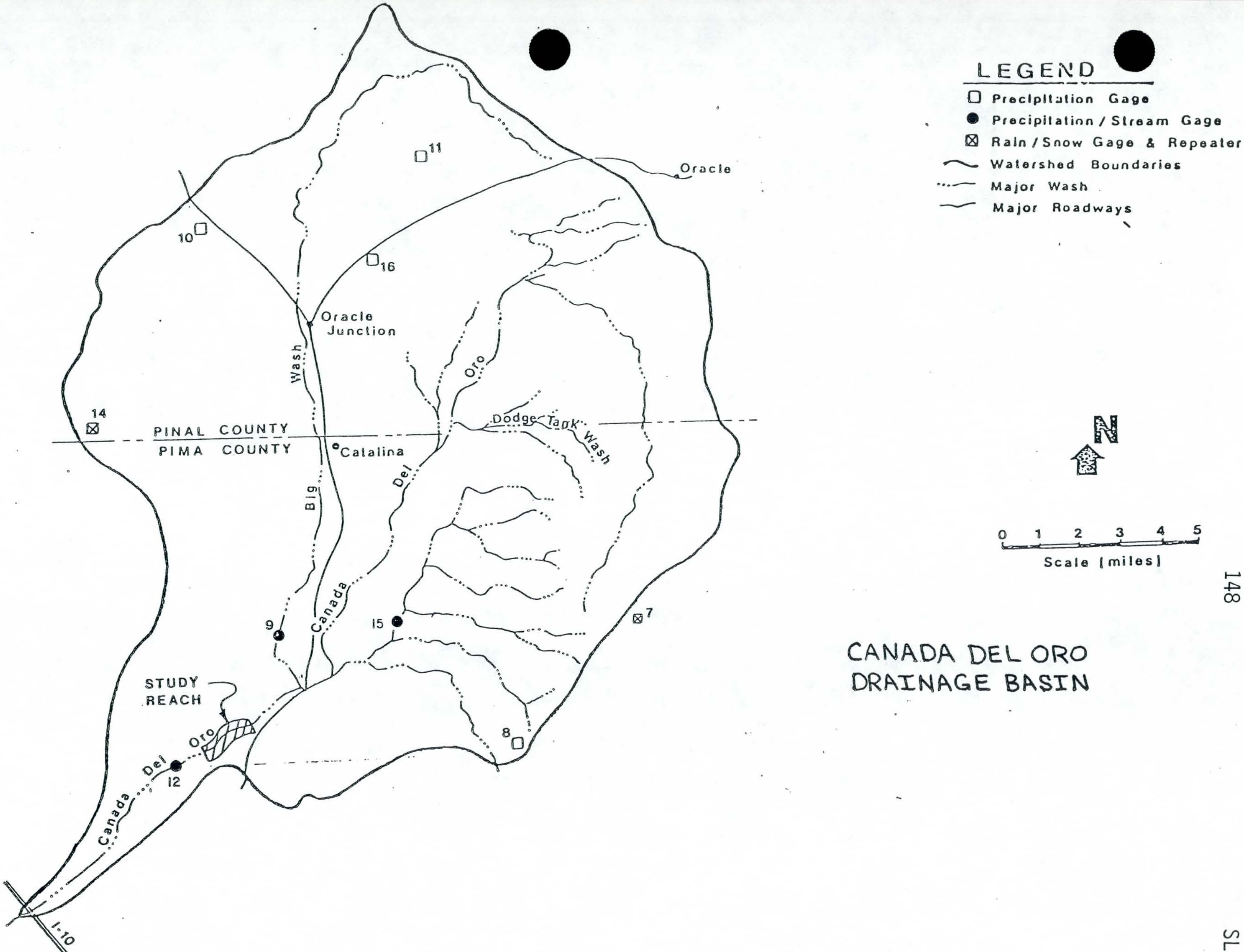
A. Review Flood History, Hydrology, Hydraulics, Erosion/Sedimentation and Flood-Damage Information

B. Determine Existing and Potential Flooding Problems for 25-, 50-, 100-, and 500-Year Floods

C. Assess Long-Term Erosion/Sedimentation Problems in the Study Area

D. Evaluate Potential Erosion/Sedimentation Problems for "As-Is" Conditions During the Passage of a Single-Event, 100-Year Flood Through the Study Area

E. Formulate and Recommend Alternative Flood-Control Plans



CANADA DEL ORO DRAINAGE BASIN

F. Through Hydraulic Analyses, Compare Improved Conditions to "As-Is" Conditions for the Various Alternatives

G. Provide Preliminary Analysis of Erosion/Sedimentation Aspects of the Various Alternatives

H. Evaluate the Alternative Plans

I. Recommend a Specific Flood-Control Plan

J. Perform Detailed Hydraulic and Erosion/Sedimentation Analysis of the Recommended Flood-Control Plan

K. Prepare a Final Report Documenting the Study Results

IV. LEVEL I - QUALITATIVE ANALYSIS

A. Flood History (Since 1950)

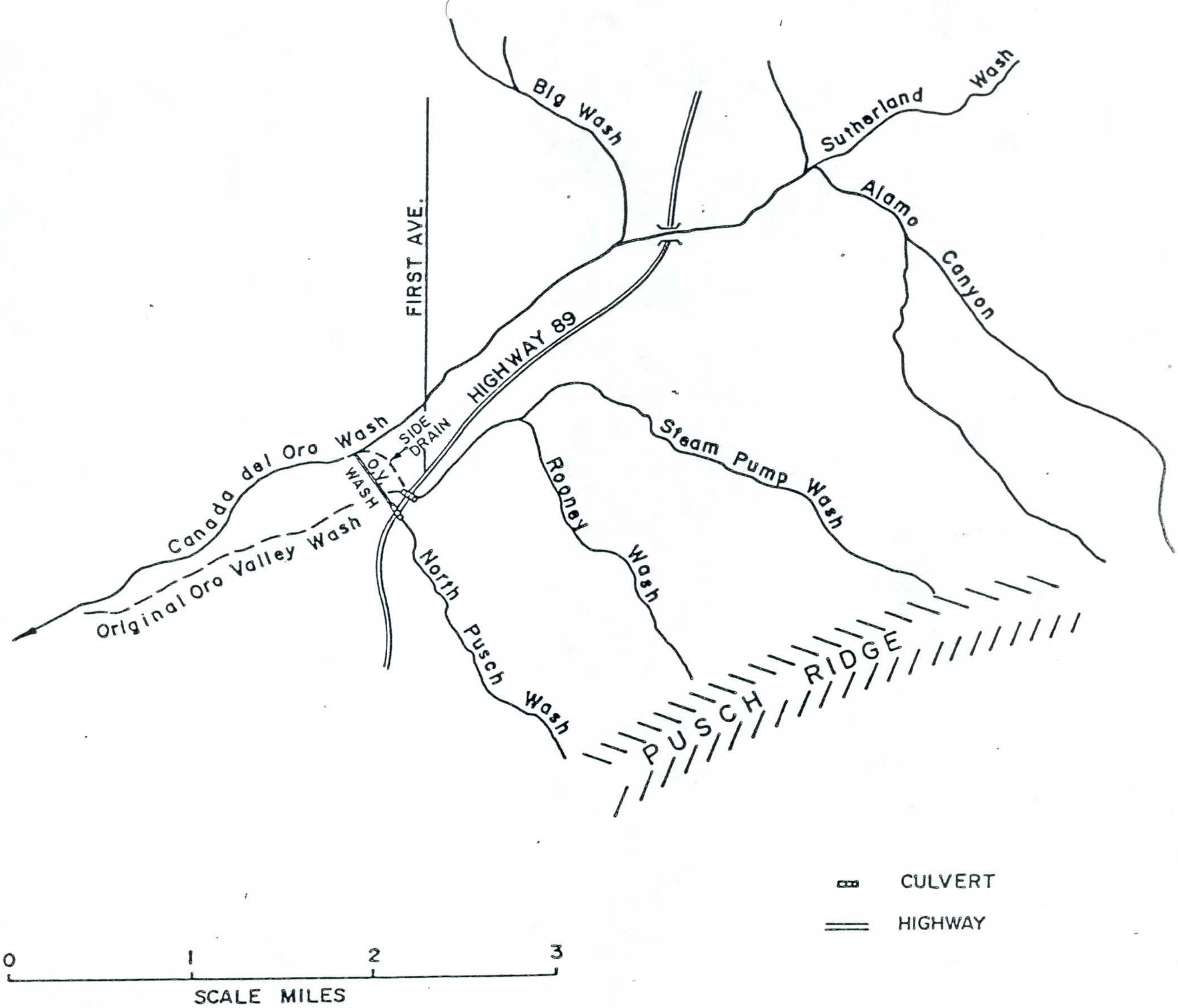
- Limited Flood Information
  - Earliest Recorded Flood in 1950
  - Largest Recorded Flood Also in 1950
- Data on Flood History Available From USGS (see Figure)
  - "Statistical Summaries of Arizona Streamflow"
- Sources of Flooding
  - Canada del Oro Wash (Includes Big Wash)
  - Combined Flows From Stream Pump, Rooney, and North Pusch Washes
- Observations of Flooding
  - Significant Sediment Transport
  - High Degree of Erosion and Sedimentation
  - Attenuation of Flood Peaks Due to Overbank Storage of Floodwaters
- Location of Tributary System (see Figure)

Significant Peak Flows on the Canada del Oro Wash.

Peak Discharge (cfs)	Date
17,000*	21 July 1950
12,000*	1 Sept 1961
8,000*	10 Sept 1964
2,290	22 Dec 1965
13,900	20 Dec 1967
4,200	17 Aug 1971
3,750	19 Oct 1972
7,700	20 July 1974
2,220	5 Sept 1976
4,500	9 Aug 1977
11,000**	19 July 1980

* Estimated by USGS on basis of data collected at North First Avenue.

** Measured on Big Wash tributary upstream of Oro Valley.



TRIBUTARY SYSTEM.

B. Channel History (Since 1941)

- Natural State Exhibited Wide, Braided Character (2,500 Feet in Width) Prior to 1960

- Confinement of Wash (400 Feet in Width) Near Oro Valley Since 1960 Has Led to Channel Instability Via Lateral Migration of the Banks

- Two Distinct Channel Morphologies Exist in the Study Reach (Braided and Confined)

IV. LEVEL II - QUANTITATIVE ANALYSIS

A. Hydrology

° Flood-Peak Estimates (see Figure)

- Canada del Oro, Modified USGS/FEMA

- Tributaries, Pima County Hydrology Method

° Hydrographs (see Figure)

- Pima County Synthetic Hydrograph Procedure Modified by SLA

B. Hydraulics (Existing Conditions)

° Channel Characteristics

- Geometric Relationships From Power Equations (see Figure)

- Channel Slope (0.0093)

- Bed and Bank Sediment Composition (see Figures)

- Manning's Roughness Coefficient (0.036 for Natural Conditions and 0.025 for Confined/Channelized Conditions)

° Hydraulic Parameters (see Figure)

- Normal Depth of Flow

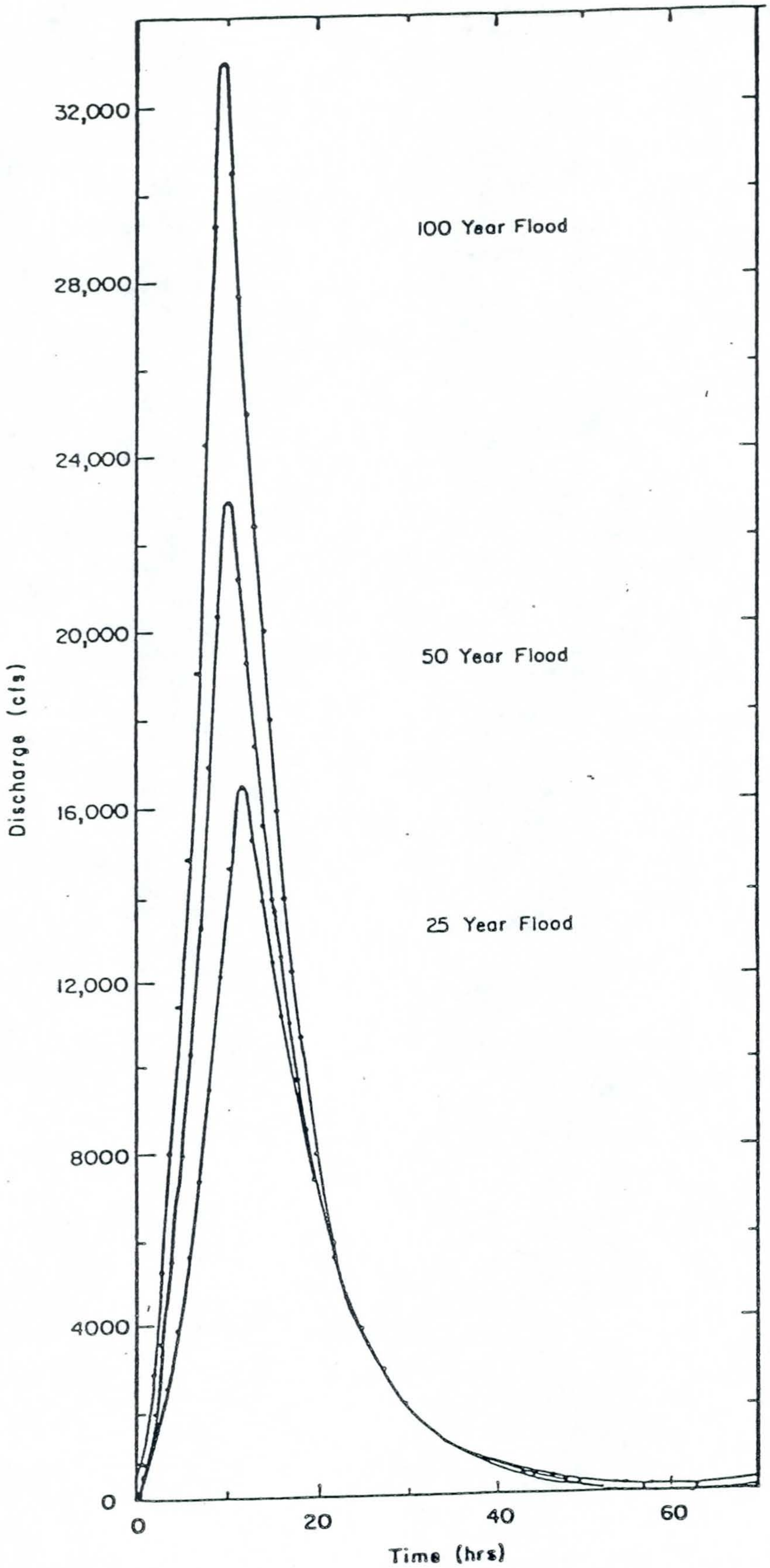
- Critical Depth of Flow

- Velocity of Flow at Normal Depth

- Froude Number at Normal Depth

FLOOD-PEAK ESTIMATES (cfs.)
CANADA DEL ORO WASH AND TRIBUTARIES,
VICINITY OF TOWN OF ORO VALLEY, ARIZONA

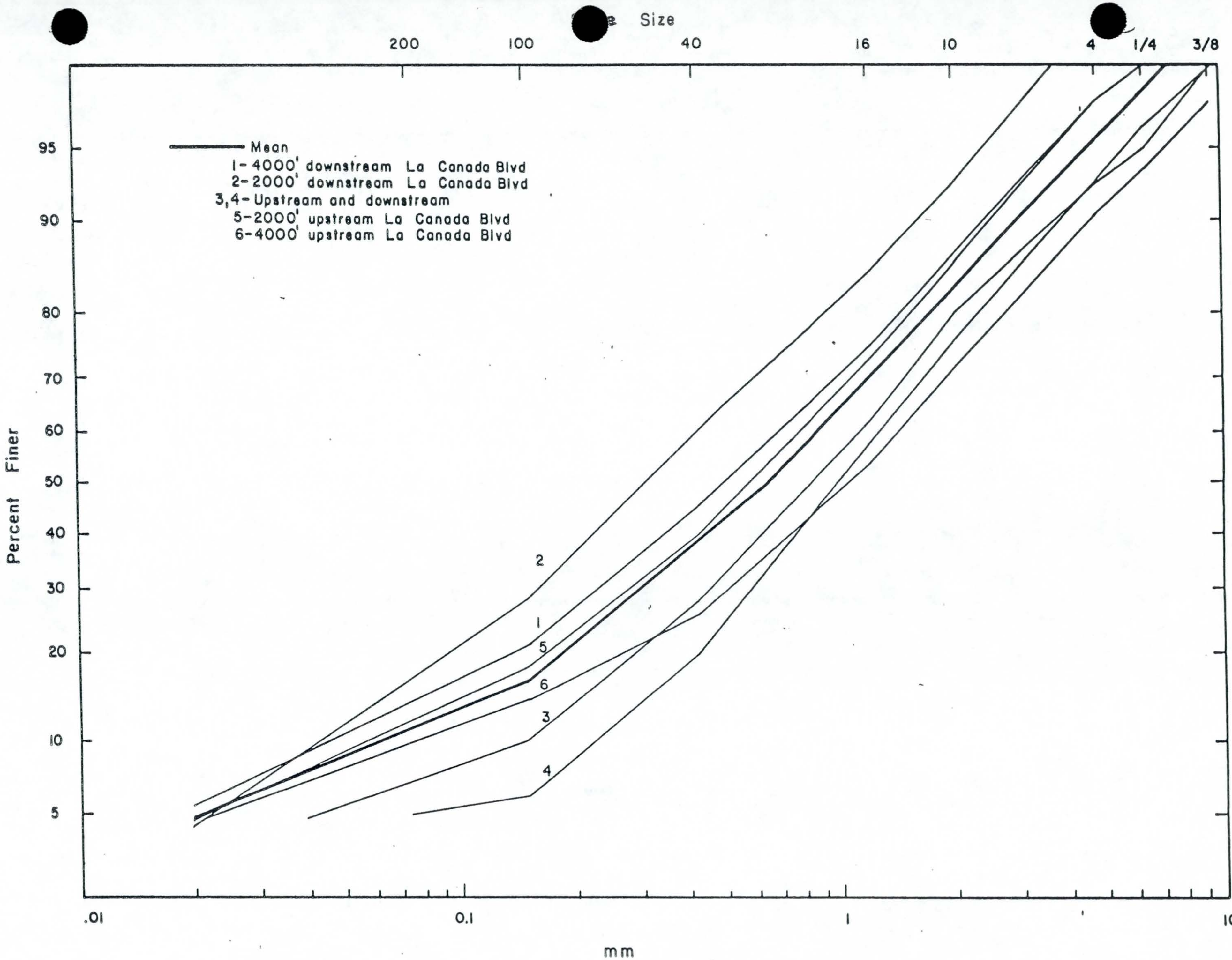
Name of Wash	Drainage Area (Square Miles)	Recurrence Interval (Years)				
		2	10	25	100	500
Canada Del Oro	250	3,300	10,500	16,500	33,000	55,400
Steam Pump	2.35	680	1,800	2,700	4,500	8,700
Rooney	1.93	590	1,560	2,340	3,900	7,500
North Pusch	1.22	420	1,120	1,680	2,800	5,400



Synthetic flood hydrographs -- Canada del Oro Wash.

Geometric Relationships.

Relationship	Upstream of Oro Valley		Through Oro Valley		Natural Conditions	
	Coefficient	Exponent	Coefficient	Exponent	Coefficient	Exponent
$A = a_1 d^{b_1}$	103	2.19	50.3	2.44	18.29	2.992
$P = a_2 d^{b_2}$	399	0.833	90.7	1.52	1468	0.2496
$T = a_3 d^{b_3}$	399	0.833	90.6	1.52	1467	0.2496



Bed material samples for La Canada reach near Oro Valley.

156

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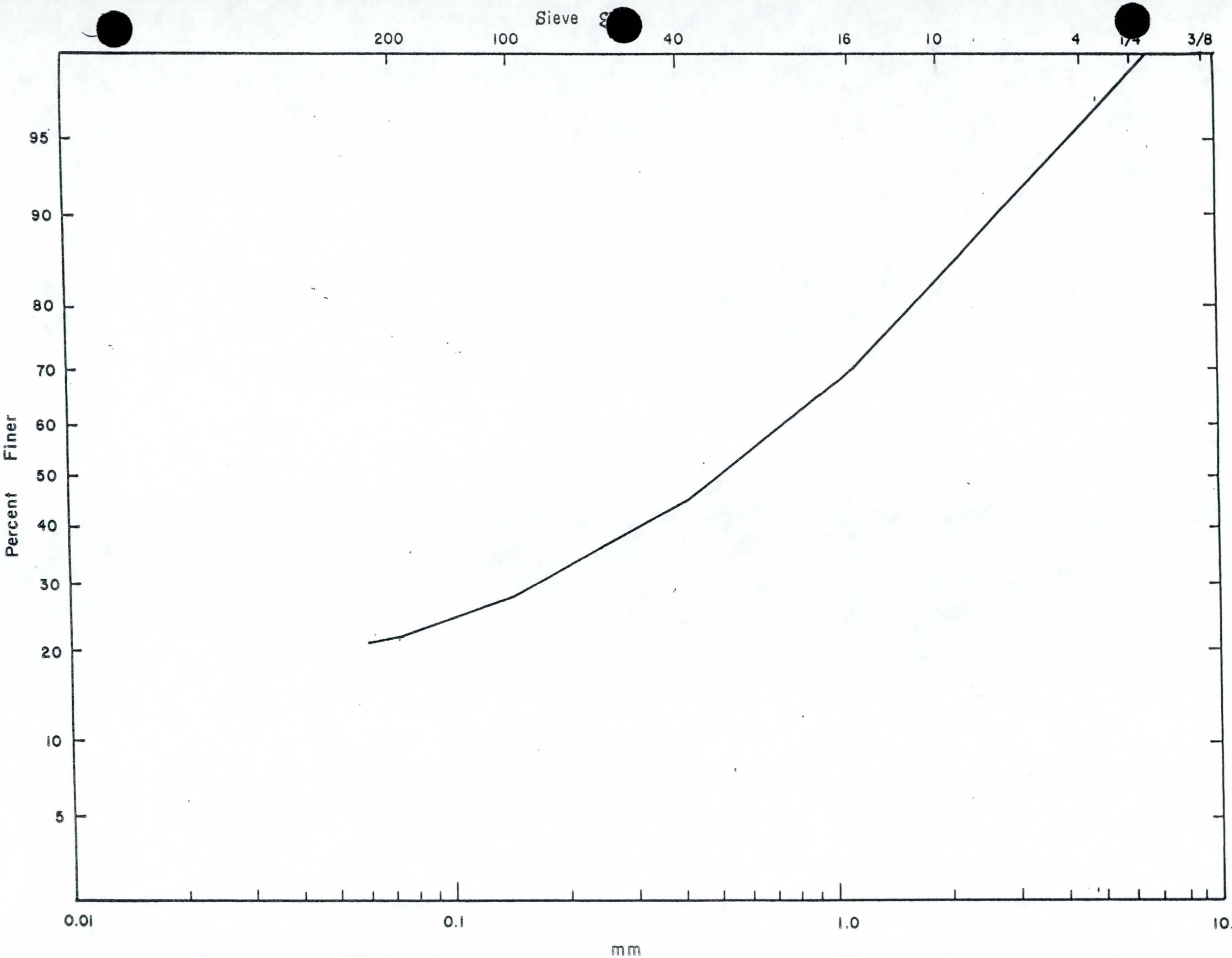

Average bank material sample for La Canada reach: near Oro Valley

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Hydraulic Parameters
(Q = 33,000 cfs)

Parameter	Upstream of Oro Valley	Oro Valley	Channel
d_n (ft)	5.5	6	6.5
d_c (ft)	5.2	5.7	6.2
V_n (fps)	7.6	13	6.6
F_n	0.82	0.94	0.80
τV (lb/ft.sec)	11.60	16.5	8.17

where $F_n = \dfrac{V_n}{\sqrt{gd_h}}$

$d_h = A_h/T_h$

- Stream Power at Normal Depth (as a Check on Manning's n), Upper Regime Flow

° HEC-2 Analyses for Existing Conditions (see Figure)

- Supercritical Flow Predominates

- Constriction of Flow Can Lead to "Chokes"

C. Erosion/Sedimentation (Existing Conditions)

° Development of Sediment-Transport Equation From Hydraulic Parameters and Sediment Data

$(Q_s = 1.21 \times 10^{-5}\ Y_h^{0.376}\ V^{3.70}\ W)$

° Estimation of Upstream Sediment Supply for Various Discharges, and Computation of Existing Q_s and Equilibrium Q_s Within the Study Reach to Determine Equilibrium Slope (see Figure)

° Application of Equilibrium Slope Concept

- Smaller Seq. than Sex. (Implies Erosion/Degradation Potential)

- Fairly Stable Bed Slope Since 1960 (Implies Erosion in Form of Bank Erosion)

- Lateral Migration Will Continue if Banks Remain Unstabilized

- If Banks are Stabilized, Degradation of the Bed Will Occur

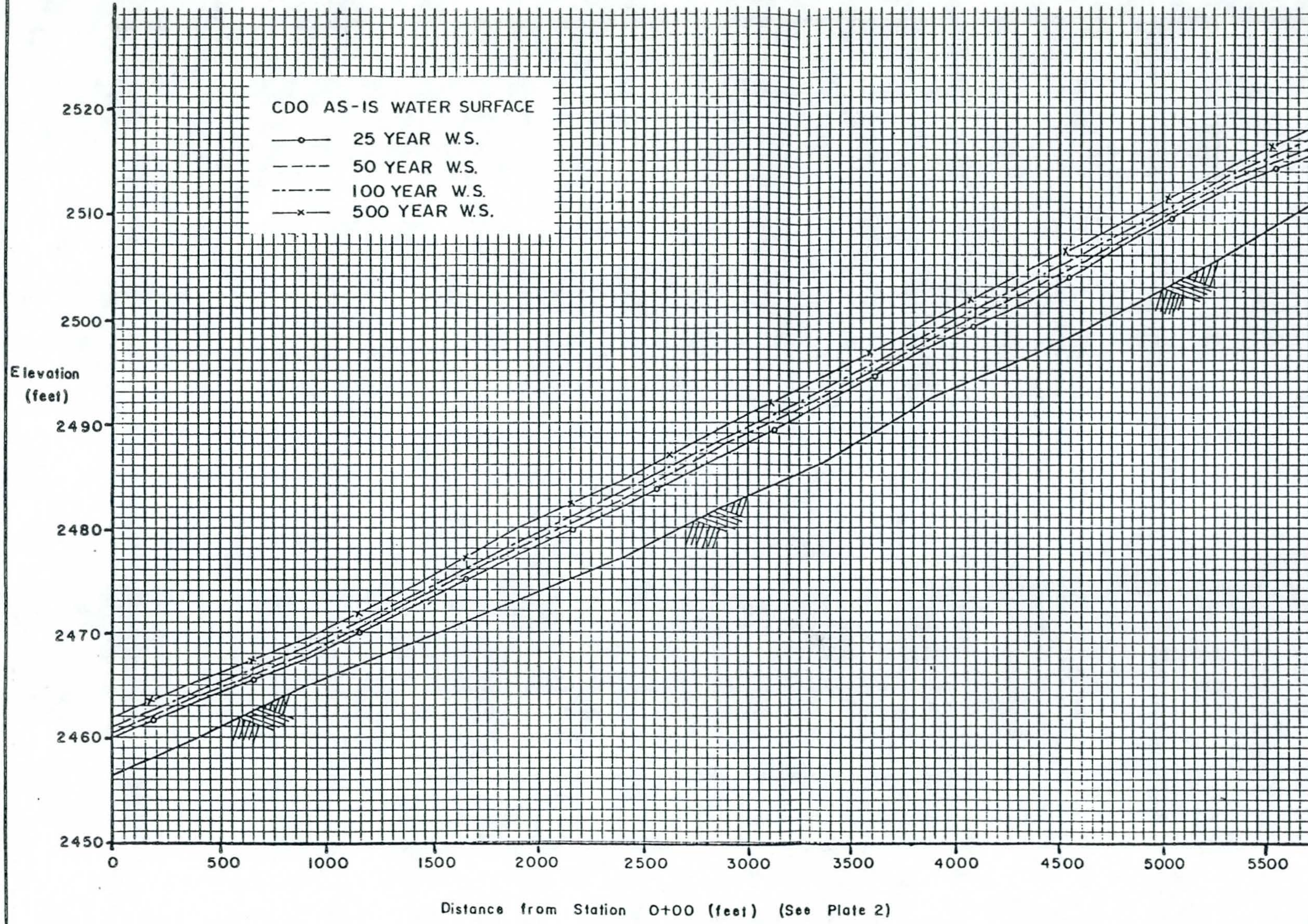
- Seq. for Stable Banks is 0.0064, Based Upon Q_2 as Dominant Discharge (No Armoring)

- Seq. Would Develop by the Process of Headcutting in an Upstream Direction

V. LEVEL III - MATHEMATICAL MODELING

A. Development of Model

° Sediment-Routing Model (HEC2-SR) Developed by SLA Used for Analysis



CANADA DEL ORO WASH
FLOOD CONTROL PROJECT
ORO VALLEY

WATER SURFACE PROFILES
FOR
CANADA DEL ORO FLOODS
AS-IS CONDITION

Overbank Flow Immediately Upstream
of Oro Valley Development

	Water Discharge (cfs)	Sediment Discharge (cfs)
25-year	7,200	2.0
50-year	11,000	3.7
100-year	17,000	8.2
500-year	31,000	20.7

Equilibrium Slopes, Oro Valley Reach

Discharge (cfs)	Upstream Sediment Supply (cfs)	Existing Sediment Discharge (cfs)	Equilibrium Slope
3,300	3.5	5.1	0.0064
10,000	18.8	24.3	0.0072
20,000	54.0	64.3	0.0078
33,000	120.0	134.0	0.0083

- Model Was Especially Calibrated to Deal With a Mix of Subcritical and Supercritical Flow Conditions Along the Study Reach

B. Model Application (Existing Conditions)

- Both Subcritical and Supercritical HEC-2 Runs Were Made, and the Most Appropriate Water-Surface Profile Selected for Use in the Model

- Used to Simulate Bed-Profile Response to a 100-Year Flood (see Figure)

C. Model Results (Existing Conditions)

- Aggradation/Degradation for the Study Reach Relatively Minor During a 100-Year Flood

- Aggradation at Downstream End of Oro Valley Town Limits Due to Flow Spread onto North Overbank of Flood Plain (Reduced Velocity = Reduced Sediment Transport)

VI. DEVELOPMENT OF ALTERNATIVE PLANS

A. Problem Statement (Canada del Oro Wash)

- Main Channel Narrowed From 2,500 Feet in Width to 400 Feet in Width Adjacent to the Town of Oro Valley

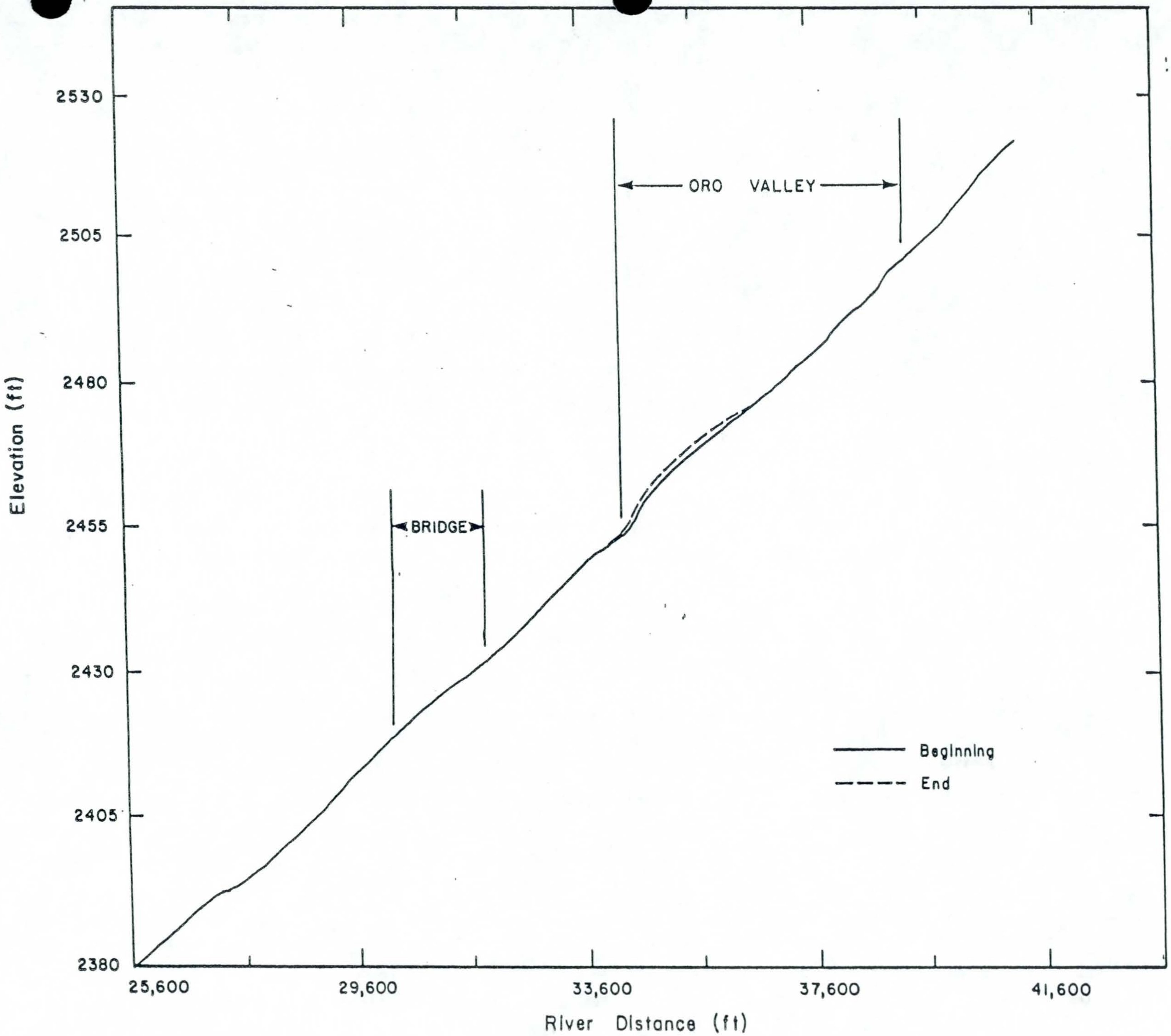
- Upstream Channelization (Circa 1960) Had Returned to Natural, Braided Pattern by 1971

- Being Developed Within the Original Braids of the Wash Portions of Oro Valley are Actually Below Present Thalweg Elevations in the Streambed (see Figure)

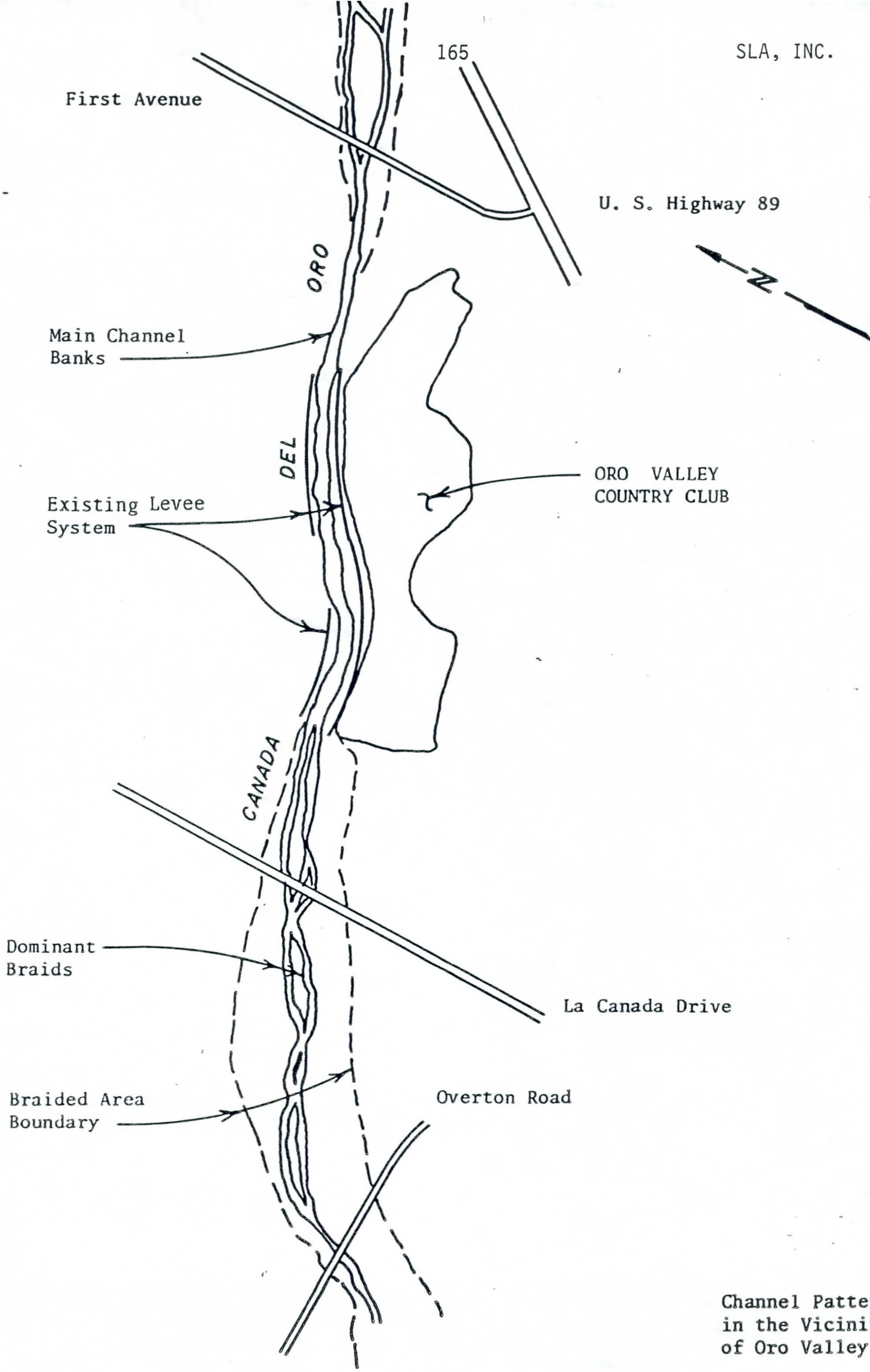
- Areas Adjacent to Wash Only Two to Four Feet Above Thalweg in Many Locations (A Levee is Therefore Required to Prevent Overbank Flooding)

- Due to Highly Unstable Conditions, Some Form of Permanent Stabilization Measures are Required

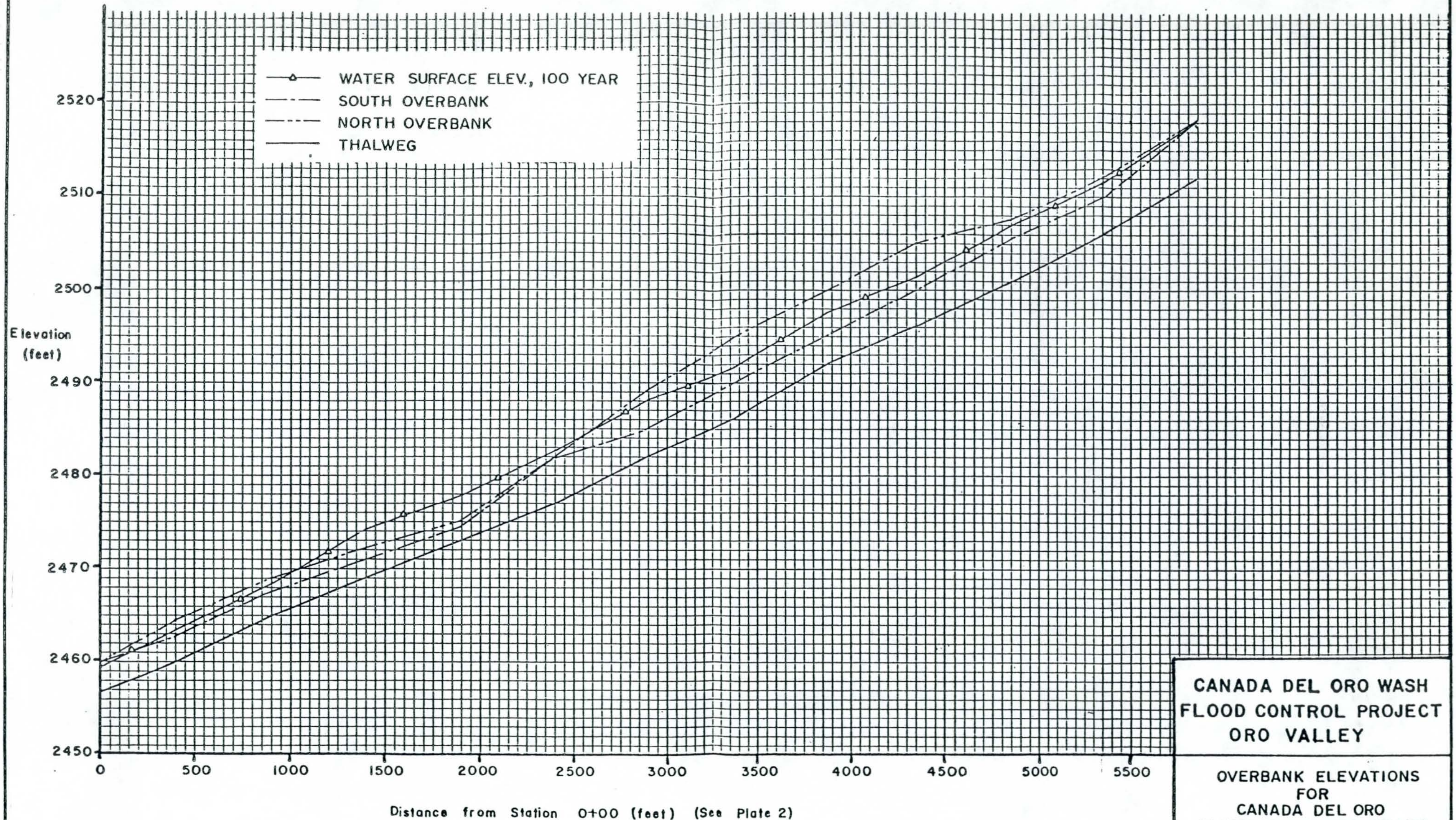
- Main Channel of Wash Conveys Approximately a 10-Year Flood, Well Below Q_{100}



Bed elevation change 100-year flood, existing conditions.



Channel Pattern
in the Vicinity
of Oro Valley.



CANADA DEL ORO WASH
FLOOD CONTROL PROJECT
ORO VALLEY

OVERBANK ELEVATIONS
FOR
CANADA DEL ORO
IN VICINITY OF ORO VALLEY

B. Problem Statement (Tributaries)

- Drainage System Required to Capture Upsteam Flows That Combine With Main Channel Breakout Flows in order to Transport Same Back Into Main Channel of the Canada del Oro Wash

- Side Drainage System Required to Transport Flows Through Oro Valley (SE to NW) and Into the Canada del Oro Wash

C. Alternatives Considered (see Figure)

- #1 - Bank Protection, With a Levee Along Only the South Bank of Canada del Oro Wash, Plus Construction of a Side-Drainage System to Accommodate Tributary Flows

- #2 - Bank Protection, With a Levee, Along Both the North and South Banks of the Canada del Oro Wash, Plus Construction of a Side-Drainage System to Accommodate Tributary Flows

- #3 - 400-Foot Wide Channelization of the Canada del Oro Wash Along Oro Valley; Plus Bank Protection, With a Levee Along Only the South Bank; Plus Construction of a Side-Drainage System to Accommodate Tributary Flows

- #4 - 400-Foot Wide Channelization of the Canada del Oro Wash Along Oro Valley; Plus Bank Protection, With a Levee, Along Both the North and South Banks; Plus Construction of a Side-Drainage System to Accommodate Tributary Flows

- #5 - 800-Foot Wide Total Channelization of the Canada del Oro Wash Along Oro; Plus Bank Protection Along Both the North and South Banks; Plus the Construction of a Side-Drainage System to Accommodate Tributary Flows

D. Comparison of Hydraulic Parameters at Peak of 100-Year Flood for "As-Is" Versus Design Alternatives (see Figures)

E. Comparison of Hydraulic Parameters at Peak of 2-Year Flood for "As-Is" Versus Design Alternatives (see Figure)

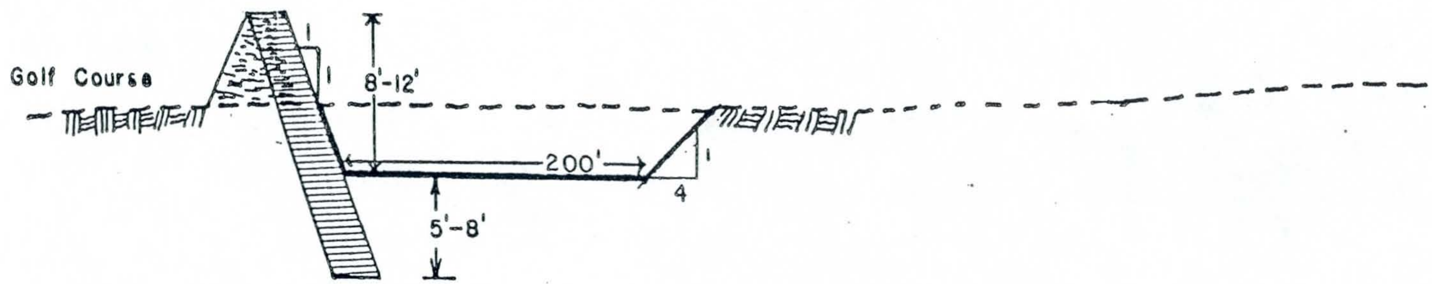
F. Sediment-Transport Analyses for Alternatives Considered Along the Canada del Oro Wash

- Equilibrium-Slope Analysis, Based Upon Previously - Developed Sediment - Transport Equation (see Figure)

- Assumption is That Banks are Rigid When Computing Equilibrium Slopes

Elements of Improvement for Each Alternative Considered

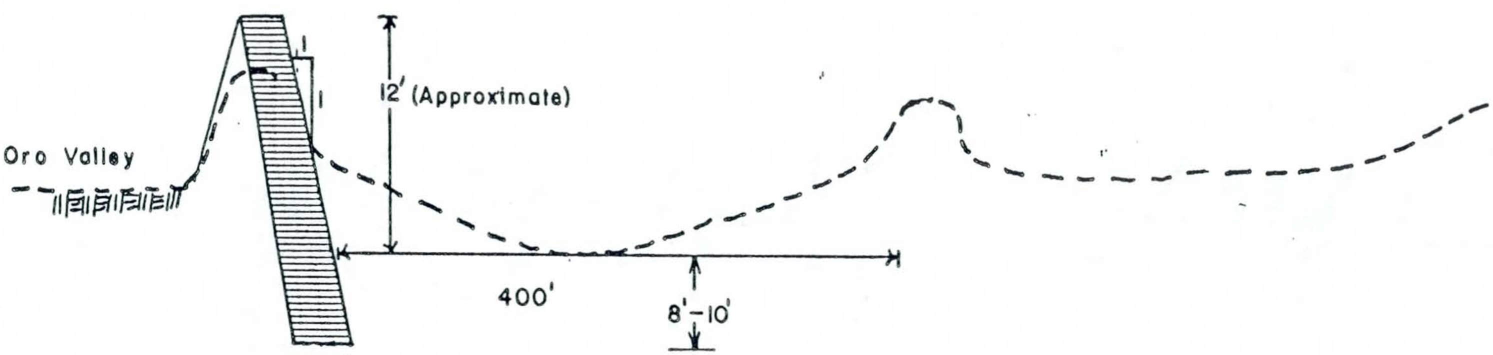
Element	#1	#2	#3	#4	#5
1. Repair of Existing South Bank Levee	5200 ft.	5200 ft.	5200 ft.	5200 ft.	———
2. New Levee on South Bank	1000 ft.	1000 ft.	1000 ft.	1000 ft.	———
3. Repair of Existing North Bank Levee	———	2760 ft.	———	2760 ft.	———
4. New Levee on North Bank	———	1700 ft.	———	1700 ft.	———
5. Bank Protection on South Bank	6200 ft.	6200 ft.	6200 ft.	6200 ft.	6200 ft.
6. Bank Protection on North Bank	———	4460 ft.	———	4460 ft.	4460 ft.
7. Channelization width	———	400 ft.	400 ft.	400 ft.	800 ft.
8. Grade Control Structures	———	5 ea	———	5 ea.	———



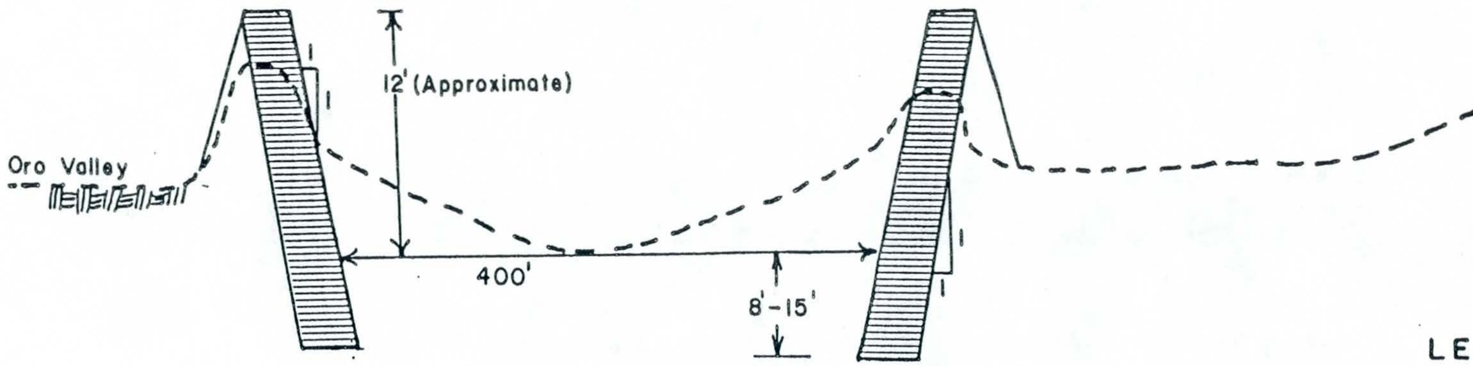
Typical Cross Section for Side Drain
Not to Scale

LEGEND

- - - Existing Cross section
▬ Channelization
▤ Soil Cement



Typical Cross Section in CDO for Alternative I
Not to Scale



Typical Cross Section in CDO for Alternative 2

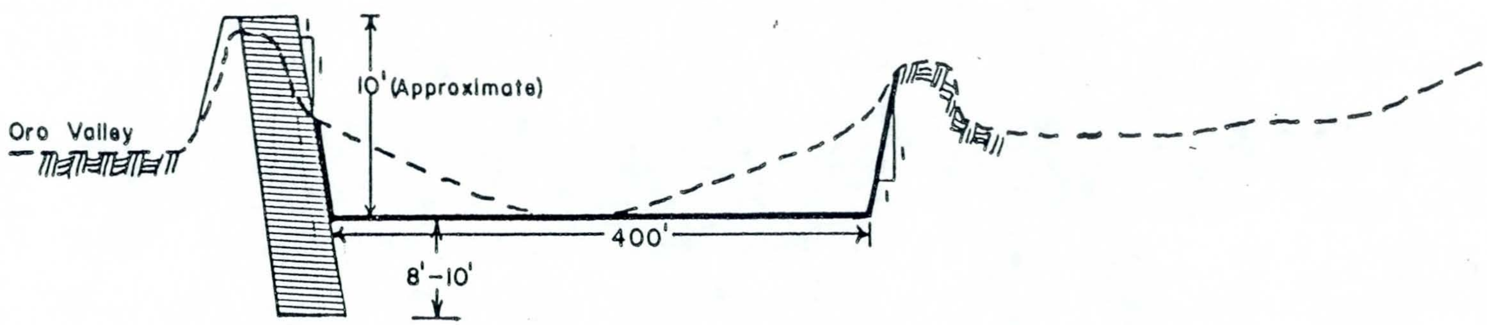
Not to Scale

LEGEND

- - - Existing Cross section

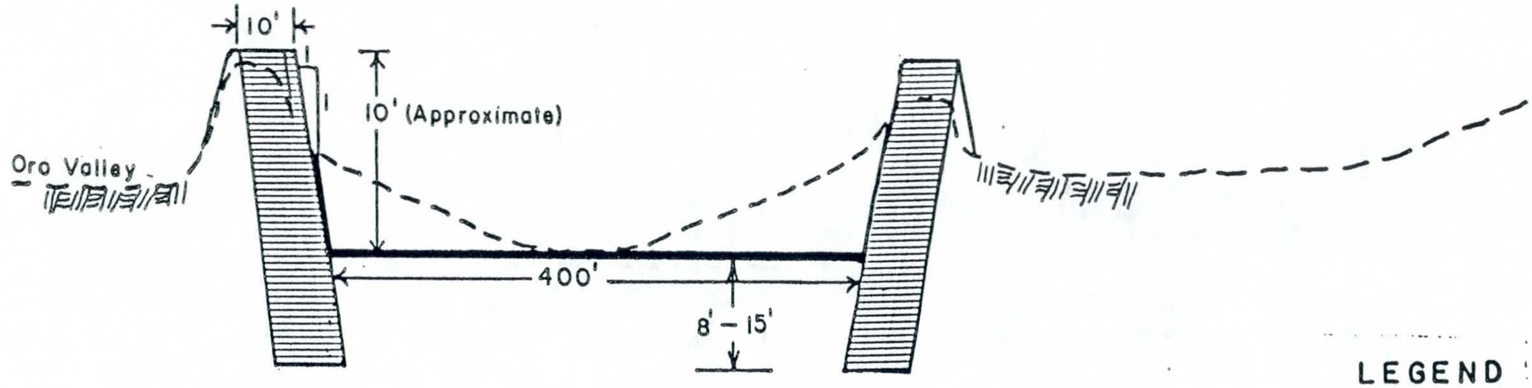
Channelization

Soil Cement



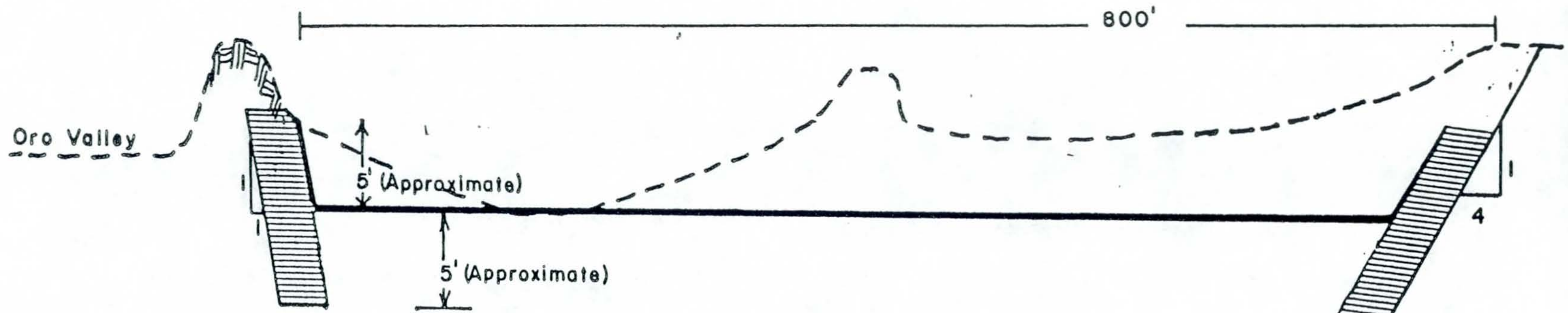
Typical Cross Section in CDO for Alternative 3

Not to Scale



Typical Cross - Section in CDO for Alternative 4

Not to Scale



Typical Cross Section in CDO for Alternative 5

Not to Scale

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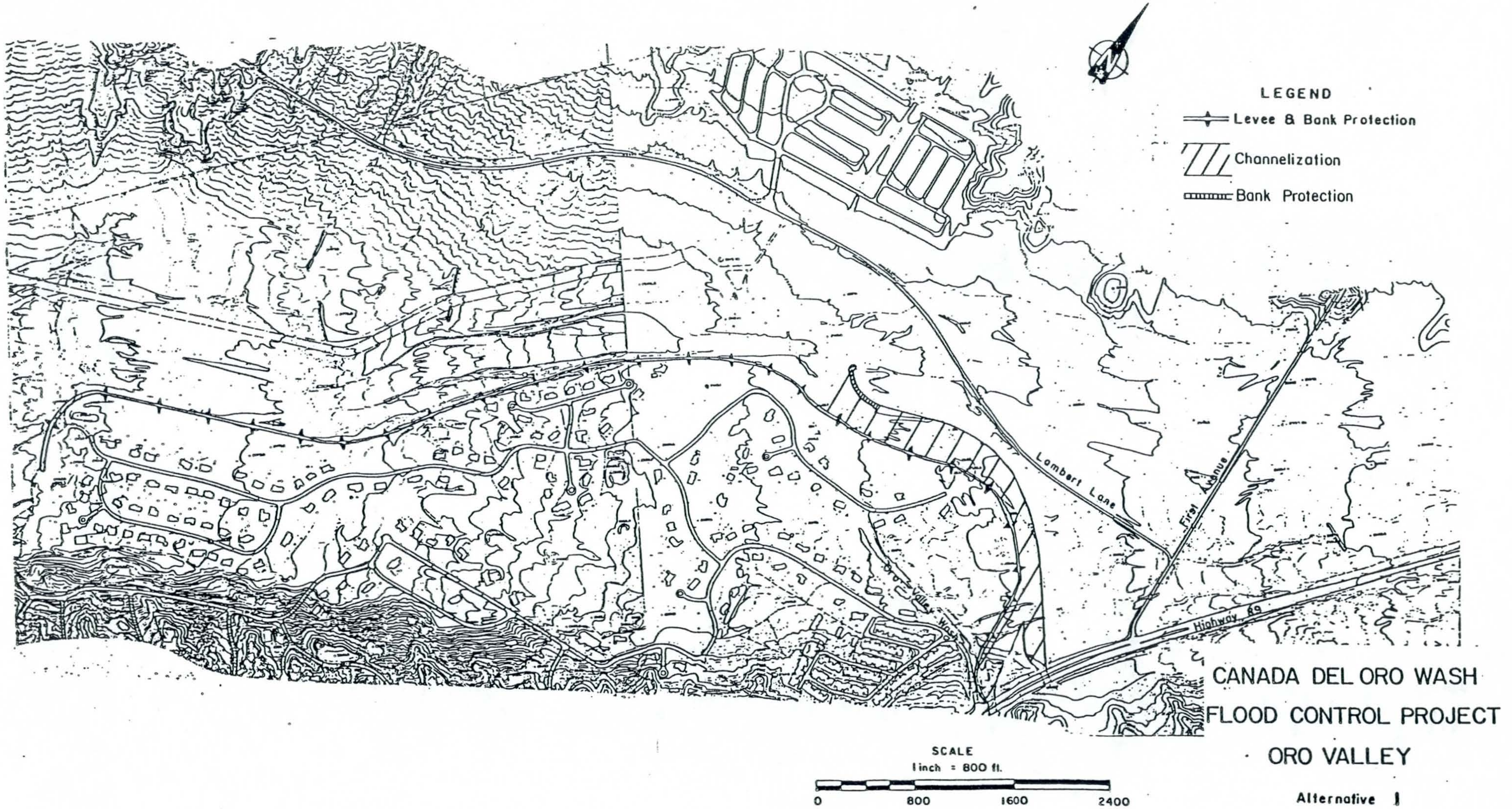
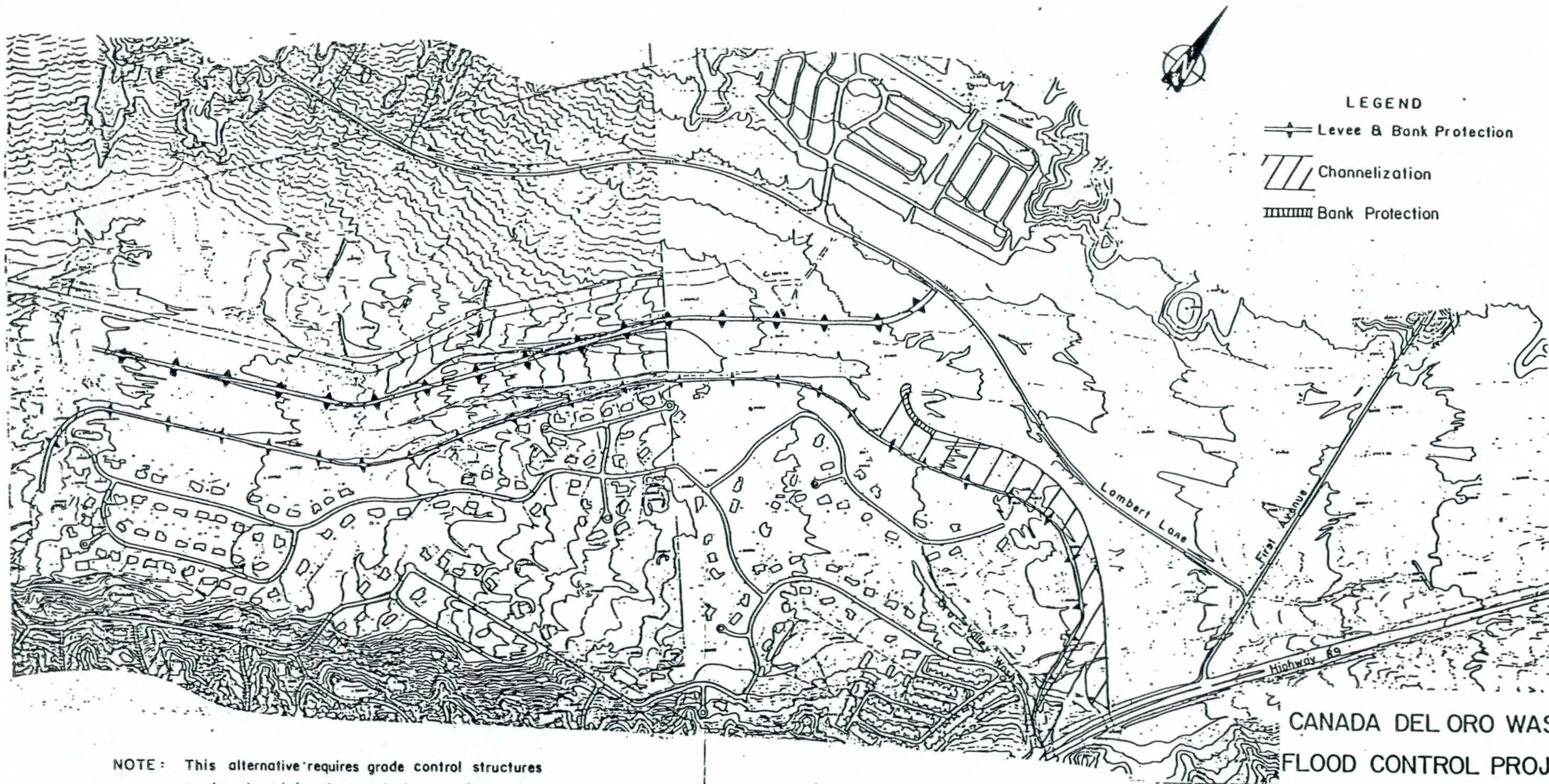

LEGEND
Levee & Bank Protection
Channelization
Bank Protection
Lambert Lane
First Avenue
Highway 89
SCALE
1 inch = 800 ft.
0
800
1600
2400
CANADA DEL ORO WASH
FLOOD CONTROL PROJECT
ORO VALLEY
Alternative 1

172

SLA, INC.



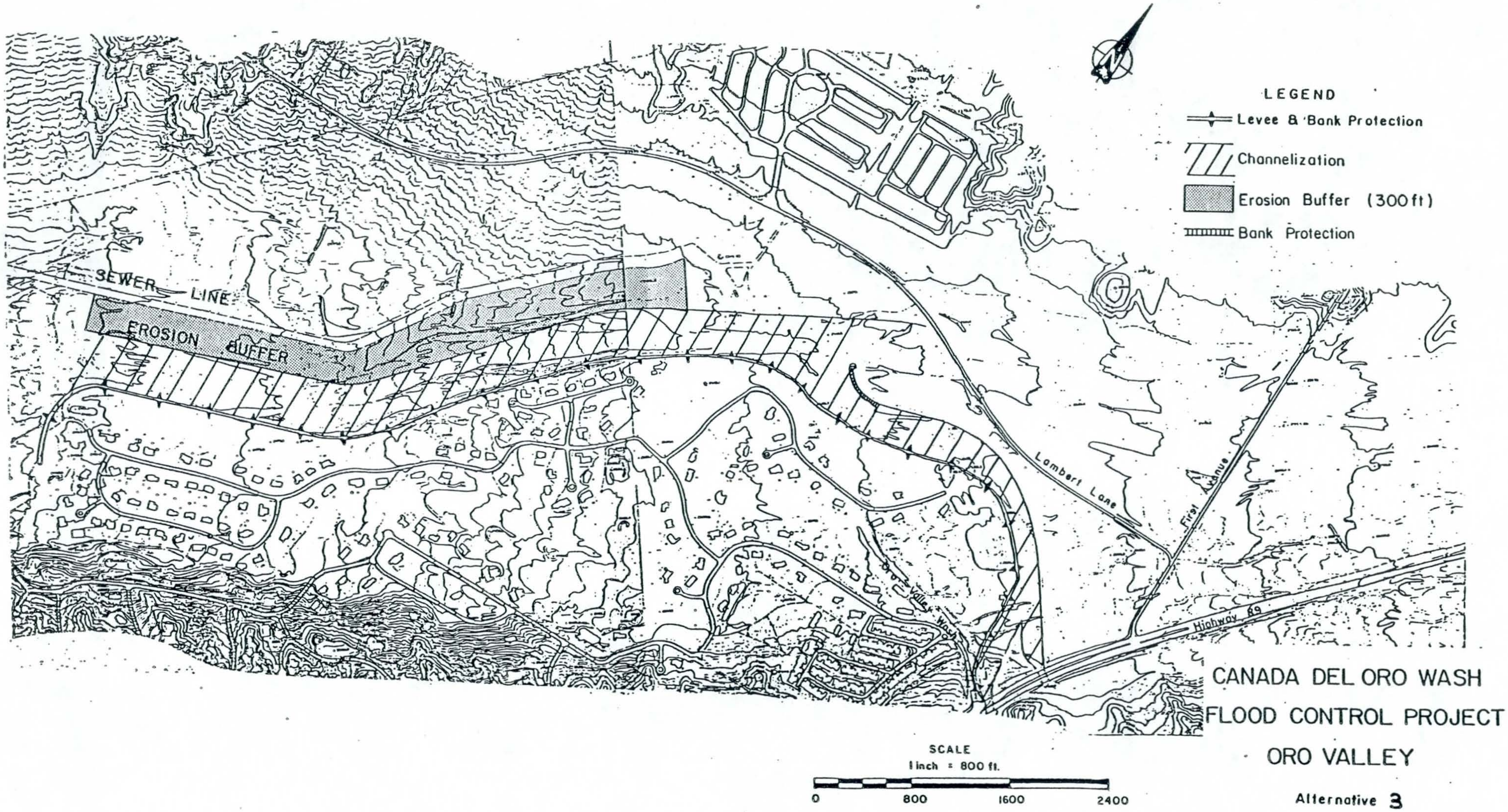
CANADA DEL ORO WASH FLOOD CONTROL PROJECT

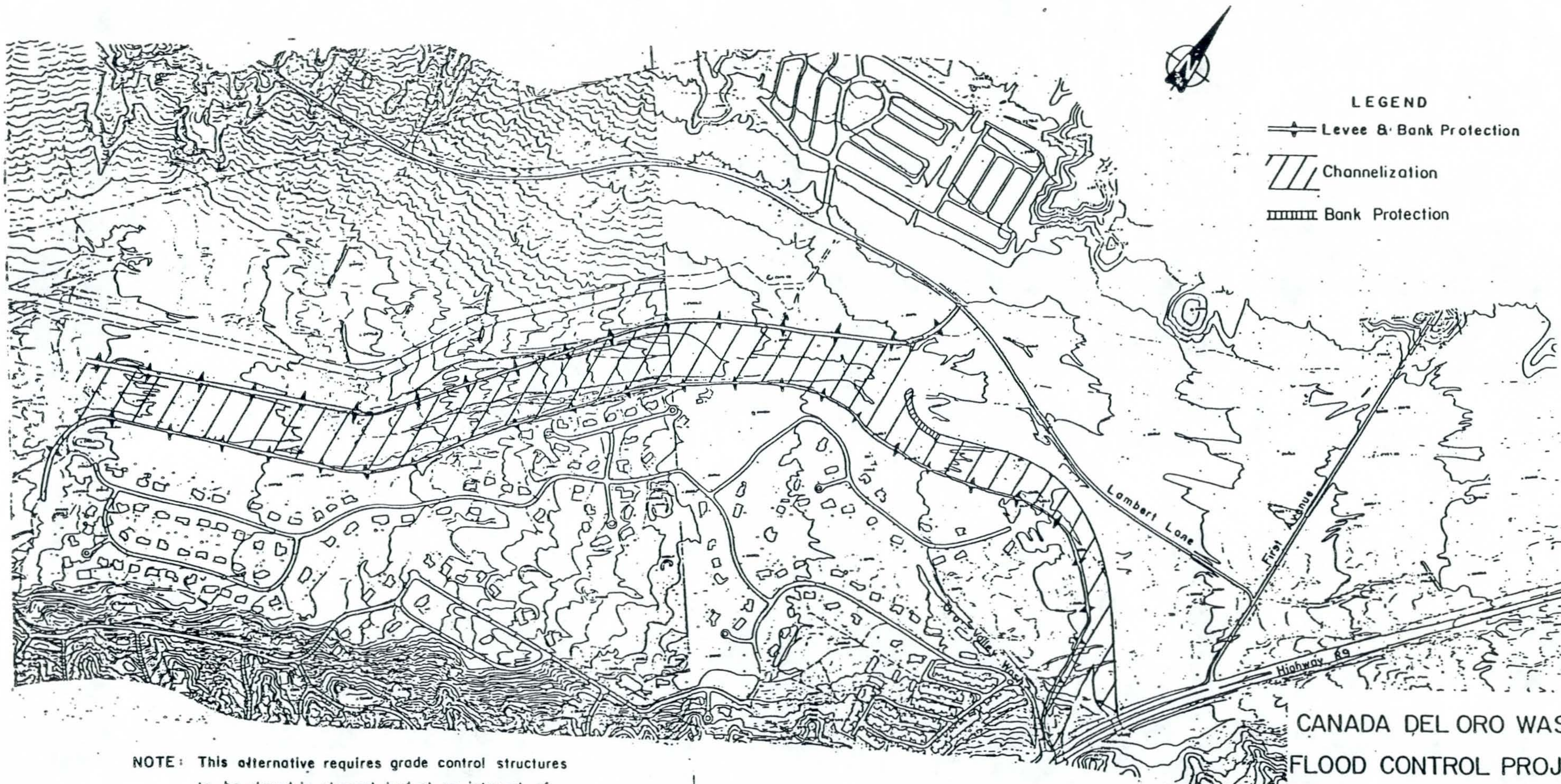
ORO VALLEY

Alternative 2

NOTE: This alternative requires grade control structures to be placed in channel bed at an interval of about 2000 feet.

SCALE
1 inch = 800 ft.
0 800 1600 2400


LEGEND
Levee & Bank Protection
Channelization
Erosion Buffer (300 ft)
Bank Protection
SEWER LINE
EROSION BUFFER
Lambert Lane
First Avenue
Highway 89
SCALE
1 inch = 800 ft.
0
800
1600
2400
CANADA DEL ORO WASH
FLOOD CONTROL PROJECT
ORO VALLEY
Alternative 3



LEGEND

Levee & Bank Protection

Channelization

Bank Protection

NOTE: This alternative requires grade control structures to be placed in channel bed at an interval of about 2000 feet.

SCALE
1 inch = 800 ft.
0 800 1600 2400

CANADA DEL ORO WASH FLOOD CONTROL PROJECT ORO VALLEY

Alternative 4

1/5

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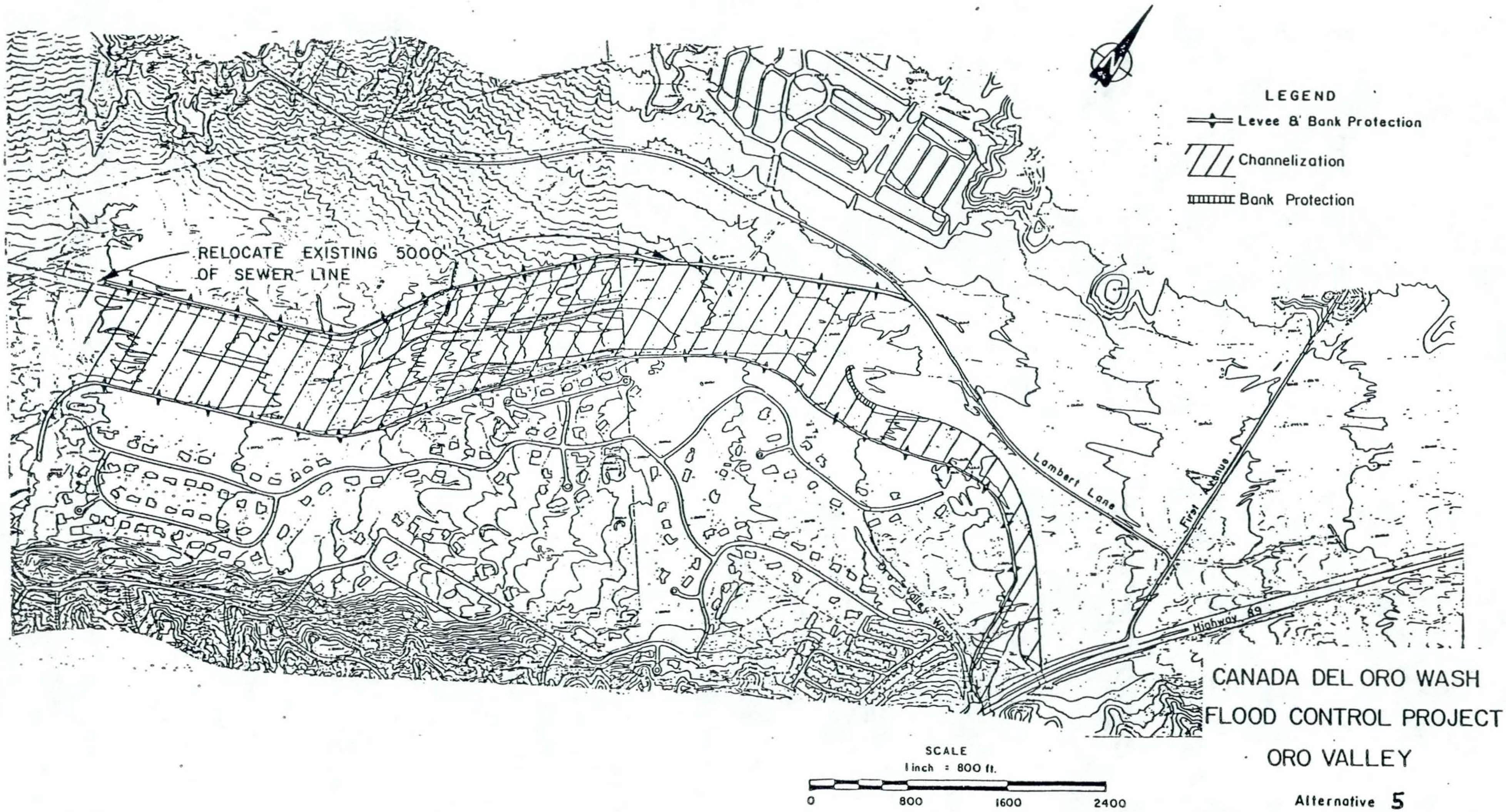


CANADA DEL ORO WASH
FLOOD CONTROL PROJECT
ORO VALLEY

Alternative 5

400 cross sectional Area
400' WIDE

A 150,000

Hydraulic Parameters Low-flow (3,300 cfs), 2-Year Peak Discharge

	Velocity (ft.)	Depth (ft.)	Effective Width (ft.)
As-is	6.8	1.7	285
Alternative 1	6.8	1.7	285
Alternative 2	6.8	1.7	285
Alternative 3	6.6	1.24	401
Alternative 4	6.6	1.24	401
Alternative 5	5.1	0.84	773

Hydraulic Parameters, 100-Year Peak Discharge (33,000 cfs)

	Velocity (fps)	Depth (ft.)	Effective Width (ft.)	Increase Velocity (fps)	Depth Increase (ft.)
As-is	13.0	6.0	423		
Alternative 1	14.1	7.3	321	1.1	1.30
Alternative 2	16.0	8.0	258	3.0	2.00
alternative 3	13.2	5.5	455	0.2	-0.50
Alternative 4	14.0	5.9	400	1.0	-0.10
Alternative 5	12.7	3.3	785	-0.3	-0.70

Equilibrium Slopes: Dominant Flow (3,300 cfs)

	Upstream Supply (cfs)	Sediment Transport Capacity (cfs)	Existing Slope	Equilibrium Slope
As-is Condition	3.5	5.1	0.0093	0.0064
Alternative 1		5.1		0.0064
Alternative 2		5.1		0.0057
Alternative 3		5.7		0.0064
Alternative 4		5.7		0.0057
Alternative 5		3.6		0.0090

Equilibrium Slopes: 100-Year Flood (33,000 cfs)

	Upstream Supply (cfs)	Sediment Transport Capacity (cfs)	Existing Slope	Equilibrium Slope
As-is Condition	120	234	0.0093	0.0083
Alternative 1		147		0.0076
Alternative 2		195		0.0057
Alternative 3		145		0.0077
Alternative 4		164		0.0068
Alternative 5		181		0.0062

- Maintenance of Bed Profile Due to Bank Erosion, and Subsequent Sediment Supply, Must Also be Considered

- Alternatives Which Include Bank Protection on Both Banks Must Incorporate Grade-Control Structures (see Figure)

- Potential Lateral-Migration is Estimated by Assuming all Sediments Come From North Bank, for Alternatives That Include Protection of Only the South Bank (Equation is Vol. = $Q_{s(out)} - Q_{s(in)}$ Times ΔT)

G. Selection of Recommended Plan (Plan #3)

- 400-Foot Wide Channelization of the Canada del Oro Wash Along Oro Valley; Plus Bank Protection, With a Levee, Along Only the South Bank; Plus Construction of a Side-Drainage System to Accommodate Tributary Flows

- Best Benefit/Cost Ratio (1.02)

- Erosion Buffer Required Along North Bank for Maintenance of Streambed Profile

H. HEC-2 Analysis of 100-Year Flood for Recommended Plan (see Figure)

I. Sediment-Routing Analysis (Bed-Profile Change) of 100-Year Flood for Recommended Plan (see Figure)

- Bank Erosion Not Considered

- Slight Degradation in Upstream Reach

- Slight Aggradation in Downstream Reach

VII. CONCLUSIONS AND RESULTS

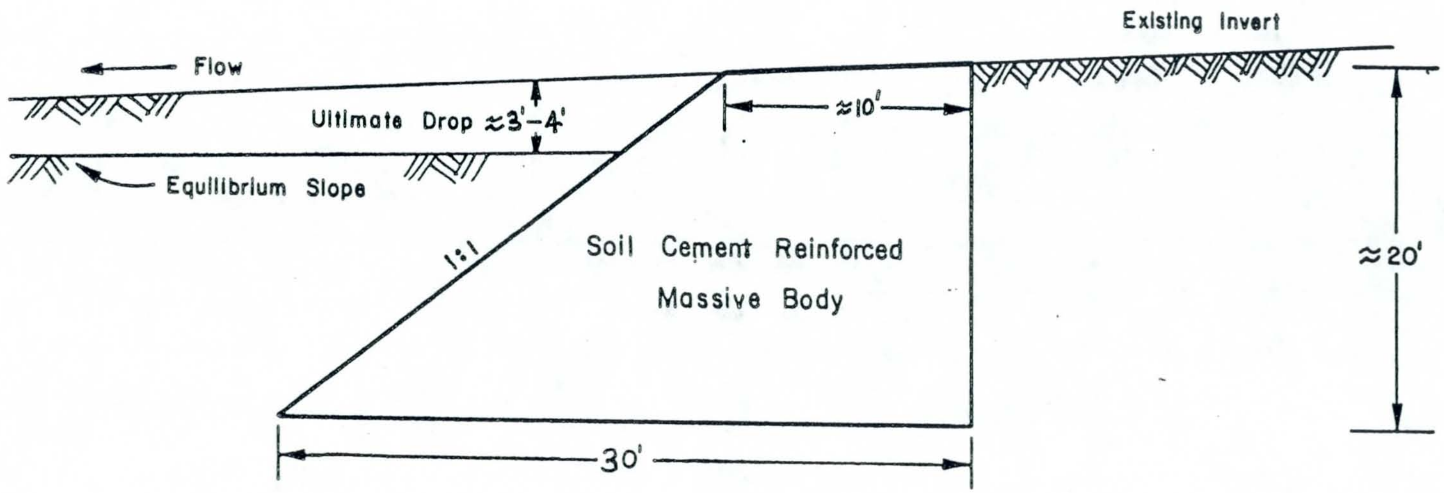
A. The Canada del Oro Wash is a Highly Dynamic, Braided River System Which Conveys Large Quantities of Sediment and is Highly Susceptible to Rapid Change in Cross-Sections Geometry

B. Flood Damages to Oro Valley Result From Flood Waters From the Canada del Oro Wash and its Tributary Channels (Stream Pump Rooney, and North Pusch Washes)

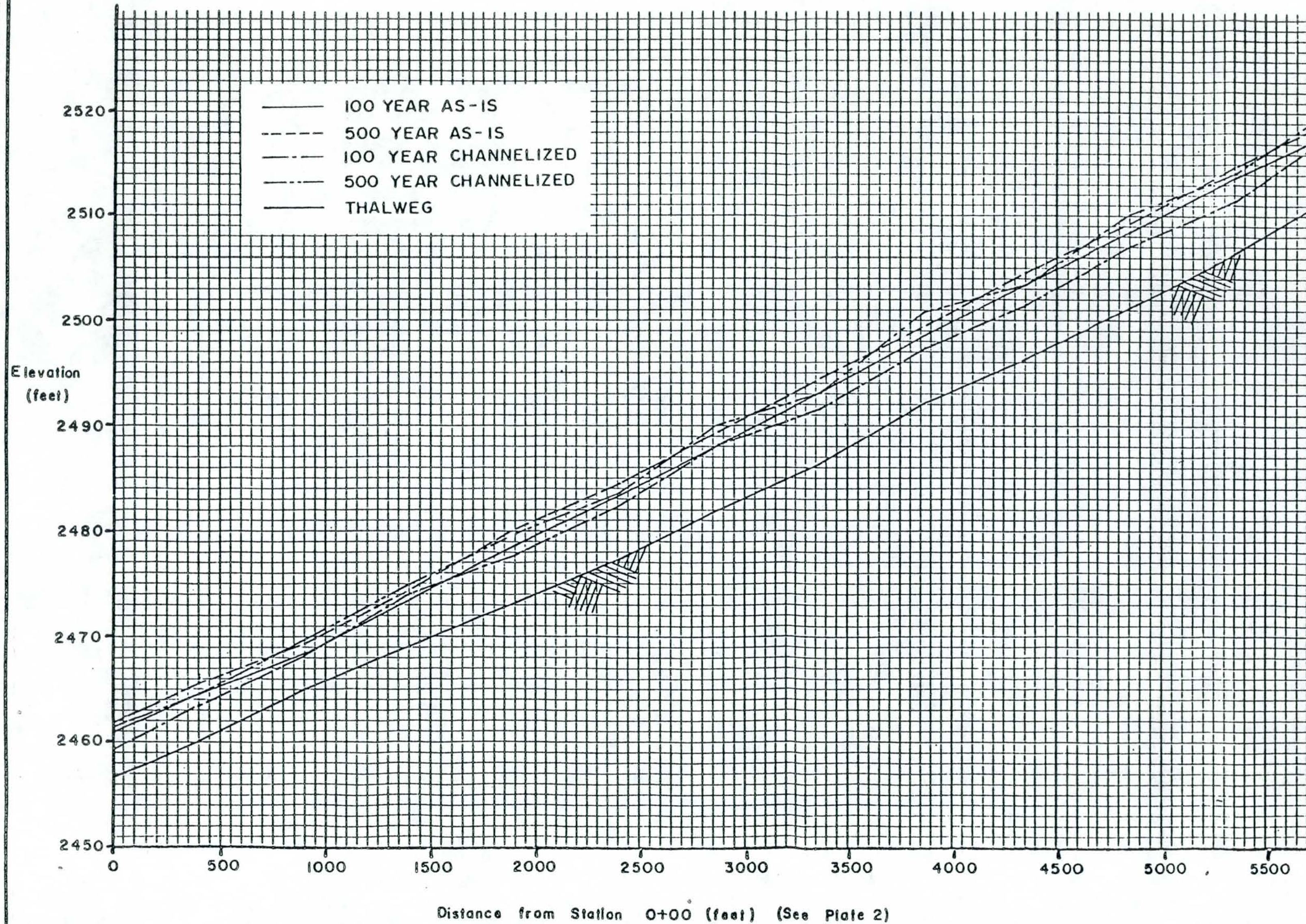
C. Channelization of the Canada del Oro Wash, Circa 1960, Narrowed its Width From 2,500 Feet to 400 Feet

TOTAL CROSS-SECTIONAL AREA	=	400 ft^2
TOTAL WIDTH	=	400 ft
TOTAL VOLUME	=	160,000 ft^3 = 5926 yd^3

NOT TO SCALE

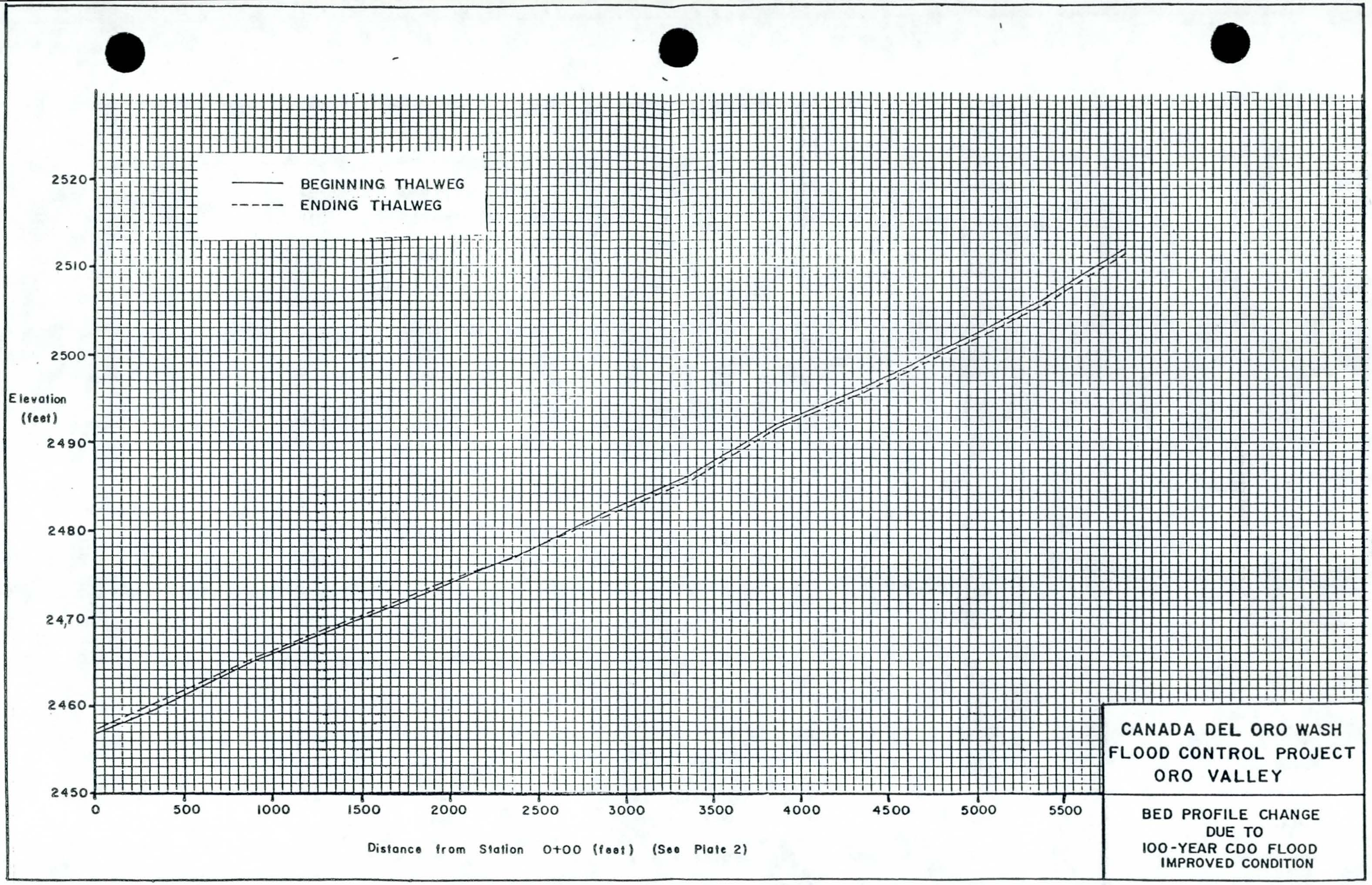


Typical cross section of grade-control structure.



CANADA DEL ORO WASH
FLOOD CONTROL PROJECT
ORO VALLEY

WATER SURFACE PROFILES
FOR
CANADA DEL ORO FLOODS
IMPROVED CONDITION


BEGINNING THALWEG
ENDING THALWEG
Elevation (feet)
2450
2460
2470
2480
2490
2500
2510
2520
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
5500
Distance from Station 0+00 (feet) (See Plate 2)
CANADA DEL ORO WASH
FLOOD CONTROL PROJECT
ORO VALLEY
BED PROFILE CHANGE
DUE TO
100-YEAR CDO FLOOD
IMPROVED CONDITION

D. Levees Were Constructed of Native Materials Insufficient to Resist Erosive Forces of Flood Waters

E. Sediment-Transport Capacity was Enhanced, Exacerbating Channel Instability

F. Flows Greater Than a 10-Year Flood Would Enter into the Town of Oro Valley

G. The Canada del Oro Wash Along Oro Valley Would be a Degrading Channel, Save for Added Sediment due to Bank Erosion/Lateral Migration

H. Five Alternatives Were Developed and Evaluated for Providing Flood and Erosion Control Along the Study Reach of the Canada del Oro Wash

I. The Recommended Plan (Plan #3) was a 400-Foot Wide Channelization Scheme Along the Canada del Oro Wash That Incorporated a Levee, With Bank Protection, Along only the South Bank; Plus the Construction of a Side-Drainage System to Accommodate Tributary Flows

J. The Engineer's Estimate for Cost to Construct the Recommended Plan was Approximately $4.3 Million

K. The Benefit/Cost Ratio for the Recommended Plan was 1.02

L. The Channel Depths and Velocities of Flow for the Canada del Oro Wash Under the Recommended Plan are Similar to Those for the "As-Is" Condition

M. The Construction of the Recommended Plan Will Have Negligible Impact Upon the Fluvial Environment Upstream of, an Downstream From, the Study Reach of the Canada del Oro Wash

N. The Construction of any Bank-Protection Measures Along the North Bank of the Canada del Oro Wash at Some Future Date Would Necessitate the Installation of Grade-Control Structure in Order to Maintain the Integrity of the Streambed Profile as it Presently Exists and Would Continue to Exist Under the Recommended Plan (Plan #3)

APPENDIX

Comparison of Prediction Equations for Bridge Pier and Abutment Scour

J. STERLING JONES

ABSTRACT

There are at least 10 prediction equations for bridge pier scour, and designers are often at a loss over which one to use. There are only three or four prediction equations for abutment scour, but these have not been highly publicized. The pier scour equations fall into three basic categories: those of the University of Iowa, those of the Colorado State University, and those based on foreign literature. The similarities among the pier scour equations and the range of data on which they are based are shown. FHWA sponsored several studies during the 1970s aimed primarily at comparing field data with the various equations to show which ones best predicted local scour for U.S. streams. These studies were somewhat inconclusive because of the many interrelated variables in the scour process, but they do show which equations are conservative and which are not.

Scour is the term used to describe erosion phenomena that involve unified flow patterns such as those at bridge piers, abutments, and outlet structures. Scour damage to highway structures has been estimated to be as high as $20 million per year (1). Years of research have been devoted to resolving the problem of scour at bridges; yet in spite of the magnitude of the damages attributed to scour, FHWA has never published an engineering circular bringing together the literature and giving guidance on how to account for scour at bridges. Lacking an engineering circular on the subject of bridge scour, there appears to be no better forum than a national conference among leading bridge engineers to reflect the knowledge that has been gained on this pertinent topic.

In a general sense, scour as defined by ASCE in Manuals 43 (2) and 54 (3) is the erosive action of running water in streams that excavates and carries away material from stream beds and banks. Many highway engineers relate to this definition and identify all stream erosion as scour. A preferred definition restricts the term to vertical stream erosion (4,5), thus distinguishing stream-bed erosion from lateral stream migration.

For the purposes of this paper it is useful to look at the components of scour and to focus on those components that are primarily related to bridges. The components are as follows:

1. Local pier scour,
2. Contraction scour,
3. Local abutment scour, and
4. General aggradation and degradation.

Although all these components may occur simultaneously and are probably interrelated in a field situation, they have been studied separately and need to be predicted independently in a design situation.

The two components of primary interest are pier and abutment scour because they are direct consequences of bridge obstructions to water flow and are therefore the primary responsibility of highway agencies. Aggradation and degradation are often the predominant components of scour but usually are site-specific phenomena associated with a stream's reaction to meander cutoffs, effective slope changes, downstream mining, reservoirs, and so on. Aggradation and degradation are probably best predicted by a sediment transport model such as the Corps of Engineers HEC-6 (6,7), the Chang model (7), or the Simons-Li model (7). Contraction scour may occur naturally because of narrowing of the floodplain or may be bridge related because of the encroachment on the floodplain by embankments. Abutment scour is a concentrated part of contraction scour that can be accounted for by empirically distributing the scour in a waterway opening.

NCHRP SYNTHESIS ON SCOUR AT BRIDGE WATERWAYS

In 1969 a synthesis of available literature and practices for dealing with scour at bridge waterways was made by the Highway Research Board. More than 100 organizations, including highway agencies, toll road agencies, consultants, railroad companies, and government agencies, were surveyed.

The synthesis report (5) cited 12 bridge pier scour prediction equations, but in the discussion of the prediction equations it was concluded that it was (5, p. 14) "quite impossible to build a feeling of confidence in any prediction method" because of a lack of field measurements with which to compare the predictions. The only guidance given for selecting the right prediction equation was to check the background of each equation and examine the variables included in each equation.

The synthesis report cited only one abutment scour equation. That was Laursen's equation in an appendix written by Laursen himself.

FHWA FOLLOW-UP STUDIES

FHWA sponsored several studies in the 1970s after the NCHRP synthesis aimed at improving confidence in some of the prediction methods. The West Virginia University study (8) had the objective of developing instrumentation and collecting field data for scour around bridge piers. The researchers found that it was less of a problem to develop instrumentation than to deploy it in a flooding environment so that it would be operational when needed.

Anderson (9) rearranged the equations to make them as similar in format as possible so as to facilitate an analytical comparison. He recommended that some large-scale laboratory studies be conducted to complement the field data, especially for extrapolating equations beyond the range of the original tests. He was able to rearrange the pier scour equations in terms of one or more of three dimensionless variables--flow depth and effective pier width, Froude number, and shear stress and critical shear stress. In other words, these three variables are the key factors that govern the pier scour process. Following Anderson's recommendation, FHWA sponsored a study at the University of Iowa (10) that had the objective of extending the range of the key factors, especially the Froude number, which includes flow velocity and depth.

PIER SCOUR EQUATIONS

The pier scour equations can be grouped into three basic categories for comparison. One category is the group based on foreign research, primarily in Pakistan and India, where fine bed materials are prevalent. A second category is patterned after the University of Iowa hypothesis that depth of flow is more important than velocity in sediment-transporting pier scour. The third category is patterned after the work at Colorado State University (CSU) and includes velocity (expressed in a Reynolds number or Froude number) as a predominant term.

The equations, identified by primary developer, are shown in the following paragraphs.

Pier Scour Formulas Based on Foreign Research

Ahmad [Pakistan 1962 (11)]

$$d_s/b = y_0/b\,(4.77F^{2/3} - 1) \tag{1}$$

Bruesers [Netherlands 1964 (12)]

$$d_s/b = 1.4 \tag{2}$$

Chitale [India 1962 (13)]

$$d_s/b = y_0/b\,(-5.49F^2 + 6.65F - 0.51) \tag{3}$$

Inglis [India 1949 (14)]

$$d_s/b = 4.05\,(y_0/b)^{3/4}\,[F^{1/2} - (y_0/b)] \tag{4}$$

where

d_s = depth of scour measured from the mean bed elevation,
y_0, y = approach flow depth,
b = projected pier width,
F = V/gy = Froude number, and
V = velocity of approach flow.

Pier Scour Formulas Patterned After University of Iowa Research

Laursen sediment continuity equation [1958 (15)]

$$d/b = 1/\left\{5.5[(1/11.5)(d_s/y_0) + 1]^{1.70} - 1\right\} \tag{5a}$$

Laursen [clear-water equation, not used in comparisons that follow (16; unpublished data, 1977)]

$$d/b = (\tau_0/\tau_c)^{1/2}/5.5\,[(1/11.5)(d_s/y_0) + 1]^{7/6}\,(\tau_0/\tau_c)^{1/2} \tag{5b}$$

Neil [from Laursen's 1956 design curve (9)]

$$d/b = 1.5\,(y/b)^{0.3} \tag{6}$$

Jain [1979 (10)]

$$d/b = 2.0\,(F - F_c)^{0.25}\,(y/b)^{0.5} \quad \text{for } (F - F_c) > 0.20 \tag{7a}$$

$$d/b = 1.84\,(y_0/b)^{0.3}\,F^{0.25} \quad \text{for } F < F_c \tag{7b}$$

where F_c is the critical Froude number when sediment transport is pending. [For $0 \le (F - F_c) \le 0.20$ use larger value from both equations.] The procedure for computing F_c is as follows:

1. Estimate the median diameter (d_{50}) for the bed material,
2. Determine τ_c from Figure 2.44 of ASCE Manual 54 (3),
3. Compute $U_{*c} = \tau_c/\rho$,
4. Compute $\delta = 11.6\upsilon/U_{*c}$ (assume $\upsilon = 1.08 \times 10^{-5}$ ft²/sec),
5. Compute d_{50}/δ,
6. Select x from Figure 2.97 of ASCE Manual 54 (3),
7. Compute $V = [2.5 \ln(11.02yx/d_{50})]\,U_{*c}$, and
8. Compute $F_c = V_c/(gy_c)^{1/2}$.

Pier Scour Formulas Patterned After CSU Research

Shen I [1969 (17)]

$$d_s = 0.00073\,Re^{0.619} \tag{8}$$

where Re is the pier Reynolds number, Vb/υ, and υ is the kinematic viscosity.

The Reynolds number is a viscosity parameter dependent on the water temperature, but few designers would be able to predict scour closely enough to account for water temperature, so Anderson reasoned that he could use $\upsilon = 1.2 \times 10^{-5}$ and approximate the exponent 0.619 by 0.66 to get

$$d_s/b = A F^{2/3}\,(y_0/b)^{1/3} \tag{9}$$

Depending on when the exponent is rounded, the coefficient A becomes 3.06 or 4.43. Anderson got 4.43 because he rounded the exponents before he multiplied through by the constants. It is reasonable to use an intermediate value of A = 3.4 and make the Shen I equation identical to the Shen II equation that follows, because both equations are based on the same data. Shen himself considered the equations to be equivalent.

Shen II [1969 (17)]

$$d_s/b = 3.4\,[V/(gb)^{1/2}]^{2/3} \tag{10}$$

which becomes directly

$$d_s/b = 3.4 F^{2/3}\,(y_0/b)^{1/3} \tag{11}$$

This will hereafter be referred to as the Shen equation.

CSU [1975 (18)]

$$d_s/b = 2.2\,(y_0/b)^{0.35} F^{0.43} \tag{12}$$

Even without considering the complexities of debris and cohesive materials, a designer is faced with nine equations to make a single computation. One might question why there are so many different equations for the same predictions if each researcher was accurate in his work. Furthermore, one might question whether the differences in the equations are as significant as other environmental factors that could not be included in the model studies used to develop the equations.

The equations are compared graphically in Figures 1 and 2. Because most of the equations are in terms of y/b as well as the Froude number, both figures

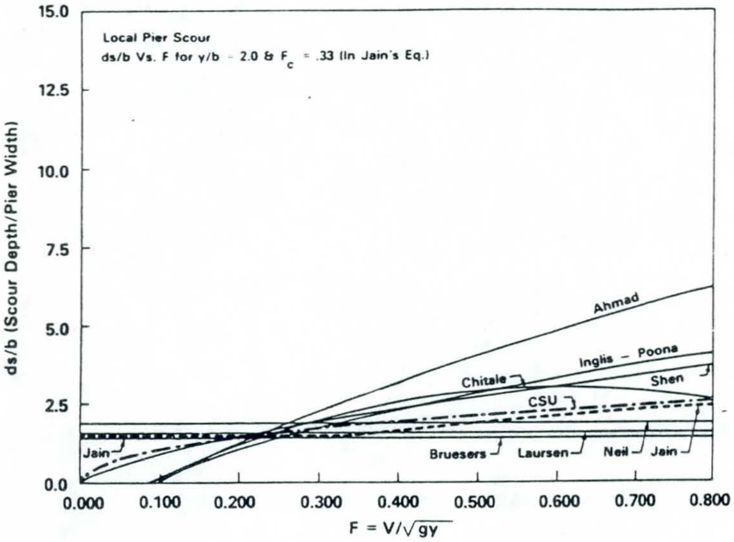


FIGURE 1 Graphical comparison of scour formulas (9) for variable Froude numbers.

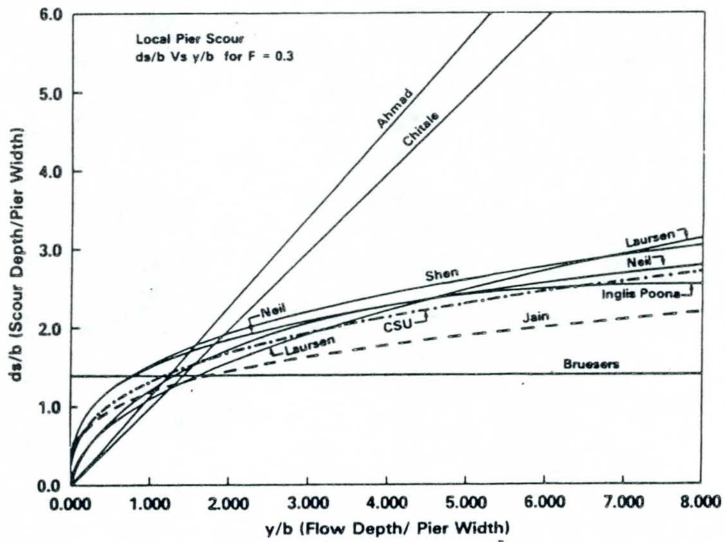


FIGURE 2 Graphical comparison of scour formulas (9) for variable depth ratios (y/b).

are needed to get a graphical comparison of the equations. Figure 1 is for an average depth ratio (y/b) of 2.0; Figure 2 is for an average Froude number of 0.3. The main difference in the equations is not so much in the data as in the way that the curves were fit to the data. All the equations are at least partly empirical and most are reasonably accurate if applied within the range of the empirical data.

SELECTING THE EQUATION FOR DESIGN

According to the NCHRP synthesis report (5), there are two approaches to selecting the most appropriate equation or equations for design. First, the equations should be compared with field data to deter- mine which ones best duplicate field measurements. Second, lacking these data, the conditions under which the equations were derived should be evaluated and the one that best matches the design conditions should be used. Most designers do not have time to review the literature to determine the derivation conditions for all of these equations, so a summary of data is shown in Figure 3 for equations from for- eign literature and in Figure 4 for equations from U.S. literature.

Looking back at Figure 2 where the equations are compared for variable values of y/b, the Ahmad and Chitale equations would not look so extreme if they were not extended beyond the range of experimental data (y/b approximately 3.5).

All of the pier scour equations derived in the literature have been for noncohesive materials with d_{50} ranging from 0.17 to 1.5 mm. The Bruesers equation is based on limited data, but because of its simplicity it serves as a good rule of thumb, which is to anticipate pier scour around 1.5 times the projected pier width. The Neil equation, which is based on the full range of Laursen's data, reduces to scour of 1.5 times the pier width when the flow depth is equal to the pier width. Neil's equa-

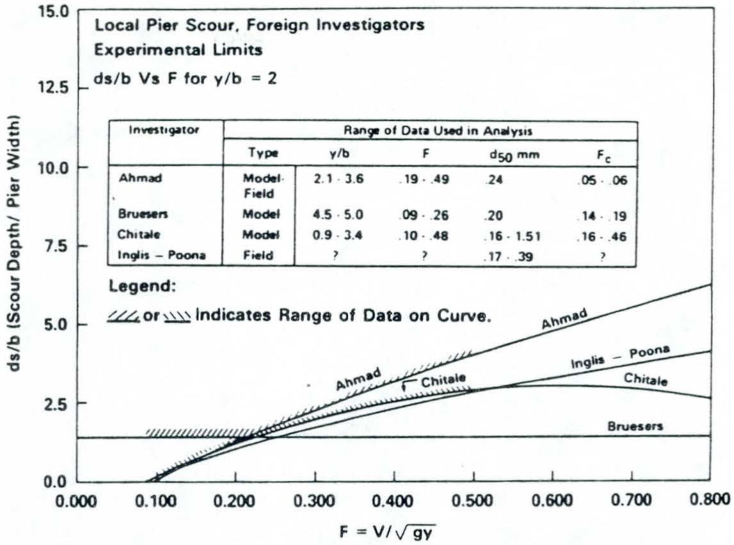


FIGURE 3 Summary of data used to derive pier scour equations: foreign literature.

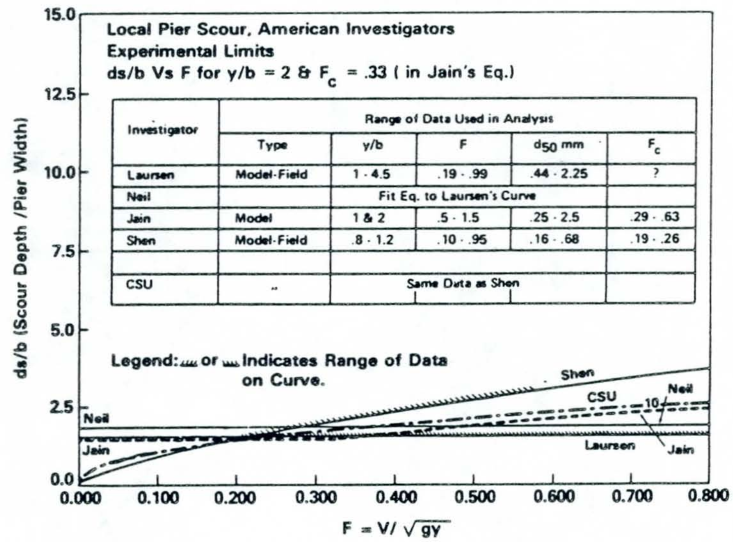


FIGURE 4 Summary of data used to derive pier scour equations: U.S. literature.

tion is just a regression fit to the design curve presented by Laursen in Iowa Bulletin 4 (19). Laursen later published his semitheoretical equations based on continuity of sediment transport, although they also have an empirical factor (Laursen's r-value) to make them fit experimental data. Nevertheless, Laursen's equation probably has the best basis for extrapolation beyond its experimental base. The main criticism of Laursen's equation is that it does not include a Froude number (or velocity term). Although continuity may be satisfied without a velocity term, it seems intuitively that velocity would affect the strength of secondary currents around a pier and therefore would be part of a prediction equation.

Shen's equation is an envelope curve that fits the uppermost scour points for all the available data [Figure 5 (17)]. This equation is appealing from a design standpoint because it is intentionally on the conservative side. The CSU equation is a best fit to much of the same data [Figure 6 (18)].

Jain's equation is somewhat of a compromise between those of Laursen and Shen. It has a Froude number, but the term has a relatively low exponent. Jain introduced the threshold Froude number (F_c) as a way of accounting for relative sediment size. One criticism of Jain's equation is that it is difficult to compute F_c.

FIELD DATA

The most convincing argument for selecting one scour equation rather than another is comparison of predictions with field measurements (8,20,21). Unfortunately, field measurements are scarce, especially under flood conditions, and those measurements that are available must be carefully scrutinized to isolate one component of scour from another.

Figure 7 (8) is a good example of some of the precautions that need to be observed with field measurements. Two floods at the same site are super-

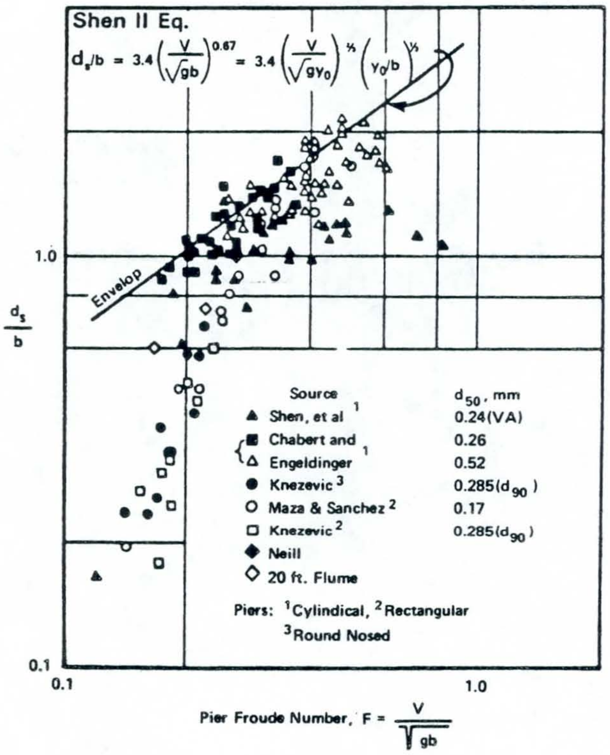


FIGURE 5 Shen's envelope fit to available data (17).

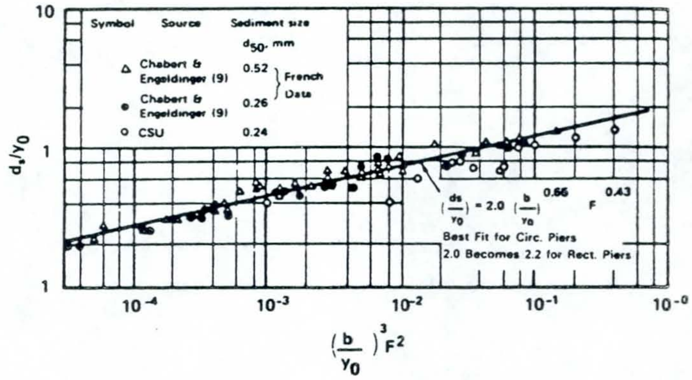


FIGURE 6 CSU fit to pier scour data (18).

imposed. The discharge of one was approximately three times the discharge of the other. The stage increased by 8 ft; the average velocity and Froude number at least doubled but there was apparently a reduction in local scour. How does one explain this discrepancy with the scour formulas?

First, this was not a good site to choose for such a comparison because the bed was already well below the footing (or pile cap); thus, the effective pier was the battered pile group. These conditions are not like any test conditions used to derive the scour formulas. Second, the entire cross section should be measured, not just one point, to even speculate how much local scour occurred. Third, the pier footing, which was approximately 20 ft square, probably served as a scour arrester for the diving currents that were generated by the actual pier at the higher stage, whereas the footing was just below the water surface at the lower stage and tended to generate rather than break up the diving currents at that stage. Data from a site like this give an intuitive feel for the magnitude of scour but they would be a weak argument for validating the scour prediction equations.

The field data gathered from Louisiana files by Chang (20) are summarized in Figure 8. Those sites had good pier configuration and well-defined scour holes, but for two reasons all the data were taken at low Froude numbers. First, the Louisiana streams are low-gradient streams and, second, the data were collected as part of a routine survey of approximately 90 sites by a hydrologic survey team. The surveys were made on a schedule and seldom coincided with floods, which may have generated some higher local velocities.

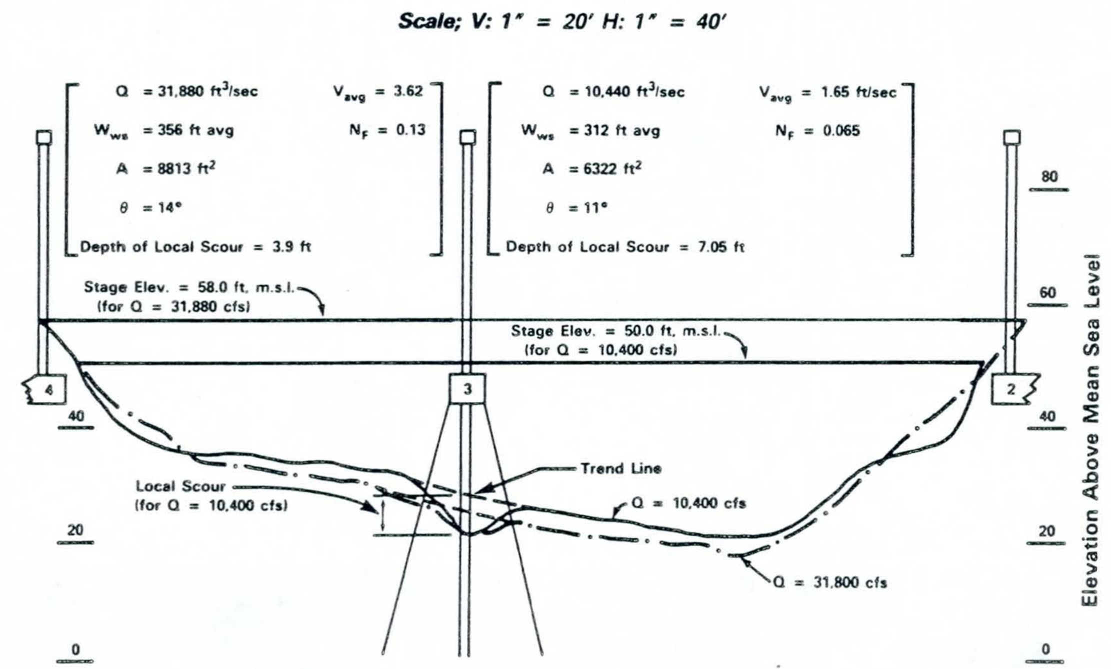


FIGURE 7 Cross section of Brazos River in Texas at bridge crossing (8).

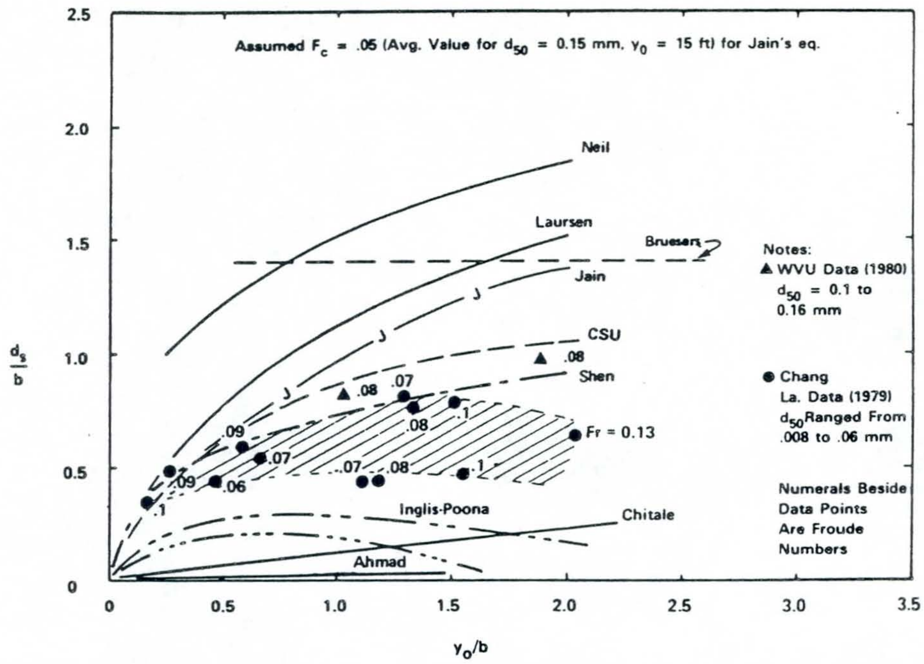


FIGURE 8 Louisiana field measurements compared with prediction equations (20).

EFFECTS OF PIER SHAPE

The pier scour equations presented in the preceding discussion are for rectangular piers. Although there have been investigations (22) of the effects of pier shape, a designer is not likely to be able to take advantage of most of the streamlined shapes because flow directions change as flood levels change. The conservative approach for a designer is to use the projected width of the pier in the direction of flow and to use the rectangular pier equations except for round piers, which scour about 90 percent as much as rectangular piers.

APPLICATION OF PREDICTION EQUATIONS

A consideration that is more important than factors like pier shape and even which equation to use is the manner in which a designer selects design parameters. None of the scour equations is based strictly on field measurements in which the cross section is irregular and flow conditions are nonuniform. Most scour equations are based on uniform, one-dimensional flow conditions. To use an equation effectively, a designer must somehow visualize the field conditions in a manner that resembles the test conditions. The tendency is to use average depths and velocities for an irregular cross section. A more reasonable approach may be to use the depth and velocity in a band of flow just upstream of the pier. Velocity is harder to predict this way, but it can be assumed to follow a logarithmic distribution.

The greatest discrepancy between laboratory conditions and field conditions is in the bed material. Most of the laboratory tests were run with uniform cohesionless soils. General practice in design is to use these equations as a conservative estimate if a soil is considered erodible. If a soil is considered nonerodible, scour is assumed to be zero. The problem is the lack of something in between, but that is a problem that must remain unsolved until someone devises a plan to deal with the effects of different soil properties.

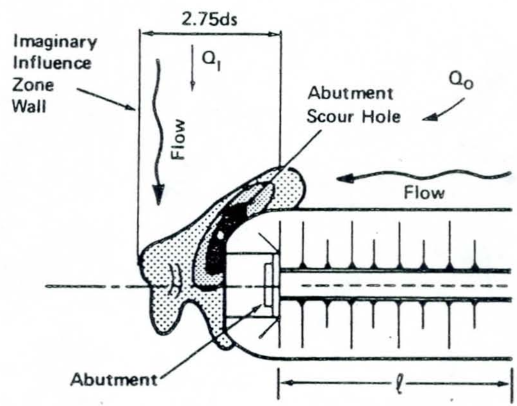


FIGURE 9 Typical scour at an abutment (18).

ABUTMENT SCOUR

Abutment scour occurs when overbank flow reenters the main channel and sets up large vortices in the bridge opening. Laursen reasoned that the continuity equation for sediment flow needed to be satisfied and conceptualized an approach abutment at the symmetrical half of a wide pier. Typically water in the main channel is transporting sediment at capacity, and water in the overbank area is relatively free of sediment (so-called clear water). When these flows mix at the abutment, there is a deficiency of sediment and this deficiency is satisfied with material from the abutment scour hole. Laursen realized that the mixing occurred primarily in the zone of flow near the bank and that it was not reasonable to dilute the overbank flow with the entire channel flow because the computed scour would decrease directly with the width of the bridge opening. He defined an influence zone as shown in Figure 9 and derived the abutment scour equations based on no mixing outside the influence zone. The width of the influence zone is 2.75 times the depth of the abutment scour hole, which means that Laursen's abutment scour equations must be implicit equations.

For the typical case where the flow in the influence zone (Q_I) transports sediment and flow from the overbank area (Q_0) is clear water, Laursen's sediment continuity equation yields the following:

$$(Q_0/Q_I)\,2.75(d_s/y) = 2.75(d_s/y)\left\{[(1/r)(d_s/y)+1]^{7/6} - 1\right\} \quad (13)$$

The recommended value for r in this case is 4.1.

For the special situation in which both the flow in the influence zone and the overbank flow are clear water, which could occur at relief bridges or where the abutments are set back far enough on the floodplain, Laursen's equation yields the following:

$$\ell/y = 2.75\,(d_s/y)\left(\left\{[(1/r)(d_s/y)+1]^{7/6}/(\tau_0/\tau_c)^{1/2}\right\} - 1\right) \quad (14)$$

where

ℓ = length of the approach embankment,
$\tau_0/\tau_c = V^2/120 d_{50}^{2/3} y^{1/3}$,
r = 4.1 for low velocities, and
r = 11.5 for high velocities.

d_s is the deepest part of the scour hole, which is assumed at the edge of the abutment.

Laursen's r-value essentially distributes the scour in a triangular hole. The larger the r-value, the larger the ratio of the deepest scour depth (d_s) to the average scour depth in the influence zone.

CSU relationships for abutment scour are as follows:

$$d_s/y = 1.1\,(\ell/y)^{0.40}\,F^{0.33} \qquad \text{if } \ell/y < 25 \quad (15a)$$

$$d_s/y = 4\,F^{0.33} \qquad \text{if } \ell/y > 25 \quad (15b)$$

There are no field data to compare with the abutment scour equations, but because there are only two equations to consider, it is reasonable to compute with both of them. A designer would have to use the equation that suited his tendency to be more or less conservative.

RECOMMENDATIONS

There is still a need to document field data for both pier scour and abutment scour. Field data should be collected during floods and should as a minimum include a full cross section at several flood stages. Data should be collected by individuals who are knowledgeable about how laboratory tests are conducted and who can document sufficient information to make valid comparisons with predictions.

Although there are a large number of pier scour equations, they can be narrowed down to three or four without much loss in data used in derivations. The recommended equations are those by Laursen, Jain, Shen, and CSU, Equations 5, 7, 11, and 12, respectively.

There are only two equations (13 and 15) to consider for abutment scour if the main channel flow in the zone next to the abutment (the influence zone) is transporting sediment. There is only one equation (14) for abutment scour for relief bridges and large abutment setback where flow in that influence zone is not transporting sediment.

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Homework Assignment
Sediment Transport

Problem #1

Calculate the smallest non-moving sediment size over the range of the flood event given. Assume that the water surface gradient and the channel gradient remain approximately equal throughout the flood at 0.005 ft/ft.

Given:

1) Flood hydrograph, figure 1

2) Stage - discharge, figure 2

Problem #2

Calculate the bed-load transport for a cobble bed channel using the Meyer-Peter Muller bed-load equation. Compare the difference in the estimated sediment transport as determined using multiple size fractions versus using a single characteristic size. If the difference is significant, explain the difference.

Given:

1) The gradation curve for a cobble bed channel was divided into five size fractions. The geometric mean and percentage of the distribution of the particle sizes in each fraction are listed in the table below.

Fraction	Percent of Sample Weight	Mean Particle Size, mm
1	10	80
2	25	30
3	30	16
4	25	8
5	10	3

2) The following hydraulic conditions exist in the channel.

Q = 2500 cfs	d = 4.7 ft
S = 0.0065 ft/ft	T = 55 ft
n = 0.035	A = 260 sq ft

(3)

$q_s = aV^{4.32} y_h^{-.30}$

$a = 6.4 \times 10^{-3} \dfrac{n^{1.17} G^{.45}}{D_{50}^{.61}}$

$q_s = 6.4 \times 10^{-3} \dfrac{(n)^{1.17} V^{4.32} G^{.45}}{y_h^{.3} D_{50}^{.61}}$

$a = 2.53 \times 10^{-5}$

$n =$

1 mm $\quad a = 1.61 \times 10^{-5}$

$G = 3 \quad b = 4.11$

$c = .112$

$q_s = 2.53 \times 10^{-5} (V)^{4.32} y_h^{-.30}$

T_s Reach Velocity Depth q_s $T\ Q_s$ ΔQ_s

$Q_s =$

Problem #3

Compare the sediment transport rate for a sand bed channel using both the Colby procedure and the empirical power relationships given in the Design Manual. For this problem, execute the Colby method by size fractions, then execute equations 5.8a and 5.8b from the Design Manual, and as a final check execute equation 5.8a by size fractions assuming G = 1 for each size fraction.

Given:

1) The following bed material gradation.

Fraction	Percent of Sample weight	Mean particle Size, mm
1	10	0.77
2	15	0.46
3	50	0.25
4	10	0.15
5	15	0.10

2) The following hydraulic conditions.

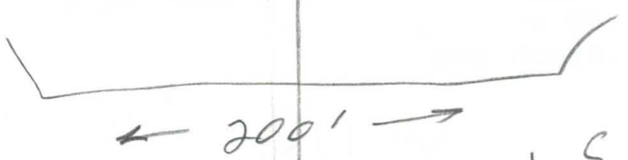
Q = 30,500 cfs	d = 10.0 ft
S = 0.0012 ft/ft	T = 320 ft
n = 0.025	A = 3190 sq ft
T = 50° F	
Cf= 10,000 ppm	

(2)

Time step	Q	Reach 1 V	Reach 1 d	Reach 1 T	Reach 2 V	Reach 2 d	Reach 2 T
1	1000	4.0	1.1	460	3.2	2.3	206
2	5000	4.7	2.8	560	5.6	4.7	253
3	11000	5.3	4.4	665	7.3	7.0	277
4	7500	4.8	3.5	609	6.2	5.8	268
5	3000	4.2	2.2	520	4.8	3.8	240
6	1000	4	1.1	460	3.2	2.3	206

upstream supply

$V = Q/yB$



bed S .003

$Q = \frac{1.49}{.03}(200)(y^{5/3})(.003)^{1/2}$

1 1000	3.5	1.5	200
2 5000	6.5	3.8	
3 11000	8.8	6.2	
4 7500	7.6	4.9	
5 3000	5.3	2.8	
6 1000	3.5	1.5	200

pg 554

$q_s = 6.44\,10^3 \frac{n^{1.77} V^{4.32} G^{.45}}{y_n^{.30} D_{50}^{.61}}$

$G = \frac{1}{2}\left(\frac{1.9}{0.8} + \frac{0.80}{.19}\right) = 3.3$

$n = .03$

$d_{50} = .80mm$

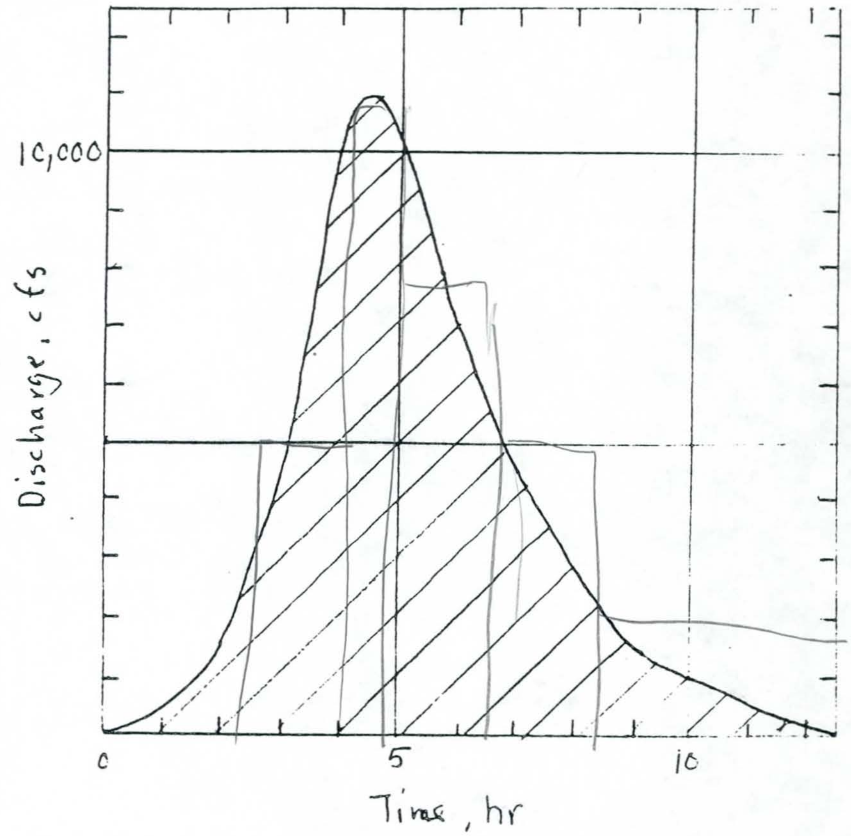


FIGURE 1

1

hr	Qm
2.5	1000
1.5	50.00
1.0	11000
2hr	7500
2hr	3000
3 hrs	1000

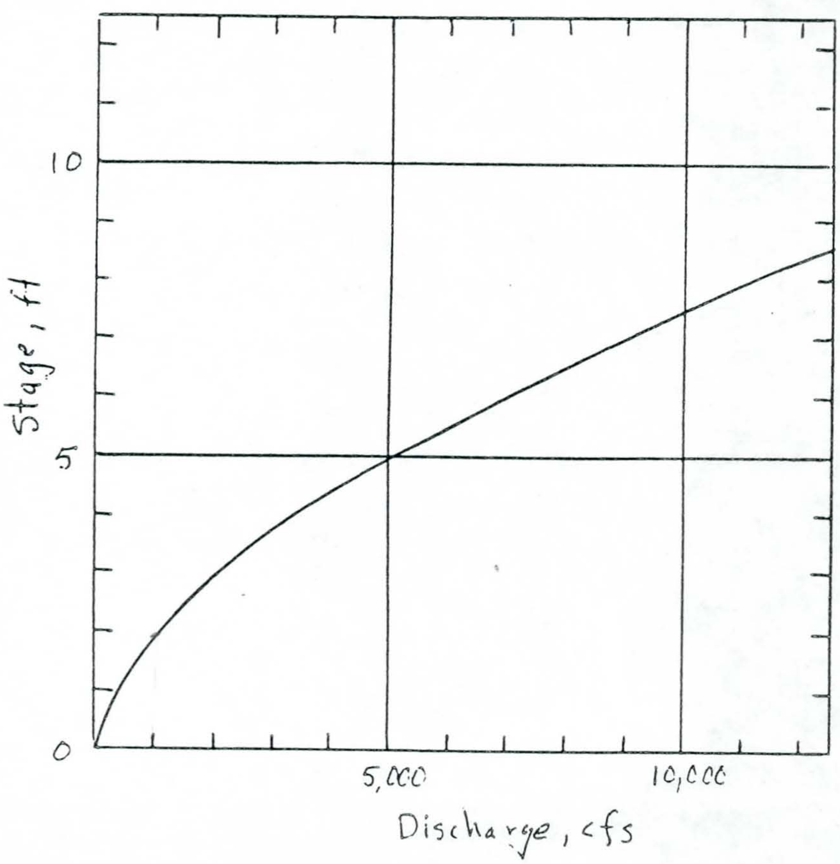


FIGURE 2

DESIGN MANUAL FOR ENGINEERING ANALYSIS OF FLUVIAL SYSTEMS

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TABLE OF CONTENTS

TABLE OF CONTENTS (continued)

TABLE OF CONTENTS (continued)

TABLE OF CONTENTS (continued)

LIST OF FIGURES

LIST OF FIGURES (continued)

LIST OF TABLES

LIST OF TABLES (continued)

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DEFINITION OF SYMBOLS

A	Area
A_w	Total wetted roughness cross-sectional area
b	Bottom width
b_p	Pier width
b_{rg}	Roughness geometry parameter in Bathurst's procedure (1978)
C	Chezy resistance factor
c	Sediment concentration (ppm by weight)
d	Depth of flow measured normal to direction of flow
D	Diameter of pipe
D_a	Sediment particle size of the armor layer
D_c	Sediment particle size at incipient motion
D_i	Percent finer particle size (i.e. D_{50}, D_{90}, etc.)
f	Darcy-Weisbach friction factor
F.B.	Freeboard distance
Fr	Froude number
Fr_p	Pier Froude number
G	Gradation coefficient
G_s	Specific gravity
g	Acceleration of gravity
h_a	Antidune height from crest to trough
h_f	Friction loss
L	Length
n	Manning's roughness coefficient
p	Probability
P	Wetted perimeter
P_c	Percent of material coarser than armor size
q	Water discharge per unit width
Q	Water discharge
q_b	Bed-load transport per unit width
q_p	Peak water discharge per unit width
q_s	Bed-material sediment discharge per unit width
R	Hydraulic radius
r_c	Radius of curvature
R_p	Pier Reynolds number

S	Slope
S_e	Slope of energy gradient
S_o	Slope of channel bed
S_{50}	Median size of short axis of particle in Bathurst's procedure (1978)
T	Top width
t_c	Time of concentration
V	Mean velocity
V_*	Shear velocity
VOL_{inc}	Water volume from probability weighting
VOL_{meas}	Water volume from gaging station data
VOL_s	Sediment volume
W	Width of flow
X	Distance downstream of a channel bend at which secondary flow becomes negligible
Y	Depth of flow measured vertically
y_c	Critical flow depth
Y_h	Hydraulic depth
y_o	Normal depth
Y_{50}	Median size cross-stream axis of particle in Bathurst's procedure (1978)
z	Side slope angle (horizontal:1)
Δy_{agg}	Additional flow depth due to long-term aggradation
Δy_d	Additional flow depth from debris accumulation
Δy_s	Additional flow depth from separation in short-radius bends
Δy_{se}	Superelevation
ΔZ_a	Depth to formation of armor layer
ΔZ_{bs}	Bend scour depth
ΔZ_{deg}	Change in bed elevation from long-term degradation
ΔZ_{gs}	General scour depth
ΔZ_i	Low-flow channel incisement depth
$\Delta Z_{\ell s}$	Local scour depth
ΔZ_{tot}	Total vertical adjustment in bed elevation
γ	Specific weight of water
γ_s	Specific weight of sediment
ρ	Density
τ_c	Critical tractive force

τ_0	Boundary shear stress (tractive force)
σ	Standard deviation
υ	Kinematic viscosity
π	Pi (3.14)

I. INTRODUCTION

The rigid boundary conditions upon which most flood control studies are currently based do not acknowledge the potential for river systems to move both laterally and vertically. Failure to address this problem in the design and construction of flood control projects, bridges or other structures located within a flood plain can lead to their premature destruction or obsolescence. Recognizing this deficiency in typical design procedures, the Arizona Department of Water Resources initiated development of this design manual.

The purpose of this design manual is to present techniques and procedures that may be used to make a thorough engineering analysis of major fluvial systems in order that the natural processes associated with such systems can be accounted for in the design of flood control projects. The importance of this is vividly illustrated by the photographs in Figures 1.1 to 1.4.

Figures 1.1 (Pantano Wash - Tucson, Arizona) and 1.2 (Rillito River - Tucson, Arizona) illustrate the lateral migration that can occur during a flood. In particular, the power line poles of Figure 1.1 illustrate the extent of lateral migration possible during a single flood. Figures 1.3 (Santa Cruz River - Tucson, Arizona) and 1.4 (Rillito River - Tucson, Arizona) illustrate the potential for both loss of life and property during a single event. In Figure 1.3, the Cortaro Farms Road bridge was completely destroyed, and in Figure 1.4 a townhome is on the verge of falling into the river. The situations illustrated all developed during the October 1983 flooding in southeastern Arizona. The need for an engineering analysis in order to predict fluvial system response, and to design adequate mitigating measures that will prevent or limit the dangers illustrated in Figures 1.1 to 1.4, is self-evident.

Information in this manual addresses the dynamics of watershed and channel systems considering hydrologic, hydraulic, geomorphic, erosion and sedimentation aspects. The emphasis is placed upon practical implementation of state-of-the-art technology in identifying, evaluating and designing for the natural processes associated with major fluvial systems. Depending upon engineering judgment and project economics, the principles discussed herein can also be applied to the design of small conveyance drainage systems. Only that information considered absolutely essential to understanding the basic theory of the application procedures has been presented, while other relevant, but

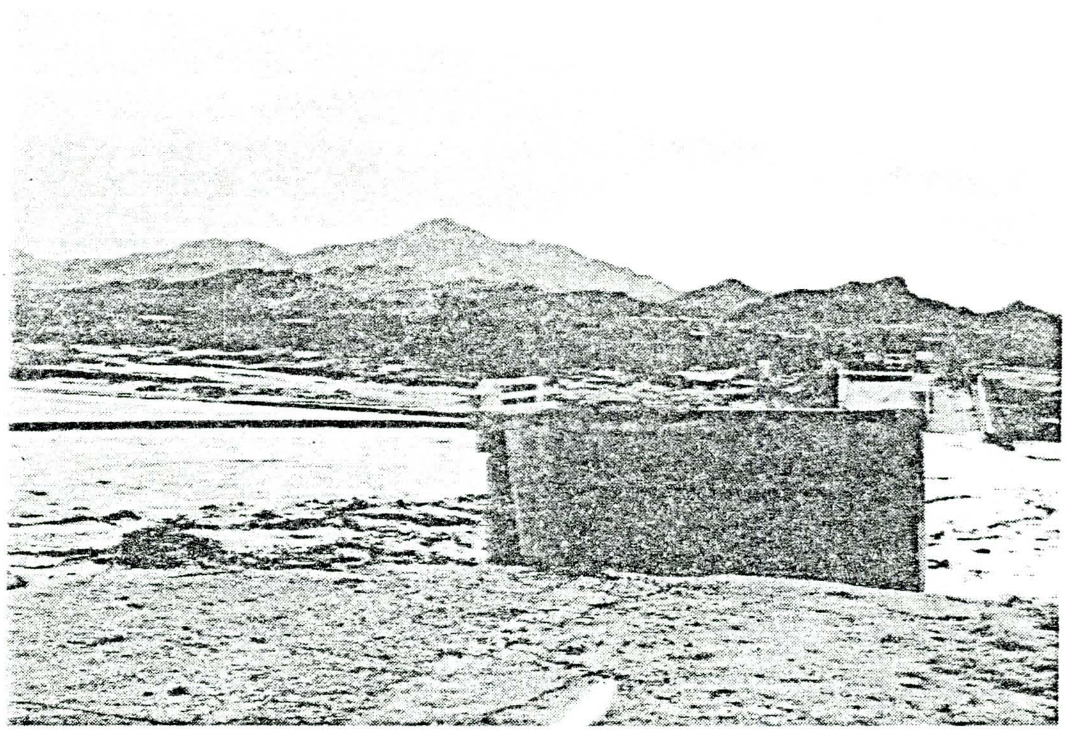


Figure 1.1. View from the Speedway Blvd. bridge looking upstream along the east bank of the Pantano Wash, Tucson, Arizona (Photo date: October,1983).

Figure 1.2. View from south bank looking northwest toward the First Avenue bridge over the Rillito River, Tucson, Arizona (Photo date: October, 1983).

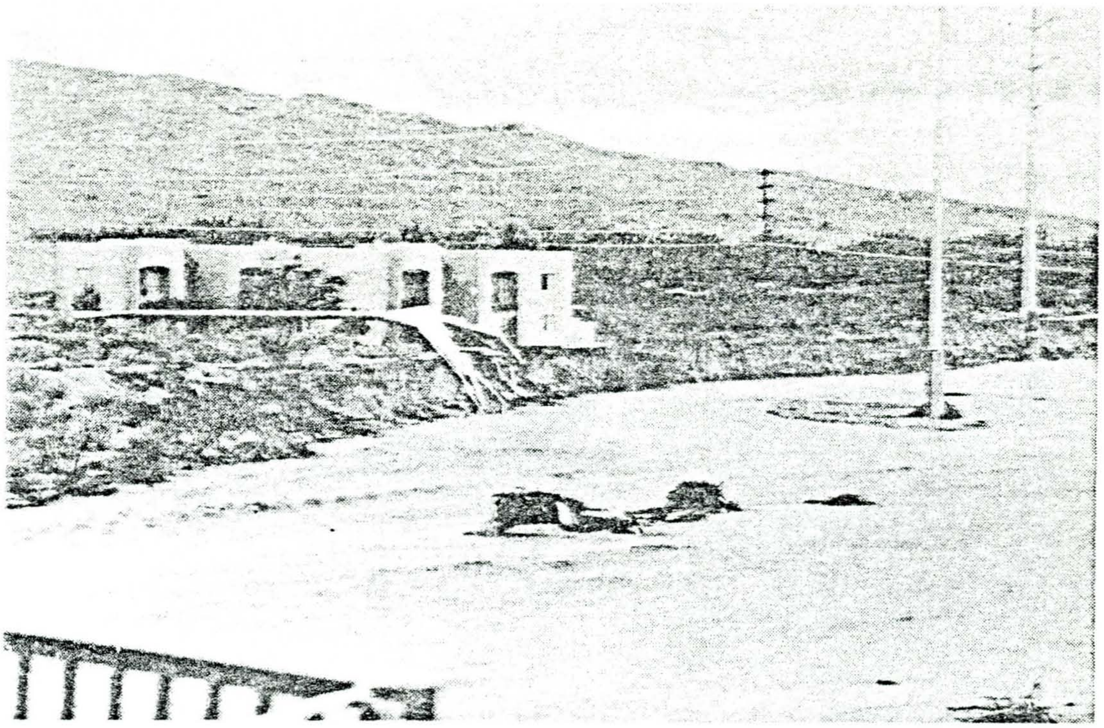


Figure 1.3. View from east bank looking west across the Cortaro Road bridge at the Santa Cruz River, Tucson, Arizona
(Photo date: October, 1983).

Figure 1.4. View from the west bank looking northeast across the Rillito River, Tucson, Arizona (Photo date: October, 1983).

non-essential, information has been cited by reference only. This approach allows the user who might be interested in details to locate the desired information, while allowing those who are not so interested in details to efficiently proceed through the design process.

Design manual organization provides a logical sequence of steps to guide the user from start to finish, both through individual elements of a single design and the overall integration of many elements of a comprehensive fluvial system analysis and design effort. Hydrologic Analysis (Chapter III) is the first major analysis after General Design Considerations (Chapter II). After completing the hydrologic analysis, information required as input for Hydraulic Analysis of Fluvial Channels (Chapter IV) is available. Similarly, results of this analysis are required prior to Sediment Transport Analysis (Chapter V). Chapter V completes the analysis component, providing the base-line data and knowledge necessary for application of various channel design techniques discussed in Chapter VI. To illustrate the integration of information resulting from each chapter, a comprehensive design example is given in Chapter VII.

The design manual is targeted for use by practicing engineers in the water resources field, or other individuals with equivalent knowledge or training. Consequently, an understanding of the basic concepts of hydrology and hydraulics has been assumed. Only that information necessary or essential to analysis of sediment transport is reviewed and/or provided in Chapters III and IV, resulting in a brief, highly-specialized treatment of the subject. Should additional information be required on general concepts, the user is referred to any hydrology and/or hydraulics textbook.

In contrast, subject material in Chapter V on Sediment Transport Analysis is presented in more detail. Beginning with Subsection 5.2, each subsection consists of three elements: DISCUSSION, APPLICATION, and EXAMPLE. The discussion material briefly describes the usefulness of the methodology and presents relevant theory and equations. The applications material presents information necessary to apply the methodology including rules of thumb and reasonable parameter values. Finally, an example is presented. Typically, it represents a simplistic case only intended to illustrate key points; however, when practical, these examples are based on case histories.

II. GENERAL DESIGN CONSIDERATIONS

2.1 Channel and Watershed Response

A generalized definition of the idealized fluvial system is the three-zone description provided by Schumm (1977). In this description, Zone 1 is the drainage basin, watershed, or sediment source area; Zone 2 is the transfer zone; and Zone 3 is the sediment sink, or region of deposition. The three subdivisions are based on the predominant processes occurring in each, since sediments are stored, eroded, and transported in all zones. Zone 1 involves primarily the upper watershed and various tributary watersheds that contribute to the channel network of Zone 2. Zone 3 concerns primarily the coastal region, since this is considered the ultimate deposition zone. Consequently, in the analysis of inland watersheds, such as those of Arizona, Zone 3 is not of immediate importance and the fluvial system is often redefined as the interaction of the watershed and the alluvial channel network. Figure 2.1 provides a conceptual drawing of the fluvial system as defined.

Limiting our scope to this definition of the fluvial system still defines a highly complex system involving the interaction of many natural processes. These natural processes, often referred to as physical processes, govern the response of the fluvial system to various inputs and/or disturbances. The two primary inputs are climatic factors and man's activities. The most important climatic factor for erosion/sedimentation analyses is precipitation, in the form of either rain or snow. Man's activities include water resources development, watershed conversion, resource acquisition (energy, sand/gravel, etc.), development and operation of transportation systems, etc.

The response of the fluvial system to these inputs and/or disturbances is governed by the relevant physical processes. For example, the physical process describing soil detachment from raindrop impact is important in evaluating system response to precipitation. The physical process of overland flow, described by the interaction of such factors as slope, roughness, and precipitation excess, defines watershed response by establishing sediment transport supply available during a given precipitation event. Similarly, within the channels of the fluvial system, the physical processes describing sediment transport capacity establish whether or not the channel will aggrade or degrade in response to the precipitation-generated water and sediment runoff.

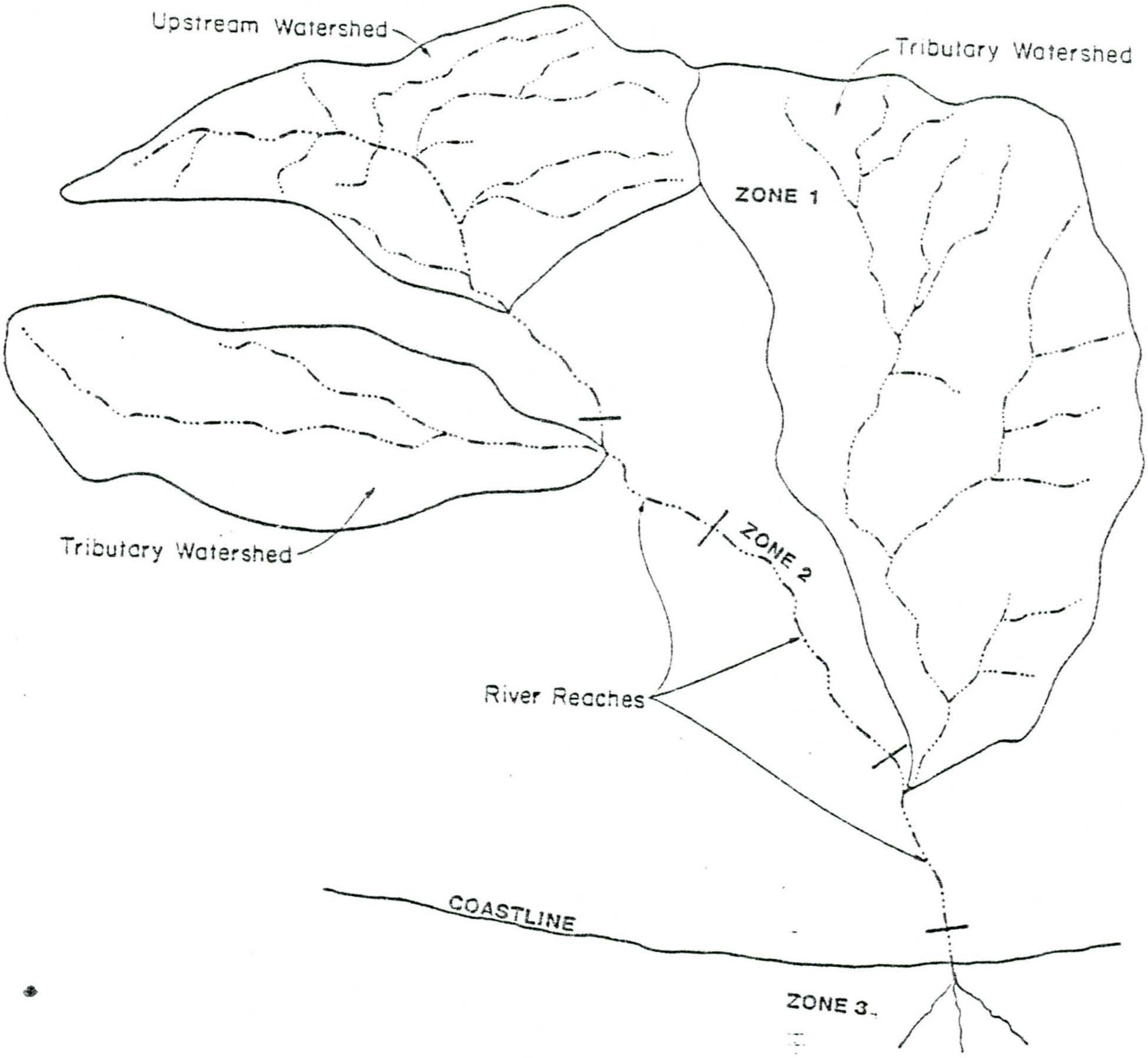


Figure 2.1. Watershed-river system.

Throughout all these events man's activities will modify fluvial system response by influencing the governing physical processes. Perhaps the most important concept to realize about fluvial systems is that they are dynamic systems attempting to achieve a state of balance or equilibrium. Consequently, the fluvial system is either adjusting to altered conditions or is in a state of dynamic equilibrium with present conditions. In either case, natural and man-induced changes can initiate responses that may be propagated through long periods of time or large areas. This dynamic nature requires that the analysis of problems (even on a small, localized scale) and development of solutions be considered in terms of the entire system. A classic example illustrating the dynamic nature of the fluvial system is the implementation of flood control reservoirs or debris basins. These structures can induce downstream degradation by limiting the delivery of upstream sediments. The dynamic action-response mechanisms of fluvial systems must be acknowledged and incorporated into any analysis or design effort, small or large.

2.2 Sand-Bed Channels

The analysis and design of fluvial systems in sandy-soil regions presents unique problems not encountered with more well-developed soils. In this context, "sandy" is used in the engineering sense to describe loose, cohesionless soils. Sandy soils are most predominant in the semi-arid and arid regions of the country. In comparison, the higher precipitation of a more humid environment produces vegetation and soils that are well developed and stabilized. Under these natural conditions, streams carry low suspended sediment loads reflecting the stability in upland watersheds. Additionally, high precipitation produces a dilution effect on the sediments that are eroded.

Vegetation and land forms in arid and semi-arid regions reflect the lack of water. Compared with more humid regions, topography is more abrupt, hillslopes are usually steeper and shorter, and soils are thinner with little organic content. Dryland conveyances are usually incised, intermittent or ephermeral channels. When the channels do flow, it is usually in response to small storm cells of limited areal extent producing high-intensity, short-duration storms. This type of storm creates "flashy" runoff, producing both excessive erosion in upland watersheds and a pronounced capacity for sediment transport in the channel system. Due to high drainage density (number of channels per unit area), water and sediment runoff occurs very efficiently. Peak discharge is high, and time to peak and flow duration are short.

The combination of large sediment yield, large transport capacity and "flashy" runoff can cause rapid changes in the configuration of sandy-soil channels. These changes include lateral migration, scour, degradation and aggradation, and can cause changes in stream form, bedform, flow resistance and other geometric and hydraulic characteristics. Designing either a stable alluvial channel (one without a channel lining) or a stable, lined channel under such dynamic conditions requires a detailed understanding of sediment transport and stream channel response. For example, unlined channels must be designed to minimize excessive scour, while lined channels must be designed to prevent deposition of sediments. Channel linings in dryland areas are typically composed of some type of artificial stabilization due to the difficulties in growing the required type of vegetation. Unlined channels are most successful when designed under the concept of dynamic equilibrium, which simply allows for sediment transport conditions without scour. These topics and others are presented in detail in the following chapters.

2.3 Cobble-Bed Channels

The erodibility or stability of any channel largely depends on the size and gradation of particles in the bed. As water flows through a channel located in a well-graded alluvium (i.e. consisting of clay, silt, sand, gravel or boulders) smaller particles that are more easily transported are carried away while the larger particles remain. This process, referred to as armoring, results in what will be defined as a cobble-bed channel, although the particles remaining on the bed can be as small as gravels. Compared to the more uniformly graded sand-bed channel, cobble-bed channels are relatively stable; however, they are still moveable boundary channels that can experience significant change during floods. Therefore, one of the important factors in cobble-bed analysis or design is evaluation of the stability of the armor layer and the maximum discharge it can sustain without being disrupted.

Another category of cobble-bed channels, in addition to those developed through the armoring process, are the boulder-lined channels of steep mountainous regions. Except in very large floods, these channels are very stable, with water cascading through sections of rapids connected by pools. This characteristic of flow and the large size of the roughness elements inhibits analysis by the more common and familiar techniques applicable to relatively flat channels. When appropriate, brief discussions of analysis and design techniques for these very specialized conditions are presented.

2.4 General Solution Approach

2.4.1 Three-Level Analysis

The recommended solution procedure for sediment transport analysis generally involves three levels of analysis. The levels are defined as (I) qualitative, involving geomorphic concepts; (II) quantitative, involving geomorphic concepts and basic engineering relationships; and (III) quantitative, involving sophisticated mathematical modeling concepts. A qualitative Level I analysis provides insight into complicated fluvial system response mechanisms. The general knowledge obtained at this level provides understanding and direction to the Level II or III quantitative analysis. Additionally, the governing physical processes are usually identified in the general solutions of Levels I and II, allowing proper selection (or development) of a model for Level III that is efficient to use and applicable to the problems being analyzed. For long-term analysis where data are continually collected and/or updated, an iterative procedure of refinement becomes an important aspect of Levels II and III. As the data base becomes more complete and accurate, the type and level of analysis can become more sophisticated.

The three-level approach has been used extensively in the Southwest, and has been found to provide the most efficient analysis approach with the greatest accuracy for a given problem. The risk is minimized, since all results and conclusions are cross-checked to the other levels of analysis. The following paragraphs discuss some of the important concepts in each level of analysis.

2.4.2 Level I - Qualitative Geomorphic Analysis

The qualitative geomorphic analysis employed in Level I relies strongly on expertise and practical experience. Geomorphology is the study of surficial features of the earth and the physical and chemical processes of changing land forms, while fluvial geomorphology is the geomorphology (and mechanics) of watershed and river systems. Qualitative geomorphic techniques are primarily based on a well-founded understanding of the physical processes governing watershed and river response. Therefore, an important first step is to assemble and review previous work and data applicable to the study area, and for key project participants to become familiar with the study area. A site visit by key personnel ensures identification of important characteristics of the study area. Additionally, being in the study area and contacting the local interest groups concerned provides excellent insight and perspective for the study. Site visits are an essential element of a successful study.

After completing the necessary site visits there are a number of simplified concepts and procedures that contribute to a qualitative analysis. These include aerial photograph analysis, historical land-use patterns, and relatively simple relationships describing basic geomorphic concepts. The Level I analysis is discussed in detail in Section 5.2.

2.4.3 Level II - Quantitative Geomorphic and Basic Engineering Analysis

In Level I, geomorphic principles are applied to predict watershed and stream response and do not require detailed data, only a general understanding of the direction of change of the stream conditions. Geomorphic principles can also be applied to available data to more accurately evaluate watershed or channel responses. This analysis, when coupled with traditional analyses involving basic engineering relationships, allows an initial quantitative evaluation of response. Analysis techniques used in Level II involve evaluation of trends in the historical thalweg elevation, quantitative evaluation of bed and bank sediments, application of the Shields relation and other geomorphic/engineering relations, application of sediment transport equations and the sediment continuity principle, frequency analysis of water and sediment transport data, etc. Level II analyses can be completed by hand calculator; however, use of a computer can expedite some calculations. For example, the analysis of sediment continuity using appropriate sediment transport relations is also often completed with the aid of computer programs. A detailed discussion of the Level II analysis is presented in Section 5.3.

2.4.4 Level III - Quantitative Analysis Using Mathematical Models

The Level III analysis is the most accurate method of analysis and involves computer application of various physical-process mathematical models. A mathematical model is simply a quantitative expression of the relevant physical processes. Various types of mathematical models for sediment routing are available, depending on the application (watershed or channel analysis) and the level of analysis necessary. For example, channel models range from application of quasi-dynamic models (such as HEC-6 or HEC-2SR) to complicated dynamic sediment routing methods. In general, available models can be directly applied, or applied with minor modifications, to meet any project requirements. Criteria for electing to proceed with a Level III analysis are presented in Section 5.4.

2.5 Data Requirements

2.5.1 General

The quality and accuracy of any analysis are dependent on the data base available to the study. The type and number of data necessary depend greatly upon the sophistication of the analysis techniques (i.e., whether Level I, II, or III); however, for any analysis the level of effort required to establish the data base can represent a significant portion of the entire level of effort. The data base is developed from available data and any data collected during the project. Below is a brief discussion of the data requirements for each of the three levels of analysis.

2.5.2 Level I Data Requirements

The data required for a Level I geomorphic type analysis involves information on general trends and conditions describing the fluvial system characteristics, rather than specific, quantitative values. Some of the geomorphic relations used to qualitatively describe system action-response (the Lane relation or the slope-discharge relation) rely on estimates of dominant slope and/or discharge; however, due to the nature of the formulas and their intended applications, these numbers do not need to be accurate, refined values.

Other data required in a geomorphic analysis involve information describing historical trends or patterns. This information is generally interpreted on a qualitative basis, relying largely on personal experience and expertise. A typical example is the analysis of aerial photographs covering a span of several years. The amount of information extracted depends in part on the years covered. Similarly, insight derived from analysis of the flood history of a given drainage depends on the length of record available.

Table 2.1 summarizes some of the major data requirements of a Level I analysis.

2.5.3 Level II Data Requirements

Level II data requirements involve specific estimates of various parameters necessary to apply a range of quantitative geomorphic and basic engineering formulas. The data required might include specific, detailed numerical information on the watershed geometry (area, slope, length, drainage density, channel characteristics), sediment produced and delivered by the watershed (water discharge, sediment discharge, soil types, geology, representative particle sizes transported, gradation), man's influence (dams, sand and gravel extraction), and so forth. For many larger watersheds, data on these processes have been collected by various governmental agencies. The quality of the data and the length of record often vary so that careful evaluation is required to insure the data are useful for the purposes of the study. For example, most sediment discharge data have been collected only during low-flow periods; however, it is commonly accepted that the majority of sediment transport occurs during relatively short periods of high flow. Plotting low-flow sediment discharge data against water discharge for a given watershed generally produces poor results with no apparent trend. Consequently, data extrapolation or establishment of a descriptive equation for the watershed would appear impossible. However, if several additional high-flow data points were available, a distinguishable trend might be established (see Figure 2.2).

This situation can develop even when data have been collected over many years if no major storms occurred during the period of record. Therefore, the available data must be carefully interpreted and used to avoid erroneous conclusions. Additionally, the available data base is typically much smaller that that required to conduct the study. Consequently, the necessary additional data must be established by field measurement or by data generation techniques. A brief overview of several basic concepts in data generation is given in Section 2.5.5.

Some of the specific data requirements necessary to conduct a Level II analysis are presented in Table 2.2.

Table 2.1. Partial Listing of Data Requirements for a Level I Analysis.

General Channel Slope & Cross Section Characteristics
Representative (Dominant) Discharge
Bed and Bank Material Characteristics
Land-Use Changes
Major Structures and History
Aerial Photographs
Flood History
Fire History
Tectonic Activity

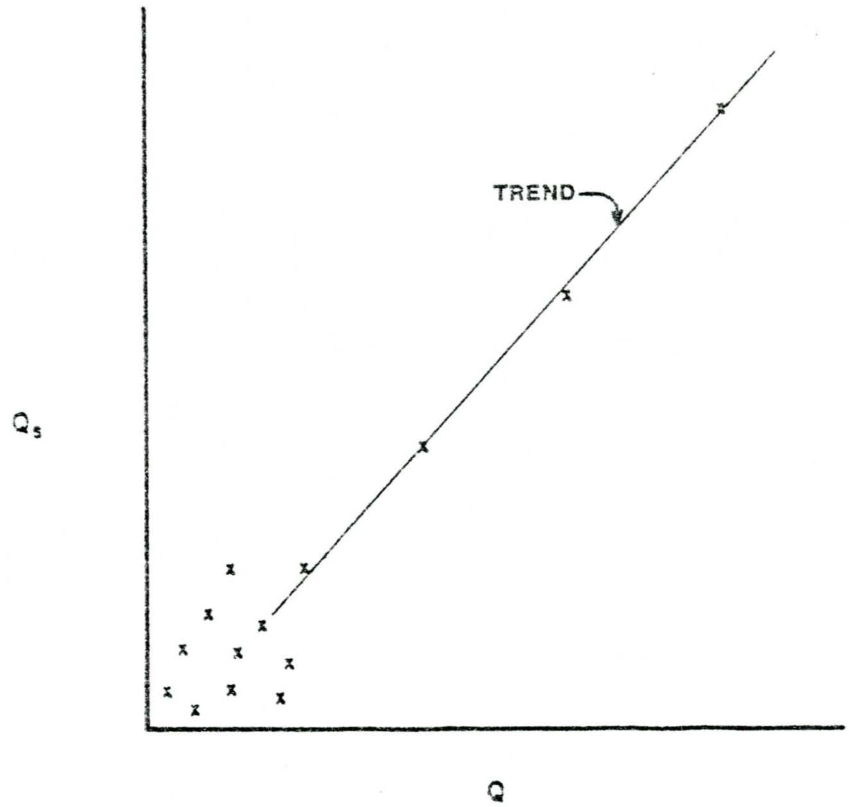


Figure 2.2. Definition sketch illustrating typical measured sediment discharges vs. water discharge relation.

Table 2.2. Partial Listing of Data Requirements for a Level II Analysis.

Watershed Geometry (Area, Slope, Length, Drainage Density)
Channel Geometry (Profile, Cross Sections, Sinuosity)
Hydraulic Data (Flow Depth, Velocity)
Water Discharge Records
Sediment Discharge Data
Discharge-Frequency Relations
Flood Hydrographs
Particle Size Gradations
Sand and Gravel Extraction Data
Reservoir Operating Procedure
HEC-2 Data/Runs
Reservoir Deposition Data

2.5.4 Level III Data Requirements

For any study involving physical-process mathematical modeling (Level III), it is necessary to define a spatial and temporal description that provides a realistic representation of the system for simulation purposes. This is particularly true for large-scale modeling where it is not practical to account for every possible inflow and outflow. Consequently, knowledge of the critical areas or areas of importance is necessary to develop the spatial representation. The actual data required to do this are not significantly different from those necessary for a Level II analysis, although more detail is often required for the mathematical modeling of Level III.

2.5.5 Data Generation Concepts

Data generation techniques can involve direct extrapolation and transposition of the available information, or indirect extrapolation through application of engineering relations based on the governing physical processes. The method of establishing the necessary additional information is determined by the priority or importance of the given area and the potential accuracy of the data generation methods available.

Data generation by direct extrapolation within a given watershed, or the transposition of data between watersheds, must be done properly to achieve accurate results. For example, transposition of sediment discharge data between watersheds cannot be accomplished accurately by assuming that a simple relation exists between water and sediment discharge rates (i.e., $Q_s \propto Q^b$), although this is generally an adequate relation for describing sediment transport rates within a given watershed without anticipated land-use changes. By considering the governing physical processes, one realizes that sediment transport is more directly related to individual hydraulic parameters, for example velocity and depth, which for a given discharge can vary significantly between various channels. Therefore, a better relation for describing sediment discharge for purposes of transposition of data is $Q_s \propto V^b d^c$. Consequently, watersheds that are similar in various erosion-related characteristics may allow adequate transposition of data by this type of relation.

Indirect extrapolation of data involves the application of a physically based engineering equation or relation. For example, one method to generate additional sediment transport data for a given watershed would be to use the available data to calibrate an applicable sediment transport equation or model, and then use the calibrated equation or model to generate new data.

The importance of understanding the governing physical processes in data generation is necessary for any variable, not just sediment discharge. Properly conducted data generation can provide accurate results that maximize the utility of available information.

III. HYDROLOGIC ANALYSIS

3.1 Relation of Hydrology to Other Analyses

Hydrologic analysis is a necessary first step to most water resource-related design projects. For example, the design of a spillway or flood-control channel is based on a design flood, where the characteristics of the flood depend on watershed and climatic variables. Similarly, hydrologic analysis is an important first step in fluvial systems analysis, since water is the driving mechanism for erosion and sediment transport. Knowledge of the runoff hydrograph provides the necessary information for determining runoff hydraulics at points of interest in the watershed or channel network.

Determination of runoff hydrology relies on evaluation of measured streamflow data or, in the absence of measured data, estimation of the runoff hydrograph through evaluation of the important physical processes. The latter is quite often the situation that the water resource engineer must deal with and the procedure involves a logical sequence of steps, beginning with the estimation of rainfall magnitudes corresponding to a specified return period and duration. After determining the relevant rainfall magnitudes, runoff volume is calculated by estimating losses, largely those due to infiltration. The volume of runoff is then used in conjunction with watershed characteristics to estimate a runoff hydrograph. The runoff hydrograph provides information on important variables such as peak discharge, flow duration, and time to peak. Methodologies also exist for direct estimation of these parameters, particularly peak discharge, without requiring the development of the runoff hydrograph.

It is not the objective of this chapter to provide a detailed discussion of the various methodologies or procedures available for a hydrologic analysis. Numerous textbooks and government publications are available with this information and it is not necessary to duplicate it here. Therefore, only a brief review of available and/or applicable techniques is provided. Adequate references are cited to allow the user to locate detailed discussions, as needed, of the various techniques.

The primary objective of this chapter is to illustrate some of the specialized applications of this hydrologic information when conducting fluvial systems analysis or design. These applications center on temporal considerations, both during a single flood (short term) and over many floods and/or years (long term). The more familiar application of hydrologic information in hydraulic structures design relies primarily on a single large flood event, the logic being that if the structure will withstand this flood, it will certainly withstand the smaller flows occurring between large events. However, with fluvial systems analysis and design, the cumulative effect of erosion/sedimentation occurring throughout all flows is important. While this cumulative effect is seldom as significant as a single large flood (it is often said that 90 percent of all river channel changes occur during ten percent of the flows), it can be an important component in some applications.

3.2 Establishing Return Period Discharges and Durations

3.2.1 General

The peak rate of runoff or peak discharge is a natural by-product of the determination of the runoff hydrograph. However, many hydraulic designs are based on direct estimates of peak discharge without requiring other hydrograph information. In general, computation of the hydrograph is the more satisfactory procedure; however, since many analyses use a peak-discharge approach, a few of the common approaches are included here and could be utilized when budget or other constraints necessitate a low level of effort.

Estimation of peak discharge is simpler than the procedures for development of the entire hydrograph. Determining the method to use depends on the available data and the applicability of a given relationship to the design conditions. For a gaged watershed the estimate is made by a hydrologic analysis of the drainage and stream, characteristics of the climate and the accumulated streamflow data. An efficient method to access data and conduct analysis on gaged watersheds is the U.S. Geological Survey (USGS) WATSTORE system. The WATSTORE system is a computerized data processing, storage, retrieval and analysis package for thousands of USGS-maintained stream gaging stations, water quality stations, sediment stations, water level observation wells and lake and reservoir monitoring stations. Typical analyses available through WATSTORE include frequency analysis and flow duration curves. Information on the availability of specific types of data, acquisition of data or products, and user charges can be obtained locally from USGS Water Resource Division district offices. Table 3.1 lists the district offices in the southwest geographical area, and also provides an address for general inquiries about WATSTORE.

To obtain information on gaged watersheds not maintained by the USGS, the NAWDEX program may be of value. The NAWDEX program, administered by the USGS,

Table 3.1. USGS Offices with WATSTORE Information.

Water Resource Division District Offices in Southwest Geographic Area
Tucson, Arizona
Menlo Park, California
Albuquerque, New Mexico

General Inquiries
Chief Hydrologist U.S. Geological Survey 437 National Center Reston, Virginia 22092

is a national confederation of water-oriented organizations working togetr to improve access to water data. Organizations involved with NAWDEX range from governmental (Federal, state and local) to academic and private. NAWDEX does not maintain the available data bases, but rather provides a variety of services to assist users in identifying, locating and obtaining the required data. The locations of local assistance centers in the Southwest for NAWDEX and for general inquiries about the system are provided in Table 3.2.

Development of hydrologic information from gaged watersheds is relatively straightforward; however, most smaller drainages are ungaged and an estimate of the design flow must be made on limited topographic and climatic data. Bibliographies by Chow (1962) and Reich (1960) identify and review many of the possible methods of estimating peak flows from ungaged watersheds. Some of the more common methods applicable to the Southwest are reviewed in the following paragraphs.

3.2.2 Rational Method

The Rational Method is a common method for peak flow estimation; however, it has many limitations that must be considered. These limitations are discussed by McPherson (1969) and others. Basically the equation $Q = C$... tends to oversimplify a complicated runoff process. However, because of the simplicity of the Rational Method, it remains widely used.

The assumptions used in developing the Rational Method are:

1. The rainfall occurs at a uniform intensity over the entire watershed.
2. The rainfall occurs at a uniform intensity for a duration equal to or greater than the time of concentration.
3. The frequency of the runoff equals that of the rainfall used in the equation.

The time of concentration t_c is defined as the time required for water to flow from the most remote (in time of flow) point of the watershed to the outlet, once the soil has become saturated and minor depressions are filled (Schwab, et al., 1966). Accurately evaluating the time of concentration is one of the major problems in using the Rational formula.

Reich (1971) cites references that indicate the potential of the Rational formula and that its prediction on the average was close to observed peaks,

Table 3.2. USGS Offices with NAWDEX Information.

Local Assistance Centers in the Southwest Geographical Area
Tucson, Arizona
Menlo Park, California
Albuquerque, New Mexico

General Inquiries
National Water Data Exchange (NAWDEX) U.S. Geological Survey 421 National Center Reston, Virginia 22092

although there is usually considerable scatter. The formula has generall been limited to watersheds of less than three square miles (2,000 acres).

3.2.3 SCS TR-55 Methods

The SCS has developed several methods that are commonly used for predicting runoff, ranging from peak flow estimation to complete hydrograph development. The method presented in SCS TR-55 (1975) is a graphical procedure for estimating peak discharges using the time of concentration and the travel time. This method is an approximation of the detailed hydrograph analysis produced by the computer program presented in SCS TR-20.

The graphical approach is applicable to a watershed where runoff characteristics are uniform and valley routing is not required. The relationship was developed by computing hydrographs for a one-square-mile drainage area, along with a range of times of concentration, and routing them through stream reaches with a range of travel times. A constant runoff curve number of 75 and a Type II (late peaking) rainfall sufficient to yield three inches of runoff were assumed.

The result of these computations is a curve relating the time of concentration t_c to the peak discharge in cubic feet per second per square m per inch of runoff, q_p. The curve is applicable for watersheds where the runoff can be represented by one curve number, CN, which implies the land use, soils and cover are similar and uniformly distributed throughout the watershed. As in the Rational method, accurate evaluation of the time of concentration is a major problem in application. The method is applicable for watersheds up to approximately 20 square miles in size. The runoff volume is obtained from a table and peak discharge is calculated from an equation.

A second graphical approach is presented in the SCS TR-55 publication for agricultural drainage areas up to 2,000 acres (three square miles). The method is reported to provide a quick and reliable estimate of peak discharge for most agricultural areas of the United States.

3.2.4 USGS Flood-Frequency Analysis

The U.S. Geological Survey (USGS) has developed graphical methods for determining the probable magnitude and frequency of floods of varying recurrence intervals for most of the United States. The graphs were developed on the basis of a comprehensive study of all flood data available in each reg by flood-frequency analysis. The relations are generally developed for rural watersheds and are based on gaging station records having ten or more years of record not materially affected by storage or diversion. Therefore, results obtained from this empirical, graphical procedure will represent the magnitude and frequency of natural floods within the range and recurrence intervals defined by the base data. The publication for the State of Arizona (Roeske, 1978) was developed as a joint effort between the Arizona Department of Transportation and the USGS.

3.2.5 Other Regionalized Methods

The literature contains many articles on experimental models for flood flow frequency estimation at ungaged locations. However, a literature evaluation by McCuen, et al. (1977) indicates that the literature does not adequately reflect what is currently being used. Instead, the literature contains many articles on experimental models that have been designed for a specific region or a specific problem. Thus, the volume of the literature on the techniques that are currently being extensively used (e.g., the Rational formula and the SCS technique) is not in proportion to the frequency of use of these techniques.

The use of a regionalized technique can often produce more reliable results than the more commonly used generalized techniques. However, care must be exercised in applying a regionalized method to ensure its validity to the given problem.

3.2.6 Channel Geometry Techniques

Several studies of alluvial stream channels of the western U.S. (Leopold, et al., 1964; Osterkamp and Hedman, 1981) have shown relationships between channel size and discharge characteristics. In perennial streams the active-channel level is nearly coincident with the stage corresponding to mean annual discharge. For ephemeral streams the active-channel capacity is usually more indicative of higher return flows, such as the 10-year flood. In general, fewer channel geometry relationships have been proposed for ephemeral streams, since there are few streamflow records of adequate length for analysis.

Greater accuracy can be achieved by considering sediment properties. Osterkamp and Hedman (1981) have presented groups of channel geometry equations according to channel type as characterized by the channel-sediment variables. They also demonstrate that consideration of channel gradient and discharge variability can improve discharge estimates.

This method can be a highly useful tool because (1) estimates are easily made, and (2) the channel size is a direct result of the water passing a given site, and thus a reliable index. However, care must be taken in selecting the site and the datum for the field channel measurements. As in all regression techniques, the accuracy of the mathematical relationships is dependent on the accuracy of the data base.

3.3 Development of Flood Hydrographs

3.3.1 General

Development of accurate flood hydrographs follows the logical sequence of steps reviewed in Section 3.2 (establishment of rainfall volume for design storm, determination of corresponding runoff volume and development of hydrograph considering watershed characteristics). This procedure accounts for the governing physical processes and is generally more accurate for peak discharge estimation than the methods reviewed in Section 3.2. Furthermore, any analysis involving routing of floods requires that the discharge hydrograph be known.

It is possible to approximate a hydrograph using a re-scaled or transformed record, i.e., re-scaling the recorded streamflow of an upstream gage by a ratio of drainage areas or by regression equations. This technique can provide acceptable results, particularly when a low level of effort is required, but when possible, hydrographs should be based on the governing physical processes. One of the most commonly used methods of hydrograph development is the Soil Conservation Service (SCS) unit hydrograph approach. This approach derives hydrographs from runoff calculations involving evaluation of precipitation amounts, interception, infiltration, surface detention, time of travel, etc. A brief review of the basic analyses for development of hydrographs is provided in the following sections, along with applicable methodologies.

3.3.2 Characterization of Design Storm

The first step in developing runoff hydrographs for an ungaged drainage is characterization of the design storm. The existence and length of record of rain gages and the size and location of the watershed determine the methods and considerations necessary in determining the character and magnitude of the storm. Since many designs are formulated in terms of return period, the volume of rainfall corresponding to a specified return period and duration must be determined by frequency analysis. In preparing a design the engineer is likely to choose one of two courses in calculating the volume of rainfall: (1) use data from an on-site gage, data suitably transferred from nearby gages with long records, or a combination of on-site and transferred data used to perform a frequency analysis, or (2) use one of the National Weather Service (NWS) publications that present the results of frequency analysis performed on their rain gage network in the form of isopluvial maps. Most statistical hydrology textbooks (e.g. Haan, 1977; Kite, 1977, Yevjevich, 1972) discuss methodologies for frequency analysis. Since most sites will not have on-site records of sufficient length, and due to the amount of work involved in synthesizing a record of sufficient length by transferring data, the second course (NWS Publications) is most likely to be used.

Currently there are three publications by NWS that are in regular use. In chronological order, they are: (1) Technical Paper No. 40 (TP40) by Herschfield (1961); (2) Precipitation-Frequency Atlas of the Western United States (11 volumes, 1973); and (3) "Five to 60-Minute Precipitation Frequency for the Eastern and Central United States (1977).. TP40 presents the results of depth-duration frequency analysis investigations for the contiguous United States performed by NWS and its precursor agency, the United States Weather Bureau. In addition, new studies for the high plains states appeared in this paper for the first time. The maps presented in TP-40 are considered most reliable for relatively flat regions. However, in the western United States, the mountainous terrain often causes large variations in precipitation. To correct this problem, the Precipitation-Frequency Atlas of the western United States was introduced in several volumes. This publication contains much larger-scale rainfall-duration-frequency maps than TP-40, and it corrects for such factors as slope, elevation, distance to moisture, location, normal annual precipitation, barriers to airflow and surface roughness, not included in TP-40. For Alaska, the publication used is TP-47, Probable Maximum Precipitation and Rainfall-Frequency Data for Alaska. Storms in the eastern United States are still characterized by TP-40; however, a more recent publication for storms of 5 to 60 minutes duration has recently been published by NWS (1977), under the title "Five to 60-Minute Precipitation Frequency for the Eastern and Central United States (HYDRO-55)." However, for 24-hour duration events, the rainfall atlas for the western United States and TP-40 and 47 are the most commonly used documents.

The procedure for using the NWS atlases to obtain point rainfall volumes is quite straightforward. Studies can utilize 24-hour or 6-hour duration storms for varying return periods (e.g. 2, 10, 25 or 100 years). In western states, isopluvial maps are printed for each of these storms. Determining the appropriate storm volume is a matter of reading the map at the watershed location.

The determination of rainfall volume is only the first step in characterizing the design storm. Obviously, if there is no water left after infiltration there will be no runoff and therefore no further need for concern regarding surface water hydrology. In small watersheds the character of the runoff hydrograph is largely determined by the character of the hyetograph and the infiltration properties of the drainage. Therefore, estimates of the peak runoff are quite sensitive to the temporal distribution of rainfall. The methods for distributing intensities over time are in common use and are standardized to some degree. However, they are somewhat subjective, requiring judgment on the part of the user.

To further compound matters, there are three distinct types of rainfall/ runoff events which can occur in Arizona. They are:

1. Convective Thunderstorms. Normally occurring in July and August, these storms are created by moisture that moves into the state from the Gulf of Mexico and combines with air movement from the heated mountainous terrain to produce intense, short-lived rainstorms. Often these storms are accompanied by thunder, lightning and strong, gusty winds. Generally their durations do not exceed one hour. However, upon occasion they have been known to continue for as long as six hours. Maximum areal coverage of individual storm cells is on the order of 90 to 100 square miles, but maximum rainfall amounts and intensities (sometimes exceeding 10 inches per hour) are usually confined to less than a two mile-square central core of rainfall. Historically, these types of storms generally have had their major impacts upon drainage catchments which are less than 25 square miles in areal extent.

2. General Summer Storms. Normally occurring in August and September, these storms originate off the west coast of Mexico as tropical storms or hurricanes and bring damaging winds and flood-producing rainfall into the state. Generally their durations range from one to four days, although they have been known to last as little as six hours and as long as ten days. Maximum areal coverage of general summer storms can easily exceed many thousands of square miles; however, maximum rainfall amounts and intensities are often concentrated within multiple isolated cells of less than 100 square miles in area. Historically, major impacts from these storms have generally occurred upon drainage catchments which range in size from 100 to 5,000 square miles.

3. General Winter Storms. Normally originating over the Pacific Ocean, these storms move rapidly eastward through the state. Precipitation from winter storms is usually of light or moderate intensity. At times, however, these winter storms have been the source of precipitation for the wettest years of record and have produced some of the most damaging floods. This has been especially true when warm rainfall has occurred over well-developed snowpacks in the higher elevations of the state, producing rapid runoff over large areas. Durations for winter rains range from a few hours to several days. Maximum areal coverage can exceed tens of thousands of square miles. The major drainage catchments in the state usually exhibit the most significant impacts from these winter storms, primarily because major catchments are fed by numerous tributaries, which cumulatively may constitute many thousands of square miles in watershed area.

There are basically two kinds of methods for constructing hyetographs from designated storm volumes. The first type is the construction of a synthetic storm through the use of depth-duration-frequency (DDF) curves or standard SCS or other regionalized rainfall distributions. When using the DDF curves, time intervals are selected and the rainfall intensity for each selected interval is computed by dividing the total amount of rainfall for that interval by the time of the interval. The result is the creation of several different rainfall intensities representative of finite time intervals during the storm. These intensities are then appropriately arranged by the user based on knowledge of local conditions. The ordering is the main source of subjectivity in these methods. Such synthetic methods have the advantage of at least providing a consistent set of rainfall intensities.

The second method is to use a storm record of many years from a nearby recording rain gage as the pattern for distributing rainfall. The difficulty in this approach is that the resulting intensities may be of differing return periods. Therefore, despite the intuitive advantage of having been recorded on site, the historical event does not provide a consistent approach to the formulation of a design storm.

3.3.3 Determination of Runoff Volume

Once the design storm has been specified, the next step in obtaining a runoff hydrograph is the determination of runoff volume. This calculation requires estimation of the effects of interception, infiltration and surface detention. With respect to rainfall on watersheds, especially in the western U.S., the most important of these processes is usually infiltration. The most commonly used method for determination of runoff volume is the SCS curve number approach. This method was developed for use with nonrecording rain gages; that is, the method is used to predict total volume of runoff from total volume of rainfall. In its publication, Design of Small Dams [U.S. Bureau of Reclamation (USBR), 1977], the USBR summarized and modified the SCS method for use with temporally arranged rainfall. This modification of the SCS method is suggested for the determination of runoff volume.

While the SCS method has gained wide acceptance and is in common use, serious errors can result in its application. In many cases methods that are based on field measurements are preferred. One of the most common of these methods is the Horton infiltration equation. Both the Horton and the SCS methods model the soil response independently of storm characteristics. This attribute, and the fact that the effects of watershed modification are difficult to reflect in either method, is leading many hydrologists to utilize more physically sound approaches, i.e., Green and Ampt (1911). Many of these approaches have been ignored in the past due to the necessity of laborious calculations. However, with the advent of extremely powerful small calculating devices, this objection is becoming obsolete (for example, see numerical solutions of Green-Ampt infiltration equation discussed by Li, et al., 1976). In the future it is likely that more physically based methods will be adopted.

Alternative infiltration approaches include methods based to a greater degree on infiltration methods at the site and that incorporate a mathematical description of the infiltration process. At this point it should be remembered that the SCS method is not entirely equivalent to calculation of infiltration. The SCS method strives to determine the retention characteristics of the drainage. Retention includes interception as well as infiltration. If one abandons the SCS method, some attempt must be made to determine losses to interception; however, infiltration is usually the most significant of these processes. In many western watersheds vegetation is so sparse that essentially all rainfall reaches the ground. Even where vegetative cover is relatively dense, infiltration is usually more significant as a hydrologic process than interception.

3.3.4 Hydrograph Development

A discussion of design flood analysis for small dams is presented in the USBR's (1977) "Design of Small Dams." This discussion includes development of unit hydrographs resulting from the runoff calculations discussed above. The USBR approach is based on runoff calculated by the SCS method; however, any method that produces temporally distributed excess rainfall provides the necessary information for calculation of a hydrograph. The user is referred to the USBR publication, or almost any hydrology textbook, for detailed procedures for unit hydrograph and triangular hydrograph analysis.

3.4 Selection of Design Event for Fluvial Systems Analysis

Selection of an appropriate design event for fluvial systems analysis is generally not as straightforward as it is for other water resource projects. For example, hydraulic structures design is usually based on a single large flood that the structure must withstand. The selection of the appropriate design event is generally based on an acceptable level of risk. By comparison, the selection of the design event for fluvial systems analysis depends largely on project objectives. For example, information on long-term cumulative erosion rates resulting from numerous floods over many years may be of interest. Conversely, the short-term erosion or scour occurring during a single event, for example at a bridge crossing, may be required. Therefore, temporal considerations established by project objectives will govern the selection of the design event.

For short-term analysis the single event is often a frequency-based flood, for example the 2-, 10- or 100-year event. Another possibility is the Probable Maximum Flood, defined as the most severe combination of critical meteorologic and hydrologic conditions that is reasonably possible in the region, or the Standard Project Flood, which results from the most severe combination of meteorologic and hydrologic conditions that is considered reasonably characteristic of the region, but excluding extremely rare combinations. Generally, the standard project storm rainfall amounts to approximately 50 percent of the rainfall for the probable maximum flood (Viessmann, et al., 1972).

For long-term analysis the objective is to evaluate the cumulative effects of a broad range of flow conditions. One approach that can be used is based on the concept of dominant discharge. The dominant discharge is that value which is predominantly responsible for the geometric characteristics of the channel. Although it is difficult to precisely establish the dominant discharge, the value is typically between the 2- and 5-year events for perennial streams and between the 5- and 10-year events for intermittent and ephemeral channels. The aggradation/degradation occurring for this dominant discharge is then assumed to represent the average annual value which can be extrapolated in time to evaluate long-term conditions (i.e., if the mean annual sediment delivery is 1,000 cubic yards, the total delivery over 10 years is 10 x 1,000, or 10,000 cubic yards).

A better approach than dominant discharge for long-term analysis of erosion/sedimentation is one which accounts for the probability of occurrence of various flood events during any one year. For example, if VOL_s is the sediment delivery at a specific location for a given flood and P is the probability of occurrence of that flood in one year, the product $VOL_s \times P$ represents the contribution of that one flood to the long-term mean annual delivery. To account for the contribution of all possible flows the integration

$$\overline{VOL_s} = \int_0^1 VOL_s \, dP \tag{3.1}$$

is required. This integration is best accomplished through use of frequency curve concepts. The frequency curve for sediment delivery can be estimated graphically by computing the sediment delivery expected for each of several floods of known return periods. Figure 3.1 illustrates the estimation of a sediment delivery frequency curve. The area under this curve (between the limits of 0 and 1) then represents the mean annual sediment delivery. This area can be computed graphically or numerically. The numerical procedure involves summing the incremental trapezoidal areas established by calculation of VOL_s for various return periods, with approximations for VOL_s at probabilities of 0 and 1 in order to satisfy the limits of integration defined by Equation 3.1. Assuming this calculation is completed for the 2-, 5-, 10-, 25-, 50- and 100-year events, the mean annual sediment delivery would be approximately

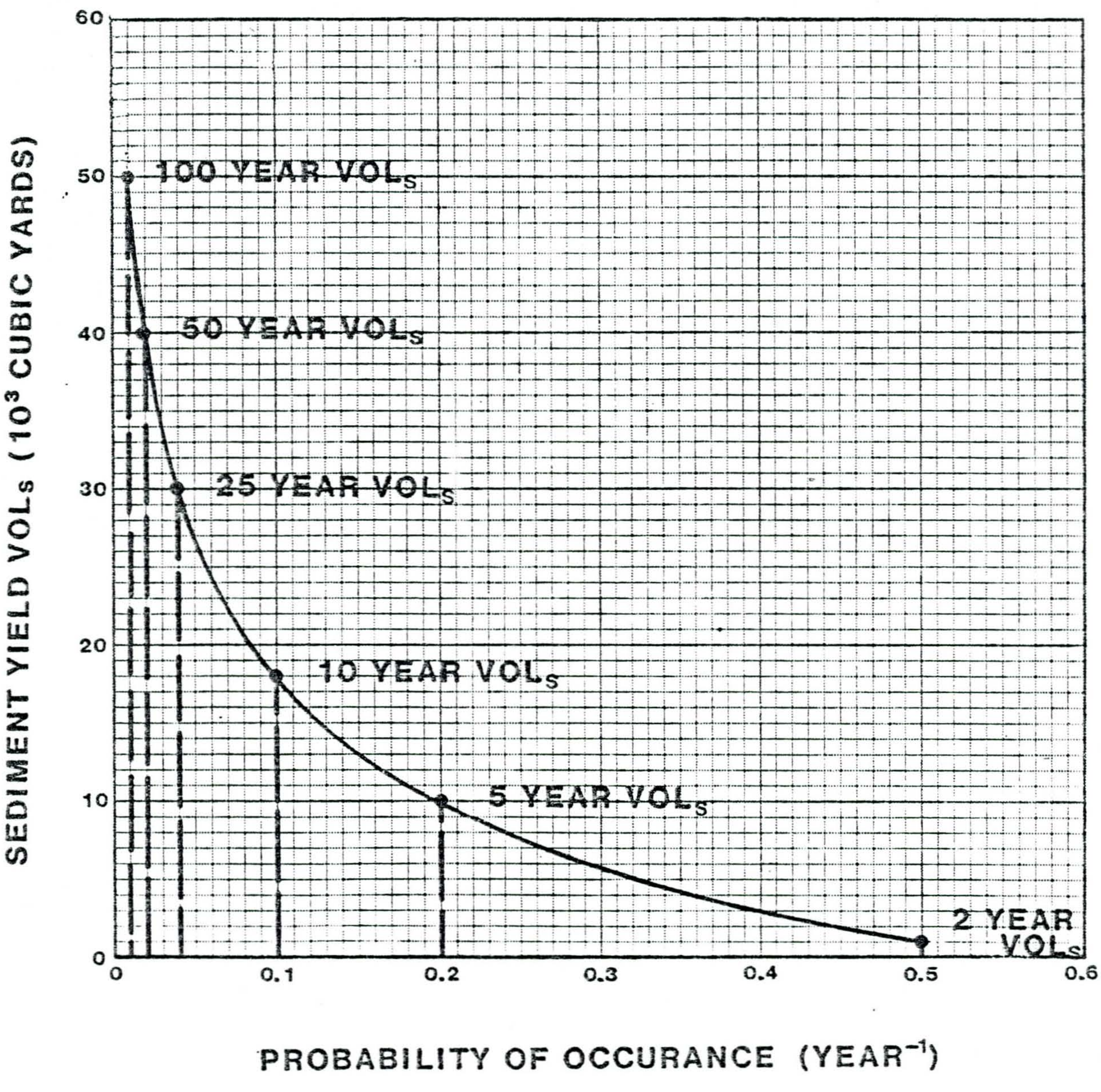


Figure 3.1. Typical sediment yield frequency curve.

$$(VOL_s)_m = 0.01\ (VOL_s)_{100} + 0.01\ (\frac{(VOL_s)_{100} + (VOL_s)_{50}}{2}) \qquad (3.2)$$

$$+ 0.02\ (\frac{(VOL_s)_{50} + (VOL_s)_{25}}{2}) + 0.06\ (\frac{(VOL_s)_{25} + (VOL_s)_{10}}{2})$$

$$+ 0.1\ (\frac{(VOL_s)_{10} + (VOL_s)_{5}}{2}) + 0.3\ (\frac{(VOL_s)_{5} + (VOL_s)_{2}}{2})$$

$$+ 0.5\ (\frac{(VOL_s)_{2} + 0}{2})$$

As a check on this calculation, it is useful to apply the weighting relationship (Equation 3.2) with the corresponding water discharge hydrographs and compare the calculated value to the mean annual water delivery as determined from stream gaging data. In an arid or semi-arid area, differences in these two estimates of long-term mean annual water yield may reflect numerical errors resulting from the trapezoidal rule approximation. Alternatively, it may reflect an inadequate record length of measured data or inadequate hydrological analysis in developing return period hydrographs. In a more humid environment, these same factors may be responsible for differences between measured and calculated water yield. Additionally, differences could result from base flows that are not adequately accounted for in the flood-based incremental probability calculation. For arid and semi-arid application, assuming adequate record length and hydrology, a correction factor for application to the probability weighted sediment delivery can be defined as

$$K = (\frac{VOL_{meas}}{VOL_{inc}})^2$$

where VOL_{inc} is mean annual water volume calculated from Equation 3.2, and VOL_{meas} is the mean annual water volume determined from gaging station data. The square of the ratio is taken since the relationship between water and sediment discharge is proportional to water discharge to the power of 1.5 to 2.0. Under the assumption of adequate record length and hydrology, the correction for numerical errors in evaluation of water yield should be relatively small, say no more than 10 to 20 percent. The maximum value for K would then be about 1.5. As a rule of thumb, this value should be assumed if the measured record length is extremely short (i.e., 10 to 20 years) or the calculated value of K is unusually large or small.

3.5 Discretizing Flood Hydrographs

To evaluate the cumulative erosion/sedimentation occurring during a flood, as will be discussed in Section 5.3.8, it is usually necessary to discretize the hydrograph. The only alternative to the discretization process is when the water discharge hydrograph can be approximated by a triangular hydrograph. Under these conditions calculation of the cumulative erosion/sedimentation can be simplified. When a triangular hydrograph approximation is not possible, it is necessary to discretize the water discharge hydrograph, which provides a series of constant discharges acting over short time intervals as illustrated in Figure 3.2. The hydraulic, erosion and sedimentation analyses are completed for each discharge level and weighted according to the time interval over which they occur. The cumulative erosion/sedimentation occurring during the flood is then the sum of the weighted values. For calculation purposes it is often efficient to maintain a uniform time interval. The discharge levels are then selected so that the total volume of the discretized hydrograph is not appreciably different from the original hydrograph (in other words, so that the incremental volume of the discretized hydrograph above the original hydrograph cancels the volumes not represented below the original hydrograph). This procedure is easily accomplished graphically (visually), which also allows slight adjustments to provide for convenient discharge levels.

Figure 3.3 illustrates the discretization of a flood hydrograph. The volume of the original hydrograph, determined by planimetering, is 4,456 acre-feet (AF), while the volume of the discretized hydrograph, determined by summing the incremental rectangular areas, is 4,473 AF. Table 3.3 summarizes the calculated sediment transport rates for the given discharge rates of the discretized hydrograph, as determined by techniques discussed in Chapter V. The total sediment delivery during the storm is then computed from the discretized hydrograph as:

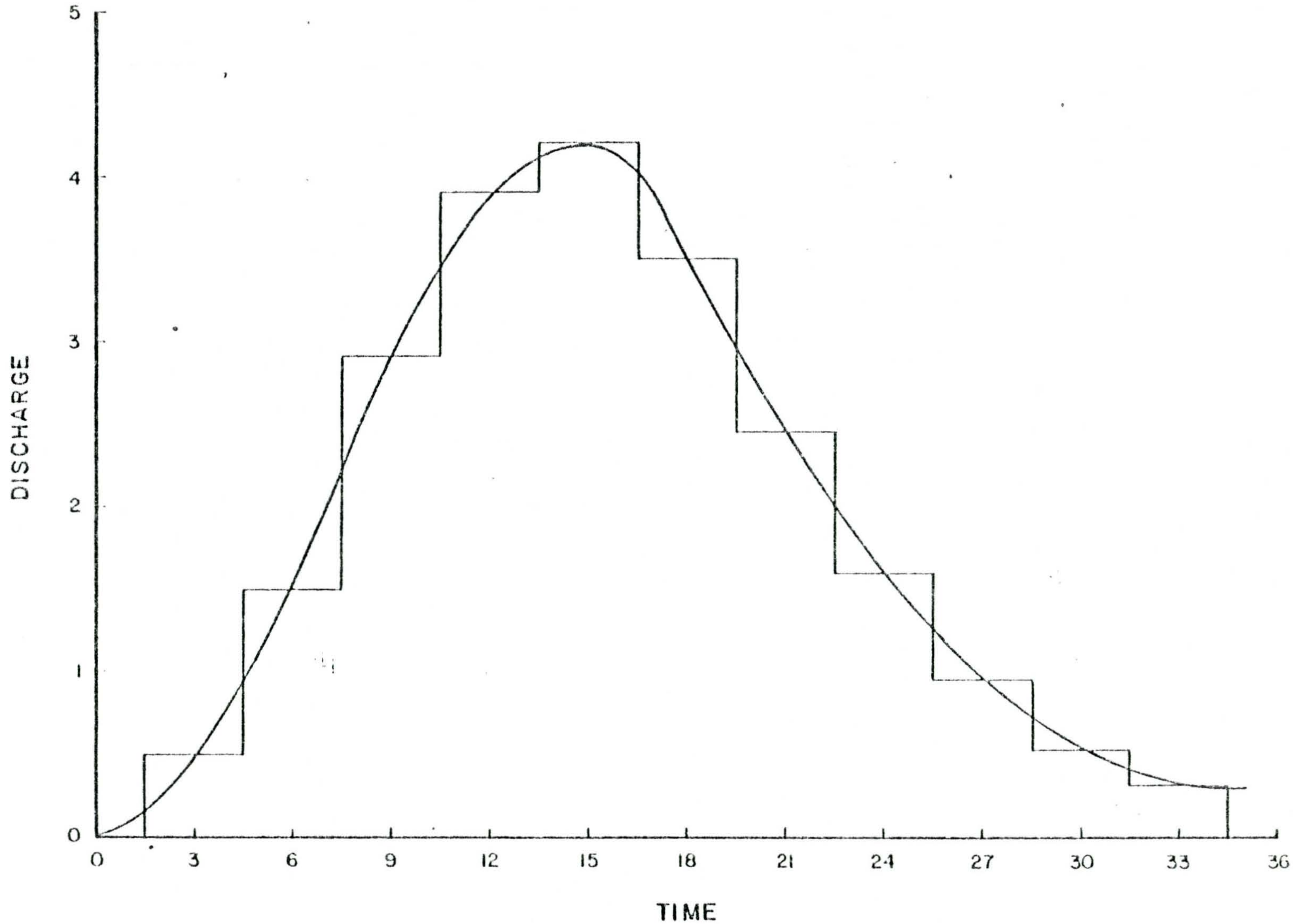


Figure 3.2. Definition sketch of the hydrograph discretization process.

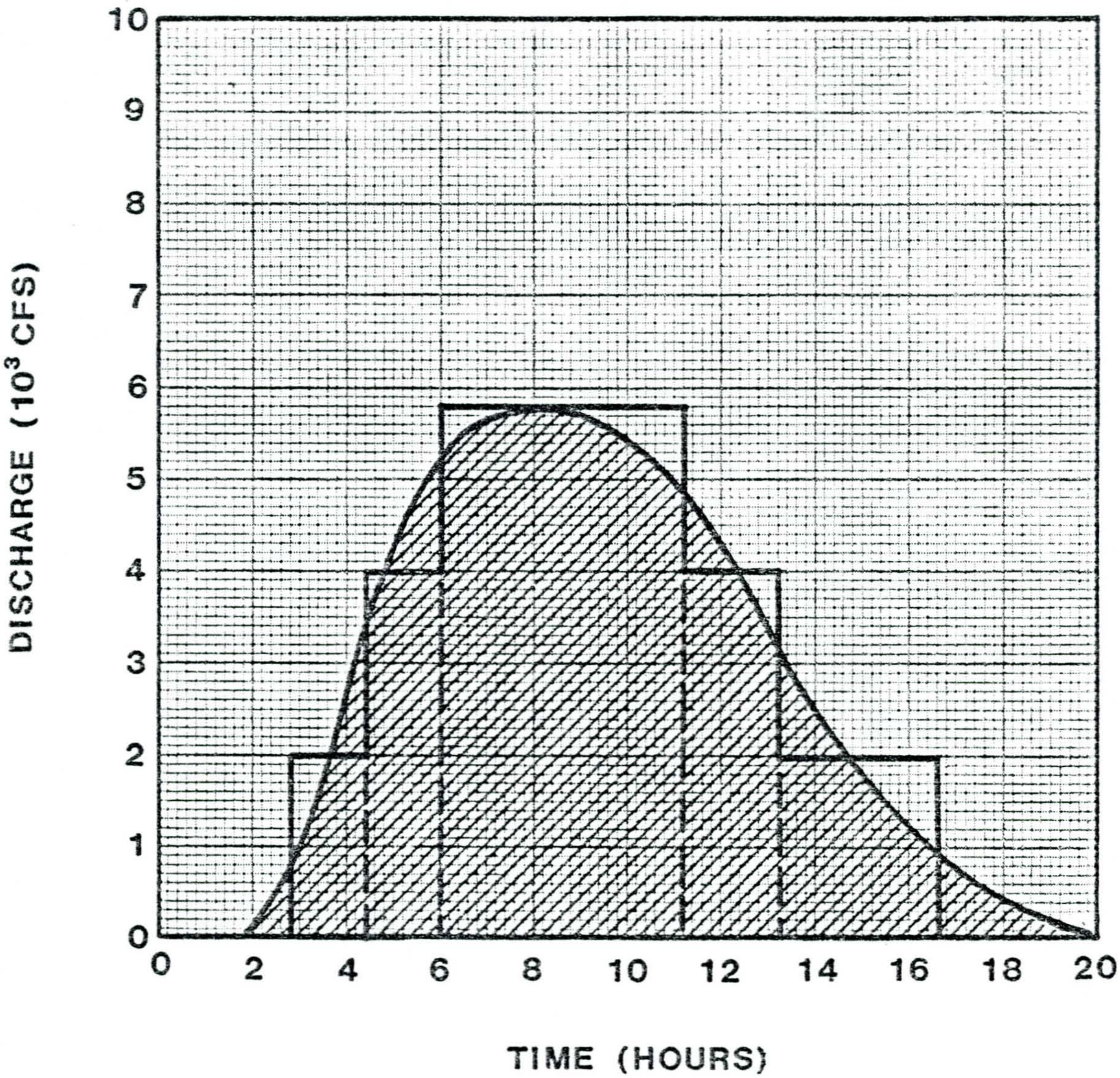


Figure 3.3. Hydrograph discretization.

Table 3.3. Water and Sediment Discharge Data for Hydrograph Discretization Example.

Water Discharge (cfs)	Sediment Discharge (cfs)
2,000	1.4
4,000	4.1
5,800	7.4

$$VOL_s = \sum_{\Delta t=1}^{5} Q_s \, \Delta t$$

$$= [1.4(1.4) + 4.1(1.6) + 7.4(5.4) + 4.1(2.0) + 1.4(2.8)] \frac{3,600}{43,560}$$

$$= 11 \text{ AF}$$

where

$\frac{3,600}{43,560}$ represents the conversion factor from cfs-hours to acre-feet.

IV. HYDRAULIC ANALYSIS OF ALLUVIAL CHANNELS

4.1 General

In open-channel flow the water surface is not confined, therefore surface configuration, flow pattern and pressure distribution within the flow will depend on gravity. In rigid-boundary open channels no deformation or movement of the bed and banks is considered. In alluvial channels, where the channel is located in a natural alluvium of silt, clay, sand and gravel, the bed and banks are free to move, and consequently channel characteristics will depend on flow conditions. Under these circumstances the concepts of moveable-boundary hydraulics must be utilized. In using procedures presented in this manual, it is assumed the reader has a working knowledge of methods to determine the erodibility of the channel bed and banks, and has applied that knowledge to the project under evaluation. Procedures for analyzing the erodibility of earth channels are presented in Technical Release No. 25 (SCS, 1977) as well as the Corps of Engineers, "Hydraulic Design of Flood Control Channels" (COE, 1970). It is assumed the engineer has applied these or similar procedures to his project and has determined the applicability of moveable boundary hydraulic/sediment transport procedures, such as those presented in this manual.

Understanding and utilization of the concepts of rigid-boundary hydraulics are essential for analysis of alluvial channels, and it is assumed that users of this manual have this knowledge. This chapter presents some of the more specialized knowledge surrounding moveable-boundary hydraulics as required for fluvial systems analysis.

4.2 Resistance to Flow

4.2.1 Common Resistance Parameters and Their Relationships

The three most common parameters for describing resistance to steady uniform flow are:

1. The Darcy-Weisbach friction factor f.

2. The Chezy resistance factor C.

3. The Manning roughness coefficient n.

The Darcy-Weisbach formula, developed primarily for flows in pipes, states that

$$h_f = f \frac{L}{D} \frac{V^2}{2g} \tag{4.1}$$

where h_f is the friction loss associated with the flow in pipes, f is the friction factor, L is the length of the pipe, D is the diameter of the pipe, V is the mean velocity of flow therein, and g is the acceleration of gravity.

Since $D = 4R$ and the energy gradient $S_E = (h_f)/L$, Equation 4.1 may be written in terms of the friction factor as

$$f = \frac{8gRS_E}{V^2} \tag{4.2}$$

where R is the hydraulic radius. Noting that $V_*^2 = gRS_E$, Equation 4.2 yields

$$\frac{V}{V_*} = \frac{8}{\sqrt{f}} . \tag{4.3}$$

Equation 4.3 can be applied to flow in open channels and sometimes is presented as

$$\frac{Fr}{\sqrt{S_E}} = \sqrt{\frac{8}{f}} \tag{4.4}$$

or

$$f = \frac{8S_E}{Fr^2} \tag{4.5}$$

where Fr is the Froude number $(\frac{V}{\sqrt{gY_h}})$.

The Chezy coefficient is related to Manning's n by

$$C = \frac{1.49}{n} R^{1/6} \tag{4.6}$$

and by definition (i.e., $V = C\sqrt{RS}$) to the Darcy friction factor f, since

$$\bar{\tau}_o = \gamma RS \tag{4.7a}$$

$$= f \rho \frac{V^2}{8} \tag{4.7b}$$

giving

$$C = \sqrt{\frac{8g}{f}} \tag{4.8}$$

where ρ is the density of water.

For the interested reader, it should also be noted that f and n are related as follows:

$$f = \frac{116.5\ n^2}{R^{1/3}} \tag{4.8a}$$

Several empirical formulas have been suggested that relate the bed-material particle size to Manning's n. For sand-bed channels, Meyer-Peter and Muller recommend

$$n = \frac{D_{90}^{1/6}}{26} \quad (D_{90} \text{ in meters}) \tag{4.9}$$

Lane and Carlson (1953), as a result of their San Luis Valley study, suggested the formula

$$n = \frac{D_{75}^{1/6}}{39} \quad (D_{75} \text{ in inches}) \tag{4.10}$$

where the beds of the canals studied were covered with cobbles. In a Highway Research Board publication, Anderson et al. (1970) recommend

$$n = 0.0395\ D_{50}^{1/6} \quad (D_{50} \text{ in feet}) \tag{4.11}$$

Engineers have varying preferences for resistance parameters. The parameter f is used for both open-channel and pressure flow. Additionally, f is dimensionally consistent, while the Manning n and Chezy C are empirically based. Consequently, the ASCE Task Force Committee (1963) recommended the use of the Darcy f for both open-channel and pressure flow.

However, the Manning n remains the most commonly used open-channel flow resistance factor. Use of Manning's n gives good results for fully rough and smooth conditions in rigid-boundary channels, but is less satisfactory for alluvial boundary flow, as its value is highly dependent on the form of bed roughness (see Section 4.2.2). Values of n for various kinds of rigid boundary surfaces have been tabulated and methods for determining the Manning's coefficient to account for a number of influencing factors such as cross section shape and channel irregularity are presented in numerous handbooks. V. T. Chow's Open Channel Hydraulics (1959) gives a detailed list of n values and methods of determining an n value in a complex channel section. A short summary of n values commonly used in alluvial conditions is given in Table 4.1.

4.2.2 Resistance to Flow in Fine-Grained Alluvial Channels

The equations developed in Section 4.2.1 assume flat-bed, rigid-boundary channels with no sediment transport and are strictly valid for these conditions only. A complicating factor in evaluating channel roughness in an erosion/sedimentation investigation is that the bed configuration of an alluvial channel seldom forms a smooth, regular boundary. Rather, it is characterized by shifting forms generated by the flow that vary in size, shape, and location as influenced by changes in flow, temperature, sediment load, and other variables. These bed forms constitute a major part of the resistance to flow exhibited by an alluvial channel and exert a significant influence on flow parameters such as depth, velocity and sediment transport.

Bed configurations that may form in an alluvial channel are plane bed without sediment movement, ripples, dunes, plane bed with sediment movement, antidunes, and chutes and pools. A detailed discussion of bed forms and their characteristics is provided by Simons and Senturk (1977) or Simons, Li & Associates, Inc. (1982).

The different bed forms are associated with two flow regimes, with a transition zone in between, used to classify flow in alluvial channels. The two regimes and their associated bed configurations are:.

A. Lower flow regime

1. Ripples
2. Dunes

Table 4.1. Manning Roughness Coefficients, n.

	Manning n Range
LINED OPEN CHANNELS:	
Gravel bottom, sides as indicated:	
Formed concrete	0.017-0.020
Random stone in mortar	0.020-0.023
Dry rubble (riprap)	0.023-0.033
UNLINED OPEN CHANNELS:	
Earth, uniform section:	
Clean, recently completed	0.016-0.018
Clean, after weathering	0.018-0.020
With short grass, few weeds	0.022-0.027
In gravely, soil, uniform section, clean	0.022-0.025
Earth, fairly uniform section:	
No vegetation	0.022-0.025
Grass, some weeds	0.025-0.030
Dense weeds or aquatic plants in deep channels	0.030-0.035
Sides, clean, gravel bottom	0.025-0.030
Sides, clean, cobble bottom	0.030-0.040
Dragline excavated or dredged:	
No vegetation	0.028-0.033
Light brush on banks	0.035-0.050
Rock:	
Based on design section	0.033
Based on actual mean section:	
a. Smooth and uniform	0.035-0.040
b. Jagged and irregular	0.040-0.045
Channels not maintained, weeds and brush uncut:	
Dense weeds, high as flow depth	0.08-0.12
Clean bottom, brush on sides	0.05-0.08
Clean bottom brush on sides, highest stage of flow	0.07-0.11
Dense brush, high stage	0.10-0.14

Table 4.1. (continued)

	Manning n Range
CHANNELS AND SWALES WITH MAINTAINED VEGETATION (values shown are for velocities of 2 to 6 fps):	
<u>Depth of flow up to 0.7 foot:</u>	
Bermuda grass, Kentucky bluegrass, buffalo grass:	
a. Mowed to 2 inches	0.045-0.07
b. Length 4 to 6 inches	0.05-0.09
Good stand, any grass:	
a. Length about 12 inches	0.09-0.18
b. Length about 24 inches	0.15-0.30
Fair stand, any grass:	
a. Length about 12 inches	0.08-0.14
b. Length about 24 inches	0.13-0.25
<u>Depth of flow 0.7-1.5 feet:</u>	
Bermuda grass, Kentucky bluegrass, buffalo grass:	
a. Mowed to 2 inches	0.035-0.05
b. Length 4 to 6 inches	0.04-0.06
Good stand, any grass:	
a. Length about 12 inches	0.07-0.12
b. Length about 24 inches	0.10-0.20
Fair stand, any grass:	
a. Length about 12 inches	0.06-0.10
b. Length about 24 inches	0.09-0.17
NATURAL STREAM CHANNELS:	
<u>Minor streams (surface width at flood stage less than 100 ft):</u>	
Fairly regular section:	
a. Some grass and weeds, little or no brush	0.030-0.035
b. Dense growth of weeds, depth of flow materially greater than weed height	0.035-0.05
c. Some weeds, light brush on banks . . .	0.04-0.05
d. Some weeds, heavy brush on banks . . .	0.05-0.07
e. Some weeds, dense willows on banks . .	0.06-0.08
f. For trees within channel, with branches submerged at high stage, increase all above values by	0.01-0.10

Table 4.1. (continued)

	Manning n Range
Irregular sections, with pools, slight channel meander; increase values in 1 a-e about	0.01-0.02
Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	
a. Bottom of gravel, cobbles, and few boulders	0.04-0.05
b. Bottom of cobbles, with large boulders	0.05-0.07
Flood plains (adjacent to natural streams):	
Pasture, no brush:	
a. Short grass	0.030-0.035
b. High grass	0.035-0.05
Cultivated areas:	
a. No crop	0.03-0.04
b. Mature row crops	0.035-0.045
c. Mature field crops	0.04-0.05
Heavy weeds, scattered brush	0.05-0.07
Light brush and trees:	
a. Winter	0.05-0.06
b. Summer	0.06-0.08
Medium to dense brush:	
a. Winter	0.07-0.11
b. Summer	0.10-0.16
Dense willows, summer, not bent over by current	0.15-0.20
Cleared land with tree stumps, 100-150 per acre:	
a. No sprouts	0.04-0.05
b. With heavy growth of sprouts	0.06-0.08
Heavy stand of timber, a few down trees, little undergrowth:	
a. Flood depth below branches	0.10-0.12
b. Flood depth reaches branches	0.12-0.16

B. Transition zone: bed configurations range from dunes to plane beds or to antidunes.

C. Upper flow regime

   1. Plane bed with sediment movement
   2. Antidunes

      a. Standing waves
      b. Breaking antidunes

   3. Chutes and pools

In lower flow regime, resistance to flow is large and sediment transport is small. Conversely, in upper flow regime resistance to flow is small and sediment transport is large. Figure 4.1 illustrates the variation of resistance to flow with bed form condition. Table 4.2 provides the range of resistance coefficients typical for each bed form and the recommended value for sediment transport analysis. The different values utilized for flood control versus sediment transport studies relate to the objectives of each study. Values in the upper range are used for flood control since a conservative estimate of flow depth is desirable. Values in the lower range are used for sediment transport, bank stability and riprap/revetment analysis since a conservative estimate of velocity is required.

Therefore, in order to properly select the Manning n of an alluvial channel, the bed form during the flood must be known. Figure 4.2 identifies bed form as a function of median fall diameter and stream power. Fall diameter may be approximated by the median diameter (D_{50}), which is known from particle size gradation analysis of a bed material sample; however, stream power, defined as the product of velocity and boundary shear stress ($\tau_0 V$) is a function of hydraulic conditions as determined by the water-surface profile calculations. Therefore, the analysis procedure requires first assuming a bed form condition in order to define Manning's n and then, after calculation, verifying that the assumed bed form was correct.

4.2.3 Resistance to Flow in Cobble/Boulder-Bed Alluvial Channels

When the relative roughness is large, such as in steep mountain rivers with cobble/boulder beds, the resistance problem has additional complications. Large-scale roughness exists when flow depth is the same order of magnitude as bed-material height. The velocity profile under these conditions is com-

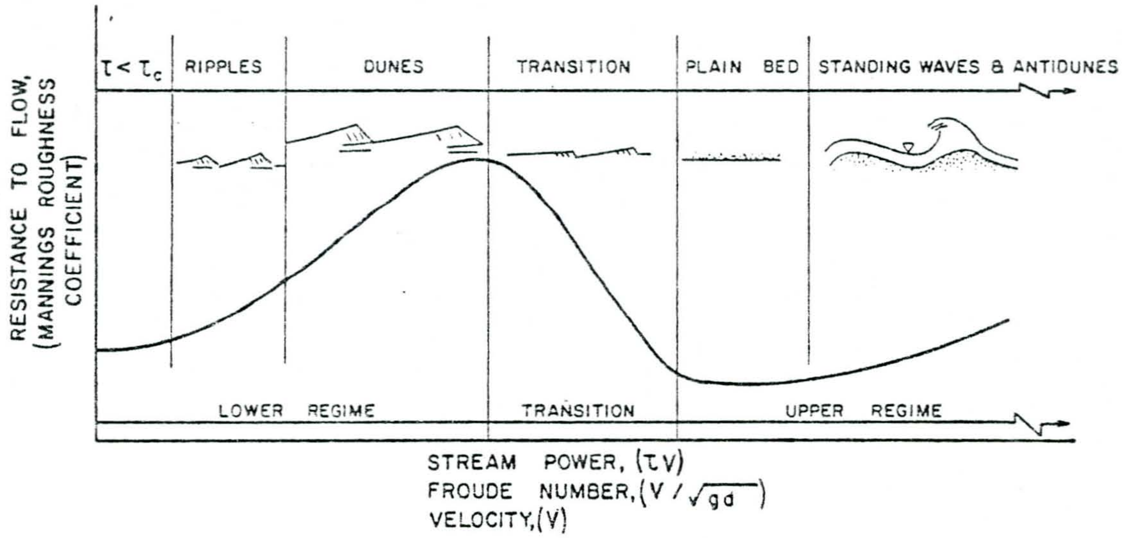


Figure 4.1. Forms of bed roughness in sand-bed channels.

Table 4.2. Values of Manning's Coefficient n for Design of Channels with Fine to Medium Sand Beds.

Bed Roughness	Typical Range	Recommended Value for Flood Studies	Recommended Value for Sediment Transport Studies
Ripples	0.018-0.030	0.030	0.022
Dunes	0.020-0.035	0.035	0.030
Transition	0.014-0.025	0.030	0.025
Plane Bed	0.012-0.022	0.030	0.020
Standing Waves	0.014-0.025	0.030	0.020
Antidunes	0.015-0.031	0.030	0.025

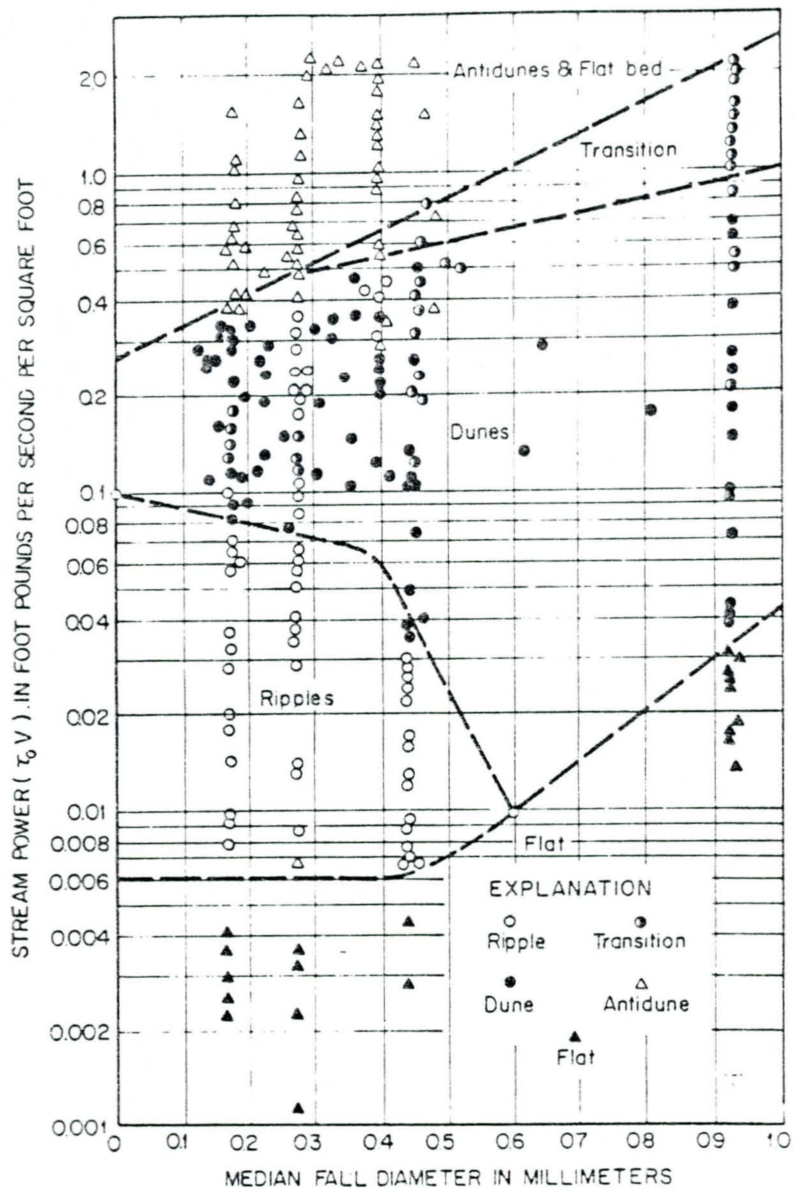


Figure 4.2. Relation of bed form to stream power and median fall diameter of bed sediment (after Simons and Richardson, 1966).

pletely disrupted and the roughness elements act individually, producing a total resistance based mainly on the sum of their form drags. Wall effects dominate the flow, so roughness geometry and distortions of the free surface around elements have the most effect on flow resistance. Channel geometry is indirectly important to the extent that it affects the flow around elements. Under these conditions the Manning equation cannot adequately describe flow conditions and a different resistance equation must be utilized. The following paragraphs describe a resistance equation for large roughness channels developed by Bathurst (1978), that should be used in place of Manning's equation for analysis of flow conditions in large roughness channels.

As discharge varies, relative roughness can change by an order of magnitude. Roughness height is represented by the length of the short axis of the bed material particles which is greater than or equal to fifty percent of the short axis of the bed material particles by count. The short axis is chosen since it more closely approximates the roughness height. A relative submergence (flow depth vs. roughness height) larger than about 15 corresponds to small-scale roughness. In this case roughness elements of the boundary act collectively as one surface, exerting a frictional shear on the flow. The shear is translated into a velocity profile, the shape of which is determined by roughness geometry, channel geometry, and any free surface distortions. Large scale roughness is considered to exist when relative submergence is less than about 4. The region between large- and small-scale roughness (relative submergence 4 to 15) is a transition region with intermediate-scale roughness. In this region flow resistance will be determined by some interaction of the two extremes.

As a result of these relative roughness relationships, different flow resistance equations may be required at the same section for different discharges. At low discharges, relative submergence will be low and cumulative form drag will be an important component to total resistance. At high discharges, relative submergence will increase and a small-scale roughness formula may become suitable. If large-scale roughness elements are removed during high flow, a sand-bed channel may be exposed. If this occurs, or if significant sediment transport occurs, the presence of bed forms should be anticipated. Such a sequence of events occurs when a cobble-bed armor layer is ruptured by high flow.

In order to provide data with which to develop a flow resistance equation for cobble/boulder channels, measurements from flume studies were compiled by Bathurst (1978). Measurements were made of flows over different roughness beds at a variety of slopes and discharges. Most of the measurements were made with fixed beds, but a few were made using loose beds in order to study the effect of bed-material movement on flow resistance.

Theoretical analysis, supported by results of the flume study, suggests that, for the range of Reynolds numbers given by $4x10^4 < V\ D_{50}/\nu < 2x10^5$, resistance is likely to fall significantly as Reynolds number increases. However, if there are roughness elements protruding through the free surface, the effect is small by comparison to Froude number effects related to the appearance of hydraulic jumps and generation of free surface drag. For the bed as a whole, free surface drag decreases as Froude number and relative submergence increase. Once the elements are submerged, Froude number effects related to free surface drag are small, but Froude number effects related to standing waves may be important.

The effect of roughness geometry can largely be described by a single parameter b_{rg}, the function of effective roughness concentration. This accounts for the variation of the roughness geometry both with depth and with bed material, although it does not make allowance for differing element shapes. Mathematically, b_{rg} is defined as follows:

$$b_{rg} = [1.175\ (\frac{Y_{50}}{W})^{0.557}\ (\frac{d}{S_{50}})]^{0.648}\ \sigma^{-0.134} \tag{4.12}$$

where Y_{50} = size of cross-stream axis of a roughness element which, by count, is greater than or equal to 50 percent of the cross-stream axes of a sample of elements

W = surface width of a section

d = mean depth normal to flow (use hydraulic depth, A/W)

S_{50} = size of short axis of a roughness element which, by count, is greater than or equal to 50 percent of the short axes of a sample of elements (note that the short axis is the shortest axis of the particle regardless of orientation, whereas the cross-stream axis is a function of how the particle is resting on the bed).

σ = standard deviation of the size distribution.

When only D_{50} data are available, Bathurst suggests that the median size of the short axis may be set equal to $0.57\ D_{50}$ and the cross-stream and long axes are equivalent, and equal to $D_{50}/0.57$. These values are considered most representative of bed material that is block-like in shape.

Similarly the effect of channel geometry is accounted for by the relative roughness area A_w/Wd' where A_w is the total wetted roughness cross-sectional area and d' = depth of flow from free surface to bed datum level. This parameter indicates the proportion of a channel cross section occupied by roughness, and therefore the degree of funneling of flow. For river channels of homogeneous boundary material, relative roughness area can be expressed as

$$\frac{A_w}{Wd'} = \left(\frac{W}{d}\right)^{-b_{rg}} . \tag{4.13}$$

Based on analysis of flume data, the resistance equation for large-scale roughness ($b_{rg} < 0.755$) is (Bathurst, 1978):

$$\frac{\bar{V}}{(gdS)^{0.5}} = \left(\frac{8}{f}\right)^{0.5} = \left(\frac{0.28}{b_{rg}}\,Fr\right)^{\log (0.755/b_{rg})}$$

$$\times \left[13.434 \left(\frac{W}{Y_{50}}\right)^{0.492} b_{rg}^{1.025\,(W/Y_{50})^{0.118}}\right] \times \left(\frac{A_w}{Wd'}\right) \tag{4.14}$$

This equation does not apply where Reynolds number effects (where viscous forces tend to damp out turbulence) are significant, where there is bed-material movement, or where there is a system of standing waves. However, within its range of application, the equation seems to work well as long as the various parameters, particularly the roughness sizes and the channel wetted perimeter, are derived or measured. In spite of its complex form, Equation 4.14 contains relatively few parameters and can be applied using a simple iteration procedure to evaluate flow conditions in large roughness channels, similar to the solution of the Manning equation for small roughness channels. The example at the end of the chapter illustrates application of the equation.

4.3 Boundary Shear Stress Calculations

Calculation of the boundary shear stress, or tractive force, is required in many alluvial channel computations. Consequently, it is important to know and understand the various methodologies that may be utilized to evaluate boundary shear stress. Equation 4.7a represents the basic theoretical equation for the mean boundary shear stress in a cross section as derived from application of the momentum principle to a control volume in uniform flow.

Equation 4.7b is derived from both Equation 4.7a and the Darcy equation as applied to open-channel flow ($D = 4R$). Consequently, the appropriate velocity to use is the mean channel velocity. Equation 4.7b is often preferred to Equation 4.7a for evaluating boundary shear stress, because it eliminates difficulties or uncertainty in defining the energy slope. Additionally, Equation 4.7b is more readily applied to evaluation of the mean boundary shear stress in overbank areas by using the mean overbank velocity.

The above equations (Equations 4.7a and b) define the mean boundary shear stress in the cross section. The variation of the boundary shear stress across the channel was first described by Lane (1955), as illustrated in Figure 4.3. This figure indicates that theoretically the boundary shear stress goes to zero at the corners of a channel; however, in reality it is more reasonable to assume that it is not zero, but rather some value less than the maximum value occurring on the channel sides or bottom. For design purposes, it is appropriate to base decisions on the maximum boundary shear stress occurring in the cross section, regardless of the specific location of interest, for example, at the toe of a riprapped channel side wall. For channels of different geometric properties, Figure 4.4 may be used to evaluate the maximum boundary shear stress on the channel sides or bottom, relative to γdS. It is important to realize that these figures are based on the boundary shear stress defined by γds, not the mean boundary shear stress in the cross section as defined by γRS (or $1/8\ \rho\ f\ V^2$). For channels of small width/depth ratio (i.e., less than 10), γdS will be larger than γRS. As the width/depth ratio becomes larger, γdS approaches γRS such that, for width/depth ratios greater than 10, they may be considered equal. Under this condition, as indicated by Figure 4.4, the maximum boundary shear stress and the mean boundary shear stress are equal on the channel bottom, while the maximum value on the side will be about 0.78 times the mean boundary shear stress. For application of Figures 4.3 and 4.4 to irregular channels, it is best to use the depth (d) defined by the hydraulic depth (A/T).

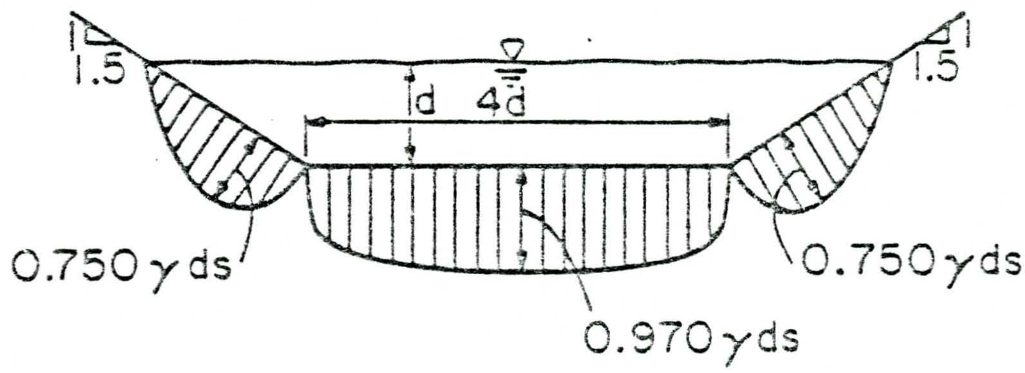


Figure 4.3. Variation of boundary shear stress in a trapezoidal cross section.

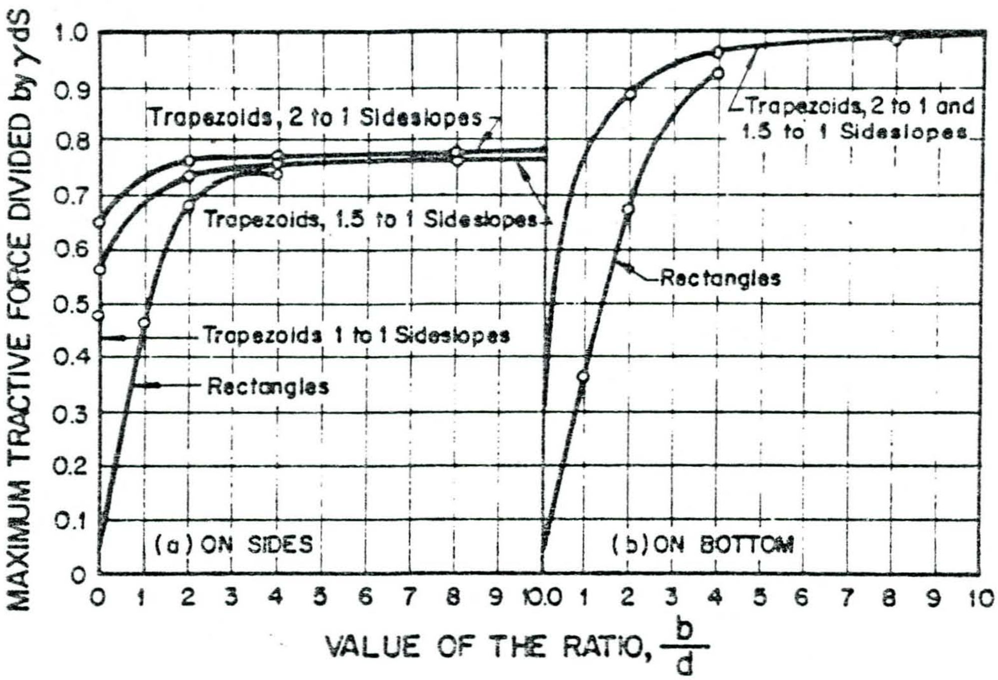


Figure 4.4. Maximum unit tractive force versus aspect ratio (b/d).

Equations 4.7a and b also define the mean boundary shear stress only for straight channels. Flow around a bend in a channel generates secondary currents that modify the velocity profile and boundary shear stress distribution; in particular, the boundary shear stress becomes greater on the outside of the bend. Figure 4.5 gives the ratio of the boundary shear stress on the outside of the bend to the mean boundary shear stress, relative to the radius of curvature of the bend.

4.4 Normal Depth Calculations

4.4.1 Definition

The hydraulic grade line, or the hydraulic gradient, in open-channel flow is the water surface, and in pipe flow it connects the elevations to which water would rise in piezometer tubes along the pipe. The energy gradient is at a distance equal to the velocity head above the hydraulic gradient. In both open-channel and pipe flow the fall of the energy gradient for a given length of channel or pipe represents the loss of energy by friction, excluding local miscellaneous losses. Figure 4.6 summarizes these definitions. When considered together, the hydraulic gradient and the energy gradient reflect not only the loss of energy by friction, but also the conversions between potential and kinetic energy.

In the majority of cases the objective of hydraulic computations relating to flow in open channels is to determine the elevation of the water surface, from which other hydraulic parameters at any desired location may be easily computed. These problems involve three general relationships between the hydraulic gradient and the energy gradient. For uniform flow the hydraulic gradient and the energy gradient are parallel and the hydraulic gradient becomes an adequate basis for the determination of friction loss, since no conversion between kinetic and potential energy is involved. In accelerated flow, the hydraulic gradient is steeper than the energy gradient; and in retarded flow the energy gradient is steeper than the hydraulic gradient. An analysis of flow under these conditions cannot be made without consideration of both the energy gradient and hydraulic gradient.

The depth of flow existing under conditions of uniform flow is defined as the normal depth. Uniform flow develops when the flow resistance is just balanced by gravitational force. Under these conditions the slope of the energy grade line S_E is equal to the bed slope, S_o. The normal depth is

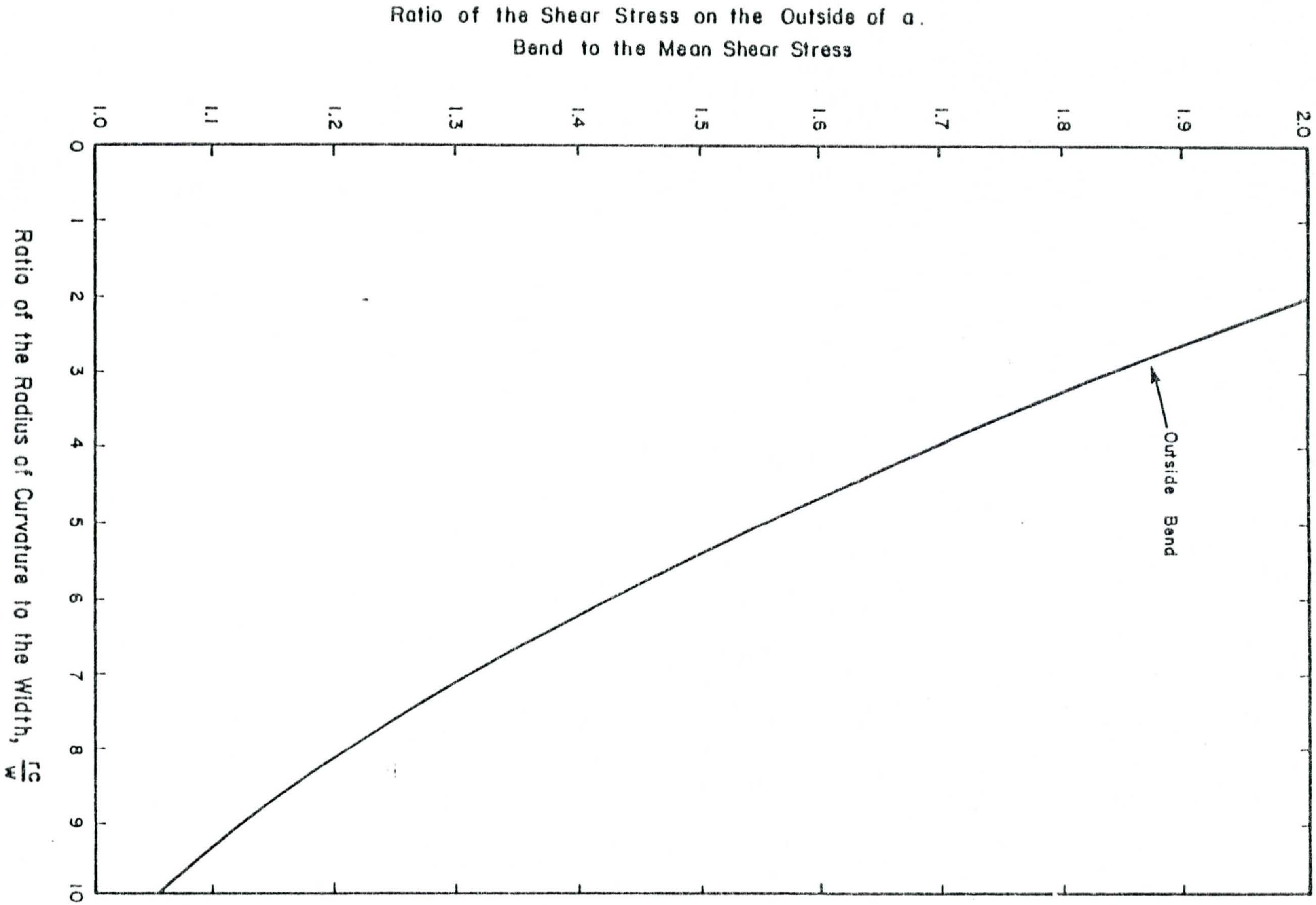


Figure 4.5. Effect of bend on boundary shear stress (after Soil Conservation Service design manual).

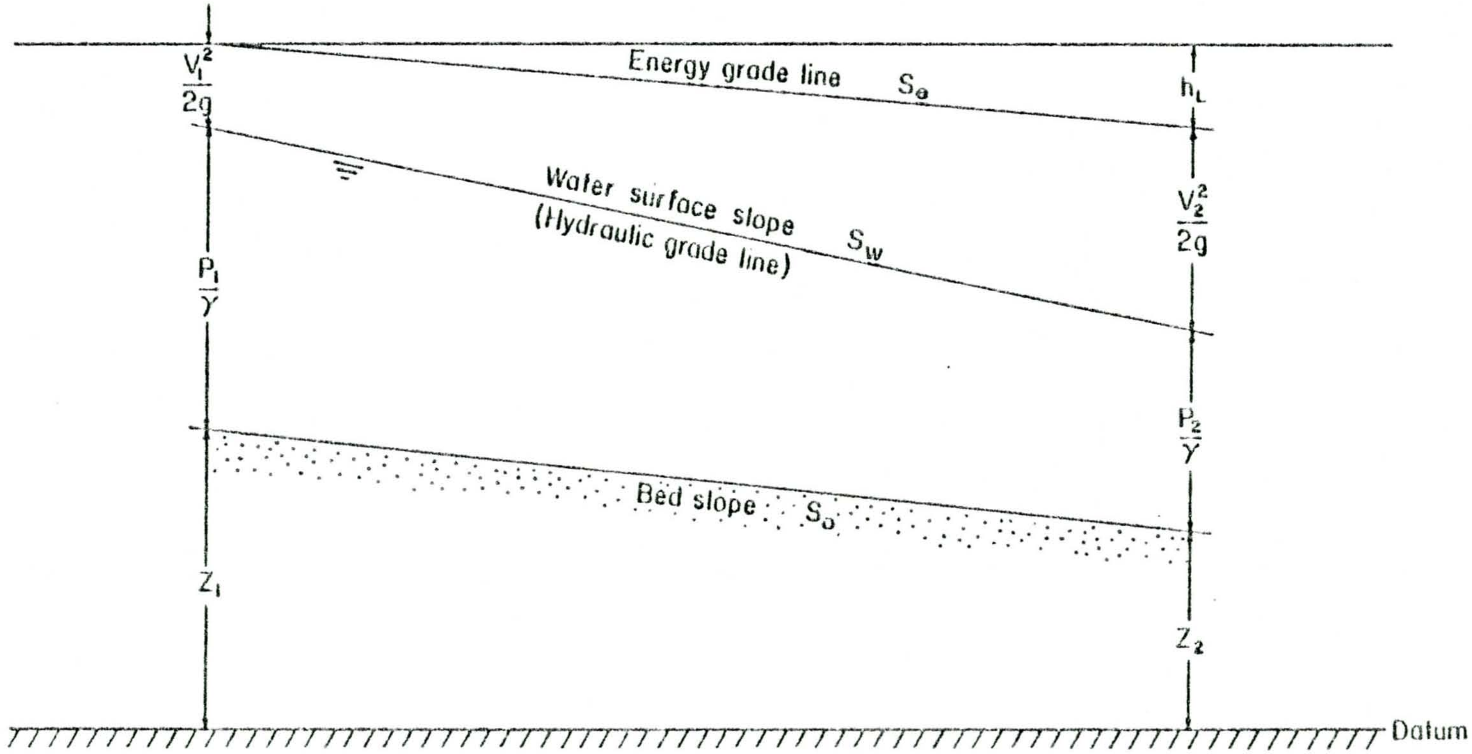


Figure 4.6. Definition sketch of the energy and hydraulic grade lines in open-channel flow.

frequently of interest, particularly when calculations of the water-surface profile are required (water-surface profiles are discussed in the next section of this chapter). The type of water-surface profile existing in a given situation depends on the relationship existing between the normal depth, the critical depth, and the existing depth of flow for a given discharge. In this section normal depth calculations in trapezoidal and natural channels will be discussed. Uniform flow very seldom exists in natural channels; however, in practice, this assumption is frequently made.

4.4.2 Normal Depth Calculation for Trapezoidal Channels

Manning's equation can be written for discharge as

$$Q = \frac{1.486}{n} AR^{2/3} S^{1/2} . \tag{4.15}$$

Area and wetted perimeter for a trapezoidal channel may be expressed as a function of depth as follows:

$$A = z\, y^2 + by \tag{4.16}$$

where z describes the side slope as the ratio of horizontal to vertical distance, b is the bottom width and y is the depth. Wetted perimeter is given by

$$P = b + 2y\,(1 + z)^{1/2} . \tag{4.17}$$

Therefore, the discharge for a given normal depth, y_0, is

$$Q = \frac{1.486}{n} \frac{(z\, y_0^2 + by_0)^{5/3} S^{1/2}}{[b + 2y_0(1+z)^{1/2}]^{2/3}} . \tag{4.18}$$

For a known discharge this equation may be solved for normal depth y_0 in terms of the other known parameters by use of an iterative technique such as Newton's iterative method. The equation actually solved, in this case for y_0, would be

$$\frac{nQ}{1.486\, S^{1/2}} = \frac{[zy_0^2 + by_0]^{5/3}}{[b + 2y_0\,(1+z)^{1/2}]^{2/3}} . \tag{4.19}$$

4.4.3 Normal Depth Calculation for Natural Channels

Using data taken at a given cross section, wetted perimeter P is often related to cross-sectional flow area A by regression. The resulting expression is usually a power function of the form

$$P = a_1 A^{b_1} \ . \tag{4.20}$$

Similarly, flow area may be related to flow depth as

$$A = a_2 y^{b_2} \ . \tag{4.21}$$

Here, a_1, a_2, b_1, and b_2 are statistically fitted coefficients and exponents. By using these expressions, hydraulic radius R in Equation 4.15 may be expressed as a function of y as follows:

$$R = \frac{A}{P} = \frac{a_2 y^{b_2}}{a_1 \left(a_2 y^{b_2}\right)^{b_1}} = \frac{a_2}{a_1 a_2^{b_1}} y^{(b_2 - b_2 b_1)} \ . \tag{4.22}$$

Therefore, Equation 4.15 may be rewritten in terms of depth of flow in a natural channel as

$$Q = \frac{1.486}{n} a_2 y_o^{b_2} \left[\frac{a_2}{a_1 a_2^{b_1}} y_o^{(b_2 - b_2 b_1)} \right]^{2/3} S_o^{1/2} \ . \tag{4.23}$$

This equation may be solved directly for y_o, resulting in

$$y_o = \left[\left(\frac{Qn}{1.49 \, S_o^{1/2}} \right)^{3/2} \left(\frac{a_1}{a_2^{(5/2 - b_2)}} \right) \right]^{\left(\frac{2}{5b_2 - 2b_1 b_2}\right)} \ . \tag{4.24}$$

4.5 Water-Surface Profiles

Water-surface profile computations assume that changes in depth and velocity take place slowly over large distances, resistance to flow dominates and acceleration forces may be neglected. This type of flow is called gradually varied flow. Calculations under these conditions involve (1) the determination of the general characteristics of the water-surface profile, and (2) the elevation of the water surface or depth of flow.

In gradually varied flow, the actual flow depth y is either larger than or smaller than the normal depth y_0, and either larger than or smaller than the critical depth y_c. The water-surface profiles, which are often called backwater curves, depend on the magnitude of the actual depth of flow y in relation to the normal depth y_0 and the critical depth y_c. Normal depth y_0 is the depth of flow that would exist for steady uniform flow as determined using the Manning or Chezy velocity equations, and the critical depth is the depth of flow when the Froude number equals 1.0. Reasons for the depth being different than the normal depth are changes in slope of the bed, changes in cross section, obstruction to flow, and imbalances between gravitational forces accelerating the flow and shear forces retarding the flow.

In working with gradually varied flow the first step is to determine what type of backwater curve would exist. The second step is to perform the numerical computation of water-surface elevations. Open-channel flow textbooks, such as Chow's (1959) or Henderson's (1966), detail the analysis of gradually varied flow. Various computer programs have also been developed for application to gradually varied flow analysis, the most widely known of which is the U.S. Army Corps of Engineers HEC-2 program.

4.6 Additional Effects on Flow Depth in Alluvial Channels

4.6.1 Importance

Calculation of flow depth based on the assumption of gradually varied flow using a suitable roughness coefficient is not always sufficient in alluvial channels. Since the bed of the channel is not uniform and the alignment of the channel is sinuous, the flow depth will vary accordingly. Hydraulic structures whose performance depends on adequate clearance above the water surface must take into consideration additional effects. Bridges, levees, and man-made conveyance channels may suffer significant damage if they are designed on gradually varied flow depths alone. The depth of flow can be significantly affected by the formation of antidunes in upper regime flow, superelevation of the flow through a bend, and the accumulation of debris.

4.6.2 Antidune and Dune Height

For natural or man-made channel segments with sand beds, it is necessary to estimate the height of bed forms moving through the channel, particularly where freeboard or scour requirements are critical. This can be done by estimating antidune or dune height.

Antidunes can form in either the transition zone (between lower and upper regime) or upper flow regime (Simons and Senturk, 1977). Kennedy (1963) made a detailed study of antidune flow. He suggested that the wave length is generally given by $2\pi V^2/g$ (g is the gravitational acceleration) and two-dimensional waves break when the ratio of wave height to wave length reaches a value of approximately 0.14. This theory assumes that the depth of flow is roughly equal to the maximum height of the antidune. Thus, the antidune height h_a from crest to trough (see Figure 4.7) can be estimated utilizing the relation

$$h_a = 0.14 \frac{2\pi V^2}{g} = 0.027\ V^2 \qquad (4.25)$$

for $h_a < y$; assume $h_a = y_o$ when the calculated value of $h_a > y_o$, since h_a can never be greater than y_o.

Lower regime flow also produces bed forms which should be considered in designing levee, channel, or bridge projects. Based on data collected from flume experiments (Simons and Richardson, 1960), dune formations have been observed at Froude numbers ranging from 0.38 to 0.60. The ratio of depth of flow to dune height (d/h) ranged from 1 to 5. When this ratio is 1.0, the dune troughs could be depressed below the natural channel bed a distance equal to one-half the depth of flow. As a conservative guideline, this value (one-half the depth of flow) may be used to account for dune troughs forming adjacent to a structure.

4.6.3 Superelevation

There are many equations for determining superelevation, but the differences in computational results that are obtained by using the different equations are small. One equation that has proven to be applicable to a wide range of conditions was first presented by Ippen and Drinker (1962). When superelevation is defined as the water surface increase above the normal water surface (see Figure 4.8a), this equation takes the form:

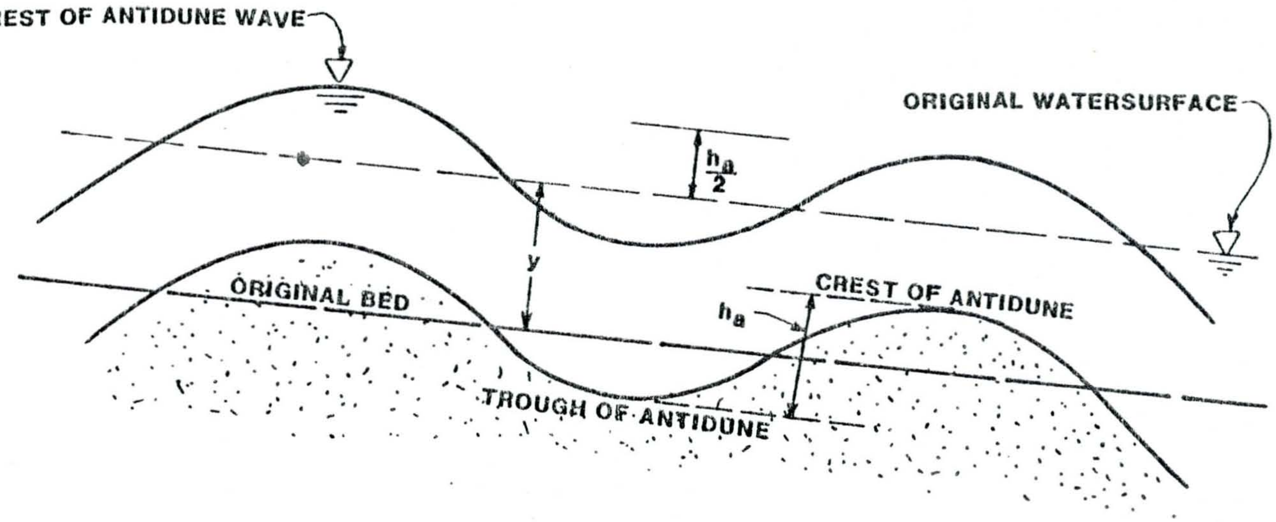


Figure 4.7 Definition sketch for antidune height.

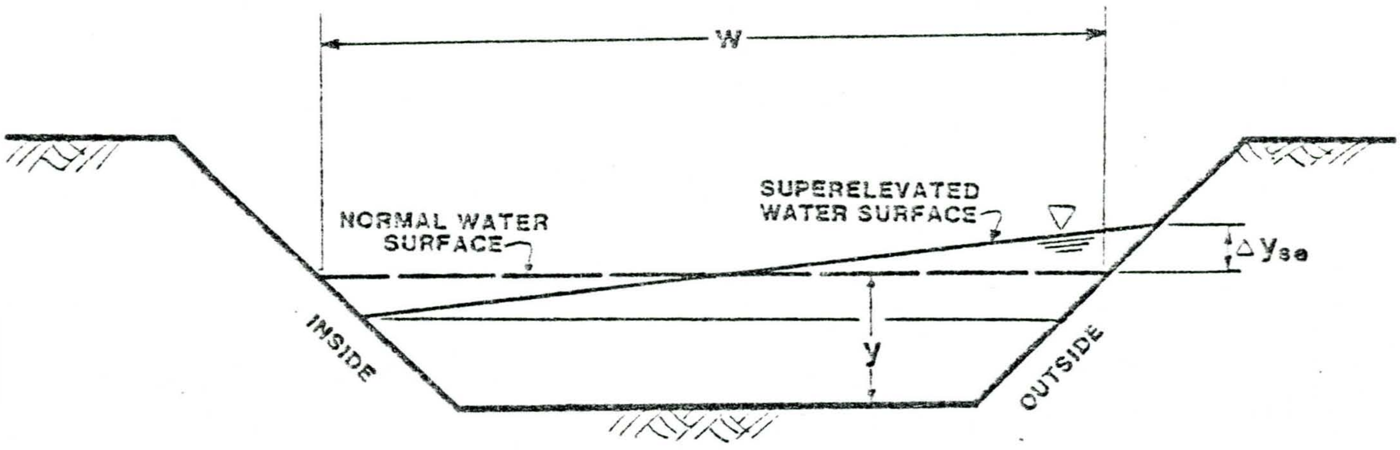


Figure 4.8a Definition sketch of superelevation in a channel bend.

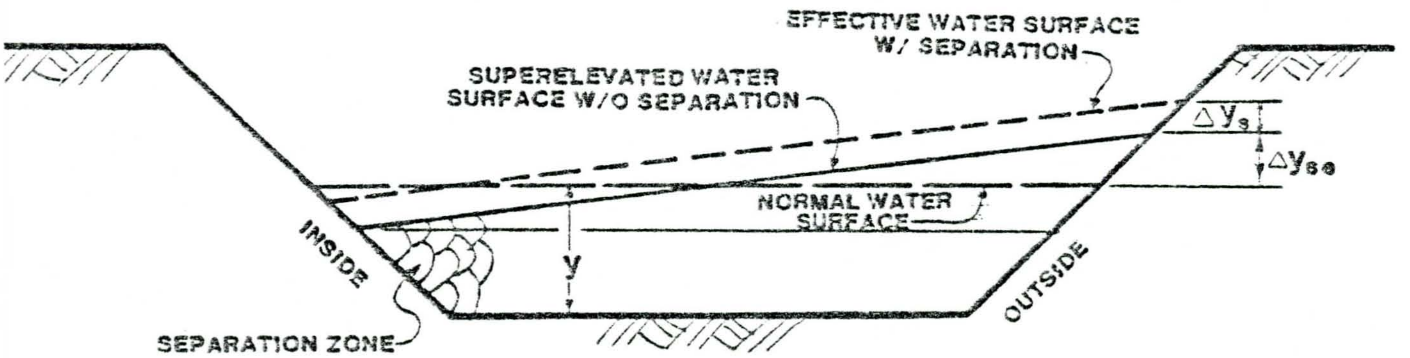


Figure 4.8b Definition sketch of superelevation and flow separation conditions in a short radius bend.

$$\Delta y_{se} = \frac{V^2}{2g}\frac{W}{r_c}\left(\frac{1}{1-\left(\frac{W}{2r_c}\right)^2}\right) \qquad (4.26a)$$

where Δy_{se} is the superelevation, r_c is the radius of the channel centerline, and W is the channel width at the elevation of the centerline water surface. When W/r_c is small (gradual curvature), Equation 4.26a simplifies to

$$\Delta y_{se} = \frac{1}{2}\frac{V^2 W}{g\, r_c} \qquad (4.26b)$$

A modified version of this equation was presented by the COE (1970) which incorporated a coefficient to account for channel and flow characteristics. The COE equation is

$$\Delta y_{se} = C\,\frac{V^2 W}{g\, r_c} \qquad (4.26c)$$

where the values of C are given in Table 4.3. It is recommended that Equation 4.26a be used for lined channels with sharp radii of curvature, and Equation 4.26c for natural, lined or unlined channels with gradual radii of curvature. It is also recommended that the values of C given in Table 4.3 be applied to Equation 4.26a as well. For purposes of this calculation, a sharp radius of curvature exists when W/r_c exceeds 0.33.

For sharp-radius bends subjected to high-velocity (near or greater than supercritical) flows, it may also be necessary to allow for an increase in the depth of flow as a result of flow separation in the bend. Flow separation from the inside boundary of the bend will reduce the effective cross-sectional area, induce deposition on the point bar, and locally increase the depth of flow (Δy_s). Conservatively, this can be taken as 25 percent of the velocity head, or

$$\Delta y_s = 0.25\,\frac{V^2}{2g}\;. \qquad (4.27)$$

The amount Δy_s is an additional depth component above the superelevated water surface, as illustrated in Figure 4.8b.

Table 4.3. Superelevation Formula Coefficients (from COE, 1970).

Flow Type	Channel Cross Section	Type of Curve	Value of C
Tranquil	Rectangular	Simple circular	0.5
Tranquil	Trapezoidal	Simple circular	0.5*
Rapid	Rectangular	Simple circular	1.0
Rapid	Trapezoidal	Simple circular	1.0*
Rapid	Rectangular	Spiral transitions	0.5
Rapid	Trapezoidal	Spiral transitions	1.0*
Rapid	Rectangular	Spiral banked	0.5

* NOTE: Equation 4.26c is based on the physics of flow in a rectangular channel. Due to the non-uniform flow distribution in a trapezoidal channel, it is recommended that these coefficients be multiplied by 1.15 if subcritical (tranquil) flow exists and 1.30 if supercritical (rapid) flow exists. This recommendation is based on information contained in the Hydraulic Design Manual published by the Los Angeles County Flood Control District.

4.6.4 Debris Accumulation

Natural rivers provide a good environment for the growth of trees and other phreatophytes. Channel banks, even in arid regions with intermittent stream flow, will support a significant number of large trees. The adjacent flood plain area will accumulate dead trees or debris from prior large floods. Both of these areas are capable of supplying floating debris to the main channel during large floods. Trees from the channel banks will be eroded in areas of active bank failure and dead trees in the overbank will be transported when the depth of flow becomes sufficient to float the debris. In urban areas, flood plain managers are faced with controlling a variety of floating debris.

Debris accumulation at bridge crossings can significantly influence bridge stability. The reduced conveyance resulting from partial blockage of flow area can increase flow depths and potential for overtopping. Additionally, since debris generally floats, it is the upper portion of flow that is restricted, which results in more flow of higher velocities near the bed. Therefore, debris accumulation can increase local scour and the potential for failure from undermining of piers and abutments.

There are no good rules to account for debris accumulation at bridge crossings. Quantification of the effect is largely subjective and relies on experience. In the absence of adequate data (watershed conditions, historical records, etc.), a generally accepted rule of thumb is to assume a debris accumulation equal to three times the pier width.

4.6.5 Total Freeboard Requirement

Freeboard is the vertical distance measured from the design water surface to the top of the channel wall or levee. In this definition, the design water surface is that resulting from uniform or gradually varied flow calculations (e.g. Manning's Equation or HEC-2 results, respectively). Freeboard is then any additional depth required to ensure overtopping does not occur in the as-built channel from factors not adequately accounted for in the design water surface calculations. These factors can include identifiable components such as long-term aggradation, superelevation, bed forms, and debris accumulation, as well as less identifiable components such as separation, excessive turbulence, variation in resistance or other coefficients used in design, and wave action. In degradational reaches it is not considered appropriate to reduce freeboard requirements due to the uncertainties in such things as bank stability. Under these circumstances the calculated freeboard will provide an extra factor of safety to account for potential channel instability.

Freeboard is often defined as a percentage of the depth of flow, plus any other increase due to indentifiable factors (superelevation, bed forms, or debris accumulation). For example, both the Soil Conservation Service (SCS) and the Bureau of Reclamation (BR) freeboard calculations are a function of flow depth. However, as discussed by the COE (1970), "The amount of freeboard cannot be fixed by a single, widely applicable formula. It depends in large part on the size and shape of channel, type of lining, consequences of damage from overtopping and velocity and depth of flow." In this regard, it is worthwhile to mention that both the SCS and BR procedures are primarily intended for application to smaller conveyances (i.e., irrigation channels, drainage ditches). For larger channels (i.e., rivers and floodways), the COE minimum guidelines are probably more applicable. These guidelines are (COE, 1970): 2.0 feet in rectangular cross sections and 2.5 feet in trapezoidal sections for concrete-lined channels; 2.5 feet for riprapped channels; and 3.0 feet for earthen levees. However, for riprap channels or earthen channels below natural ground levels, the minimum amounts may be somewhat reduced to reflect the lower hazard under these conditions.

When calculations for superelevation, bed forms, debris accumulation, and other identifiable variances to flow are available, an initial estimate of freeboard can be calculated. For channel walls below natural ground level, which incorporate an erosion-resistant bank lining such as soil-cement or riprap, it is recommended that the freeboard for the bank lining alone be computed as:

$$F.B_{BL} = \frac{1}{2} h_a + \Delta y_{se} + \Delta y_s \tag{4.28a}$$

The freeboard dimension for the total channel wall height (whether above ground or below ground) should include the following components:

$$F.B._{TOT\ EMB} = \frac{1}{2} h_a + \Delta y_{se} + \Delta y_s + \Delta y_d + \Delta y_{agg} \tag{4.28b}$$

h_a = antidune height defined by Equation 4.25

Δy_{se} = superelevation defined by Equation 4.26a or 4.26c, as appropriate

Δy_s = increase in flow depth from separation in short-radius bends (Equation 4.27)

Δy_d = increase in depth from debris accumulation

Δy_{agg} = increase in depth due to long-term aggradation (see Chapter V)

It is also recommended that the freeboard for bank lining (riprap, soil-cement, etc.) on above ground levee embankments be computed with Equation 4.28b. If excessive freeboard dimensions are computed with Equation 4.28b, the engineer should consider a redesign to eliminate causes of high freeboard.

If the river reach under study has a Federal Emergency Management Agency (FEMA) flood plain delineation, the minimum FEMA freeboard requirements must be complied with before channel or levee improvements will be recognized by FEMA as altering the original flood plain delineation. Under these circumstances, if the freeboard dimension calculated by Equations 4.28a or 4.28b is less than the minimum FEMA requirements, the FEMA criteria should be used. In the absence of FEMA regulation, the final decision will rely on engineering judgment and experience, particularly when the freeboard requirements vary significantly from one reach to the next.

4.7 Examples

4.7.1 Analysis of Resistance to Flow in Sand-Bed Channels

For the 2-year flood (425 cfs), a channel is observed functioning essentially as a plane bed without sediment movement. A bed-material sample is laboratory-analyzed and provides the following information:

D_{90} = 0.80 mm

D_{50} = 0.35 mm

D_{10} = 0.15 mm

Channel geometry and flow characteristics available from gaging station measurements near the peak discharge of the 2-year event yield the following:

(flow area) $A = 210\ ft^2$

(top width) $T = 178$ ft

(hydraulic radius) $R = 1.2$ ft

(bed slope) $S = 0.0005$

(channel velocity, Q/A) $V = 2.0$ fps

Similarly, during a 100-year event (13,000 cfs):

$A = 1,275\ ft^2$

$T = 350$ ft

$R = 3.6$

$V = 10.2$ fps

What is the resistance to flow during each flood?

a. For the 2-year event the channel can be analyzed by rigid-boundary equations assuming insignificant sediment transport and hence bed-form movement.

The Darcy f is computed from Equation 4.5. First, evaluate the Froude number:

$$Fr = \frac{V}{\sqrt{gY_h}}$$

where Y_h is the hydraulic depth $(\frac{A}{T})$

Therefore,

$$Fr = \frac{2.0}{\sqrt{32.2\ (210/178)}} = 0.32$$

Second, assuming $S_E = S_o$, then

$$f = \frac{8\ (0.0005)}{(0.32)^2} = 0.039$$

The Chezy C is then computed from Equation 4.8.

$$C = \sqrt{\frac{8\ (32.2)}{0.039}} = 81$$

Manning's n is computed from Equation 4.6.

$$n = \frac{1.49}{81}\ (1.2)^{1/6} = 0.019$$

For comparison, use Equation 4.9 for Manning's n

$$n = \frac{0.0008^{1/6}}{26} = 0.012$$

The difference of these two values reflects the difference between an analytically calculated n (0.019) using various theoretical and empirical formulas that do not directly account for bed-material characteristics, and that value based on a purely empirical calculation (0.012) that incorporates primarily bed-material characteristics. If the assumption of uniform flow with insignificant sediment transport is valid, the analytically determined n is a better estimate, since it represents a calibration of n based on measured flow data. Furthermore, as a calibrated value, this estimate implicitly accounts for both bed-material and rigid boundary characteristics.

b. For the 100-year event the evaluation must be made under the assumption of moveable bed conditions. First, the bed form condition must be established. From Equation 4.7a, assuming $S_e = S_o$ only for purposes of bedform classification, the stream power is

$$\tau_o = \gamma RS = 62.4(3.6)(0.0005) = 0.11 \frac{\text{lb}}{\text{ft}^2}$$

$$\tau_o V = 0.11(10.2) = 1.1 \frac{\text{lb}}{\text{ft}\cdot\text{s}}$$

From Figure 4.2 with $\tau_o V = 1.1$ and $D_{50} = 0.35$ the flow condition is upper regime with antidune bed forms. From Table 4.2 the range of Manning's n is 0.015 to 0.031, with a value of 0.025 recommended for sediment transport.

For comparison, apply the rigid-boundary formulas.

From Equation 4.5 with

$$Fr = \frac{10.2}{\sqrt{32.2\ (1{,}275/350)}} = 0.94$$

$$f = \frac{8\ (0.0005)}{(0.94)^2} = 0.005$$

From Equation 4.8

$$C = \sqrt{\frac{8(32.2)}{0.005}} = 227$$

From Equation 4.6

$$n = \frac{1.49}{227} (3.6)^{1/6} = 0.008$$

Unlike the above example, the analytically determined result (0.008) from rigid-boundary equations does not represent an accurate calibration because it does not properly account for the form roughness effects from the antidune bedforms. Specifically, energy dissipation in the separation zones downstream of the bedforms further complicates the nonuniform flow conditions (i.e., $S_e \neq S_o$). Additionally, the measured depth and area used in the rigid-boundary formulas may not adequately represent the actual contributing depth and area due to the ineffective flow area in the separation zones. Therefore, with movable boundary conditions the estimate of 0.025 is considered the more reliable.

4.7.2 Analysis of Flow in Rough Channels

The following example illustrates the iterative application of Equation 4.14 for evaluation of flow in large-roughness channels. The calculation is for conditions of field measured data by Virmani (1973) to allow evaluation of the accuracy of the computed result. The first step in application of Equation 4.14 is development of a relationship between channel width, W, and mean depth, d, for the given channel. Taking Virmani's site 10-0115 as an example, the data show that:

$$W = 64.05\ d^{0.1858}$$

Since $Wd = A$, $W = A/d$, and equating these two expressions for W yields:

$$\frac{A}{d} = 64.05\ d^{0.1858}$$

or

$$A = 64.05 d^{1.1858}$$

The mean velocity, $\overline{V}$, is equal to $\frac{Q}{A}$, and substituting the previous expression for A,

$$\overline{V} = \frac{Q}{64.05\ d^{1.1858}}$$

Substituting for W and $\overline{V}$ in Equation 4.14 and using Equation 4.13 to describe relative roughness area, depth is related to just discharge and the parameters of roughness geometry:

$$\frac{Q}{200.6\ d^{1.6858}\ S^{0.5}} = \left[\frac{0.001396\ Q}{b_{rg}\ d^{1.6858}}\right]^{\log(0.755/b_{rg})}$$

$$\times \left[104\ \left(\frac{d^{0.1858}}{Y_{50}}\right)^{0.492}\ b_{rg}^{1.675(d^{0.1858}/Y_{50})^{0.118}}\right]$$

$$\times \left[64.05\ d^{-0.8142}\right]^{-b_{rg}}$$

where

$$b_{rg} = \left[0.1158\ Y_{50}^{0.557}\ \frac{d^{0.8965}}{S_{50}}\right]^{0.648}\ \sigma^{-0.134}$$

Virmani's data show that:

$$D_{50} = 0.144\text{m}$$

$$\sigma = 0.313$$

$$S = 0.0117$$

Assuming that $S_{50} = 0.57 \times D_{50}$ and that the cross-stream axis Y_{50} and the long axis L_{50} are equivalent and equal to $D_{50}/0.57$, then

$$S_{50} = 0.0821\ \text{m}$$

$$Y_{50} = 0.253\ \text{m}$$

Using Equation 4.12, the calculated value of the function of effective roughness concentration, b_{rg}, is therefore $0.7268\ d^{0.6787}$. Substituting yields

$$\frac{Q}{21.7\ d^{1.6858}} = \left[\frac{0.000192\ Q}{d^{2.3645}}\right]^{\log(1.039/d^{0.6787})}$$

$$\times\ \left[204.5\ d^{0.0914}\left(0.7268\ d^{0.6787}\right)^{1.969\ d^{0.02192}}\right]$$

$$\times\ \left[\frac{64.05}{d^{0.8142}}\right]^{-0.7267\ d^{0.6787}}$$

The only two unknowns in this equation are discharge and depth, so specifying one allows the other to be calculated. Virmani's data show that at a discharge of 0.906 $m^3 s^{-1}$ the depth is 0.146 m. If, however, the depth were unknown it could have been calculated by the following iterative technique.

The known value of discharge and a guessed value of depth are substituted into the right-hand side. With depth set at, say, 1 m, the value of the right side is 4.775. Equating this with the left side of the equation, and including the known value of discharge, a calculated value of depth equal to 0.0601 m is obtained.

Using this derived value as the new guessed value of depth for the right side of the equation, the next iteration gives a depth equal to 0.11234 m. Subsequent iterations give depths of 0.1546 m, 0.1623 m and 0.1625 m. As the difference between the last two values is insignificant, the final value can be assumed to be the required value. Five iterations, therefore, seem to be sufficient for the calculation of depth, and the result is about 10 percent in error relative to the measured value.

V. SEDIMENT TRANSPORT ANALYSIS

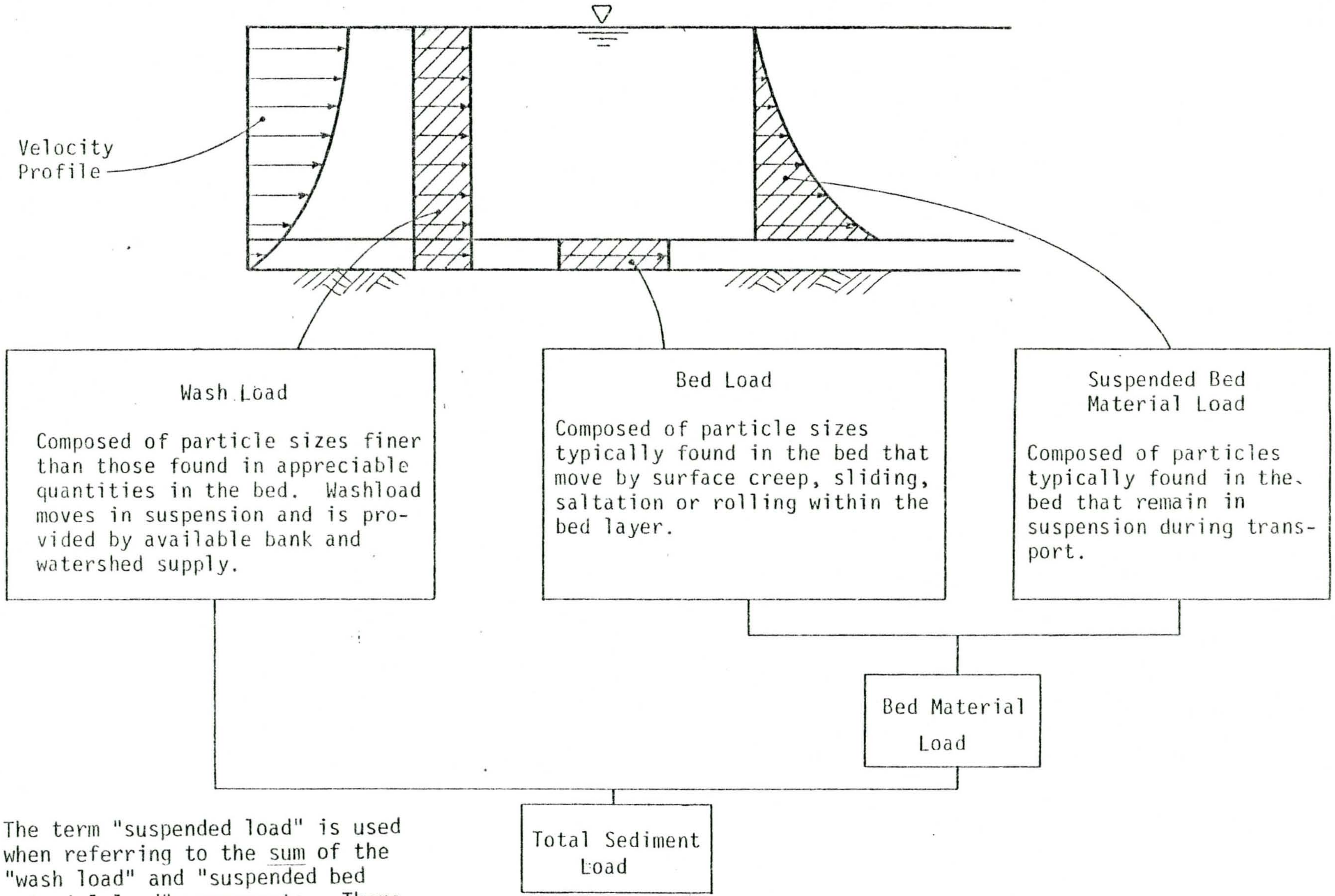
5.1 General Concepts

5.1.1 Basic Sediment Transport Theory

Sediment particles are transported by flowing water in one or more of the following ways: (1) surface creep, (2) saltation, and (3) suspension. Surface creep is the rolling or sliding of particles along the bed. Saltation (jumping) is the cycle of motion above the bed with resting periods on the bed. Suspension involves the sediment particle being supported by the water during its entire motion. Sediments transported by surface creep, sliding, rolling and saltation are referred to as bed load, and those transported by suspension are called suspended load. The suspended load consists of sands, silts, and clays. Total sediment load is defined as the sum of the bed load and suspended load. Generally, the amount of bed load transported by a large river is on the order of 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, the form of bed roughness, and other factors.

The total sediment load in a channel may also be defined as the sum of bed-material load and wash load. The bed-material load is the sum of bed load and suspended bed-material load and represents that part of the total sediment discharge which is composed of grain sizes found in the bed. The wash load is that part composed of particle sizes finer than those found in appreciable quantities in the bed (Simons and Senturk, 1977). The presence of wash load can increase bank stability, reduce seepage and increase bed-material transport. Wash load can be easily transported in large quantities by the stream, but is usually limited by availability from the watershed. The bed-material load is more difficult for the stream to move and is limited in quantity by the transport capacity of the channel. Figure 5.1 summarizes the various definitions of the components of sediment load and their contribution to total sediment load.

There is no clear size distinction between wash load and bed-material load. As a rule of thumb, engineers assume that the size of bed-material particles is equal to or larger than 0.0625 mm, which is the division point between sand and silt. The sediment load consisting of grains smaller than this is considered wash load. A more reasonable criterion, although not necessarily theoretically correct, is to choose a sediment size finer than ten per-



Note The term "suspended load" is used when referring to the sum of the "wash load" and "suspended bed material load" components. Therefore, an alternate definition of total sediment load is the sum the suspended load and bed load.

Figure 5.1. Definition of sediment load components.

cent of the bed sample as the dividing size 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 wash load is deposited on or in the stream bed. Wash load thus deposited with the coarse material is usually only a very small fraction of the total bed material.

The amount of material transported, eroded, or deposited in an alluvial channel is a function of sediment supply and channel transport capacity. Sediment supply includes the quality and quantity of sediment brought to a given reach. Transport capacity is a function of the size of bed material, flow rate, and geometric and hydraulic properties of the channel. Generally, the single most important factor determining sediment transport capacity is flow velocity. Additionally, since transport capacity is generally proportional to the third to fifth power of velocity, small changes in velocity can cause large changes in sediment transport capacity. Either supply rate or transport capacity may limit the actual sediment transport rate in a given reach.

5.1.2 Basic Terminology

A variety of terminology has been used to describe channel response to changing sediment transport conditions. In a very general sense, erosion and sedimentation are used in a generic fashion to describe any loss or gain of sediment. Other terminology is then used to more precisely define the erosion and sedimentation occurring under specific circumstances. For example, vertical channel response is often described by words such as aggradation, degradation, general scour and local scour, while horizontal response is typically referred to as lateral migration. The terminology describing vertical channel response has become somewhat confusing as different authors and/or publications have used the words in slightly different ways. To facilitate future discussions and to avoid confusion, the following definitions are adopted in this manual.

Aggradation and degradation are the raising or lowering of the channel bed, respectively, occurring over relatively long reaches and long time periods from changes in such things as sediment supply, controls, river geomorphology, and man-induced effects. General scour refers to a more localized vertical lowering of the channel bed over relatively short time periods, for example, the general scour in a given reach after passage of a single flood.

Special cases of general scour include contraction scour occurring in the vicinity of bridges that encroach on the flood plain and the scour that occurs downstream of a gravel pit. Unlike degradation, which has the antonym "aggradation," an accepted antonym for general scour is more difficult to define. In this manual "deposition" will be used as the counterpart to general scour. Local scour is caused by vortices resulting from local disturbances in the flow such as bridge piers and embankments. In general, the vertical changes in a channel are additive so that, for example, local scour could be occurring in a reach experiencing general scour and/or aggradation.

Lateral migration is defined as bankline shifting due to processes of bank erosion. Since aggradation/degradation, general scour/deposition, and/or any local scour along an embankment can promote bank instability, the vertical and horizontal shifting on a channel are interrelated. Degradation, general scour, local scour and lateral migration can endanger adjacent property, bridges and other hydraulic structures, while aggradation and deposition can reduce channel capacity, increase lateral erosion and increase flooding potential.

5.2 Level I Analysis

5.2.1 Plan Form Characteristics

Discussion - Rivers can be classified broadly in terms of channel pattern, that is, the configuration of the river as viewed on a map or from the air. The patterns are straight, meandering, braided, or some combination of these (Figure 5.2).

A straight channel can be defined as one that does not follow a sinuous course. Leopold and Wolman (1957) have pointed out that truly straight channels are rare in nature. Although a stream may have relatively straight banks, the thalweg, or path of greatest depth along the channel, is usually sinuous (Figure 5.2b). As a result, there is no simple distinction between straight and meandering channels.

The sinuosity of a channel, defined as the ratio between the thalweg length and the down-valley distance, is most often used to distinguish between straight and meandering channels. Sinuosity varies from a value of unity to a value of three or more. Leopold, Wolman and Miller (1964) took a sinuosity of 1.5 as the division between meandering and straight channels. It should be

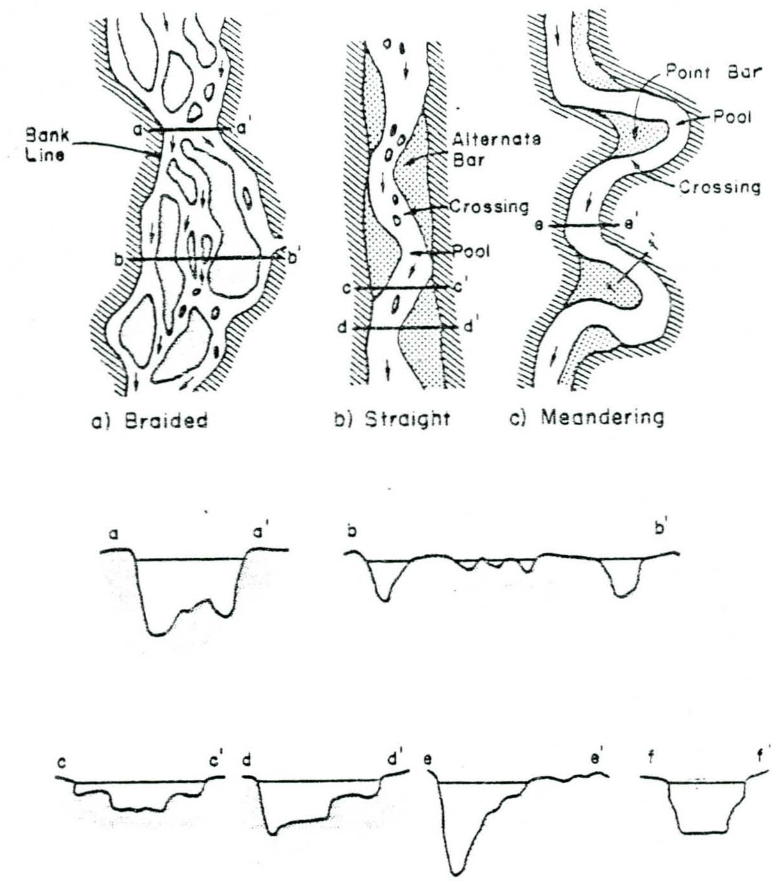


Figure 5.2. River channel patterns.

noted that in a straight reach with a sinuous thalweg developed between alternate bars (Figure 5.2b), a sequence of shallow crossings and deep pools is established along the channel.

A braided stream or river is generally wide with poorly defined and unstable banks, and is characterized by a steep, shallow course with multiple channel divisions around alluvial islands (Figure 5.2a). Braiding was studied by Leopold and Wolman (1957) in a laboratory flume. They concluded that braiding is one of many patterns which can maintain quasi-equilibrium among the variables of discharge, sediment load, and transporting ability. Lane (1957) concluded that, generally, the two primary causes that may be responsible for the braided condition are (1) overloading, that is, the stream may be supplied with more sediment than it can carry, resulting in deposition of part of the load; and (2) steep slopes, which produce a wide, shallow channel where bars and islands form readily.

A meandering channel is one that consists of alternating bends, giving an S-shape appearance to the plan view of the river (Figure 5.2c). More precisely, Lane (1957) concluded that a meandering stream is one whose channel alignment consists principally of pronounced bends, the shapes of which have not been determined predominantly by the varying nature of the terrain through which the channel passes. The meandering river consists of a series of deep pools in the bends and shallow crossings in the short straight reach connecting the bends. The thalweg flows from a pool through a crossing to the next pool forming the typical S curve of a single meander loop.

Application - Knowledge of the various channel types and their characteristics provides the engineer or designer with a basic understanding of channel behavior. Alluvial channels of all types deviate from a straight alignment. The thalweg oscillates transversely and initiates the formation of bends. In general, the engineer concerned with channel stabilization should not attempt to develop straight channels. In a straight channel the alternate bars and the thalweg (the line of greatest depth along the channel) are continually changing, thus the current is not uniformly distributed through the cross section but is deflected toward one bank and then the other. When the current is directed toward a bank, the bank is eroded in the area of impingement and the current is deflected and impinges 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 extent of erosion.

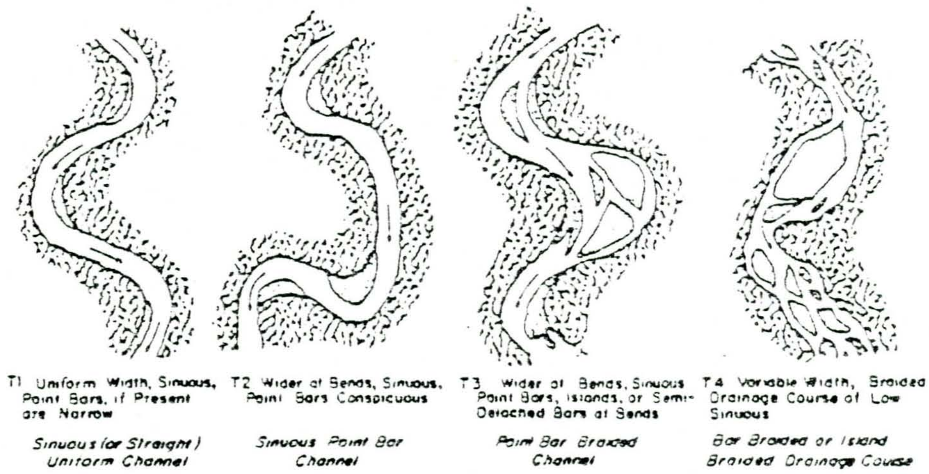
In general, bends are formed by the process of erosion and deposition. Erosion without deposition to assist in bend formation would result only in escalloped banks. Under these conditions the channel would simply widen until it was so large that the erosion would terminate. The material eroded from the bank is normally deposited over a period of time on the point bars that are formed downstream. The point bars constrict the bend 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 point bars formed in the bendways clearly define the direction of flow. 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.

Because of the physical characteristics of straight, braided, and meandering streams, all natural channel patterns intergrade. Although braiding and meandering patterns are strikingly different, they actually represent extremes in a continuum of channel patterns. On the assumption that the pattern of a stream is determined by the interaction of numerous variables whose range in nature is continuous, one should not be surprised at the existence of a complete range of channel patterns. A given channel, then, may exhibit both braiding and meandering, and alteration of the controlling parameters in a reach can change the character of a given stream from meandering to braided or vice versa.

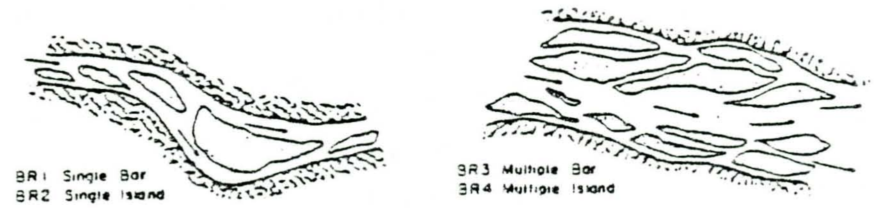
Figure 5.3 summarizes the subclassifications of river channels within the major types of meandering, straight and braided channels that are of use to the geomorphologist and engineer. Information in this figure provides guidelines for qualification of channel characteristics for practical applications.

Example - From field observations and review of recent aerial photographs, the following characteristics have been determined:

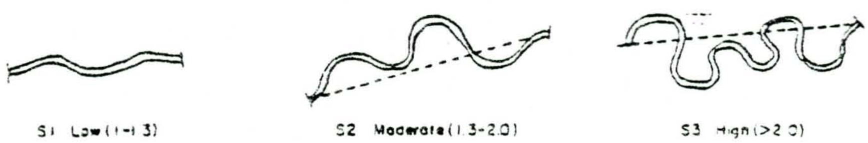
- sinuosity = 1.2
- wide, braided channel



(a) Variability of unvegetated channel width: channel pattern at normal discharge



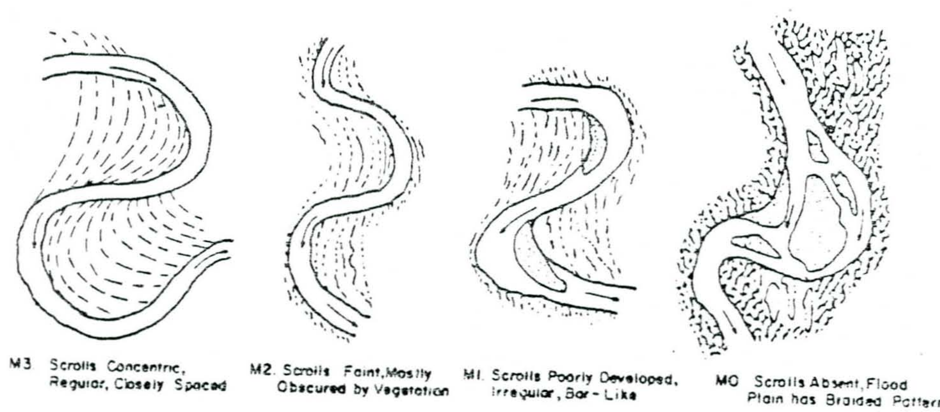
(b) Braiding patterns



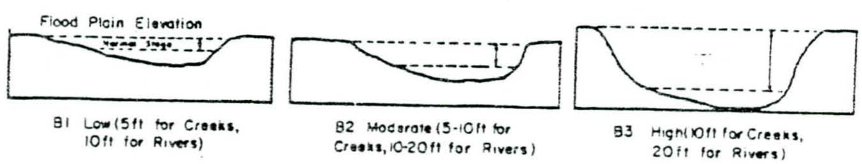
(c) Types of sinuosities

Figure 5.3. Classification of river channels (after Culbertson et al., 1967)

(d) Oxbow lakes on floodplain

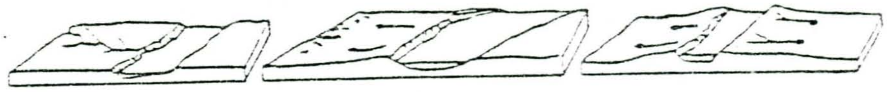


(e) Types of meander scroll formations



(f) Types of bank heights

Figure 5.3. (continued)

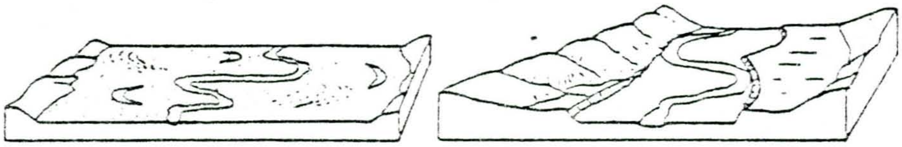


L0 No Levees

L1 Levees Mainly on Concave Bank

L2 Levees Well Developed on Both Banks

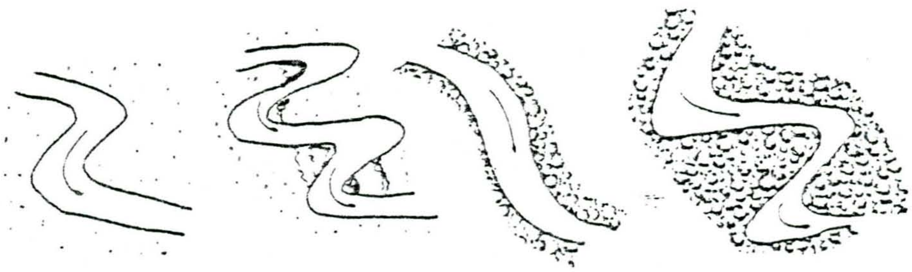
(g) Types of natural levee formations



F2 Broad in Relation to Channel Width

F1 Narrow, Confined by Terraces or Valley Sides

(h) Types of modern floodplains



V0 Very Limited or No Vegetation Along Banks

V1 Vegetal Growth on Inside of Bends of the Channel

V2 Narrow Belt of Dense Vegetation Along Both Bends

V3 Dense Vegetation Growth in the River Basin

(i) Types of vegetal patterns

Figure 5.3. (continued)

- low-flow bank height, about 2 to 3 feet
- evidence of meander scars in flood plain

From these observations, it can be concluded that the channel is presently, and has been historically, unstable. The low sinuosity, braided character and low banks suggest a steep, wide water course with poorly defined, unstable banks.

5.2.2 Lane Relation and Other Geomorphic Relationships

Discussion - A number of geomorphic relationships are available that can provide insight on the general characteristics of a channel and its response to various impacts or changes. The usefulness of these procedures is to provide the engineer or designer with a qualitative understanding that will guide quantitative calculations and assist in formulating conclusions.

Application - A basic physical process that occurs in a channel is its tendency, in the long run, to achieve a balance (equilibrium) between the product of water flow and channel slope and the product of sediment discharge and sediment size. The most widely known geomorphic relation embodying this equilibrium concept is known as Lane's principle. The basic relation is (Lane, 1955):

$$QS \propto Q_S D_{50} \tag{5.1}$$

where Q is the water discharge, S is the channel slope, Q_s is the sediment discharge and D_{50} is the median diameter of the bed material. Figure 5.4 illustrates the equilibrium concept as proposed by Lane.

A similiar set of relationships was given by Schumm (1977):

$$Q \simeq \frac{b_1 d_1 \lambda}{S} \tag{5.2a}$$

and

$$Q_s \simeq \frac{b_1 \lambda_1 S}{d_1 P} \tag{5.2b}$$

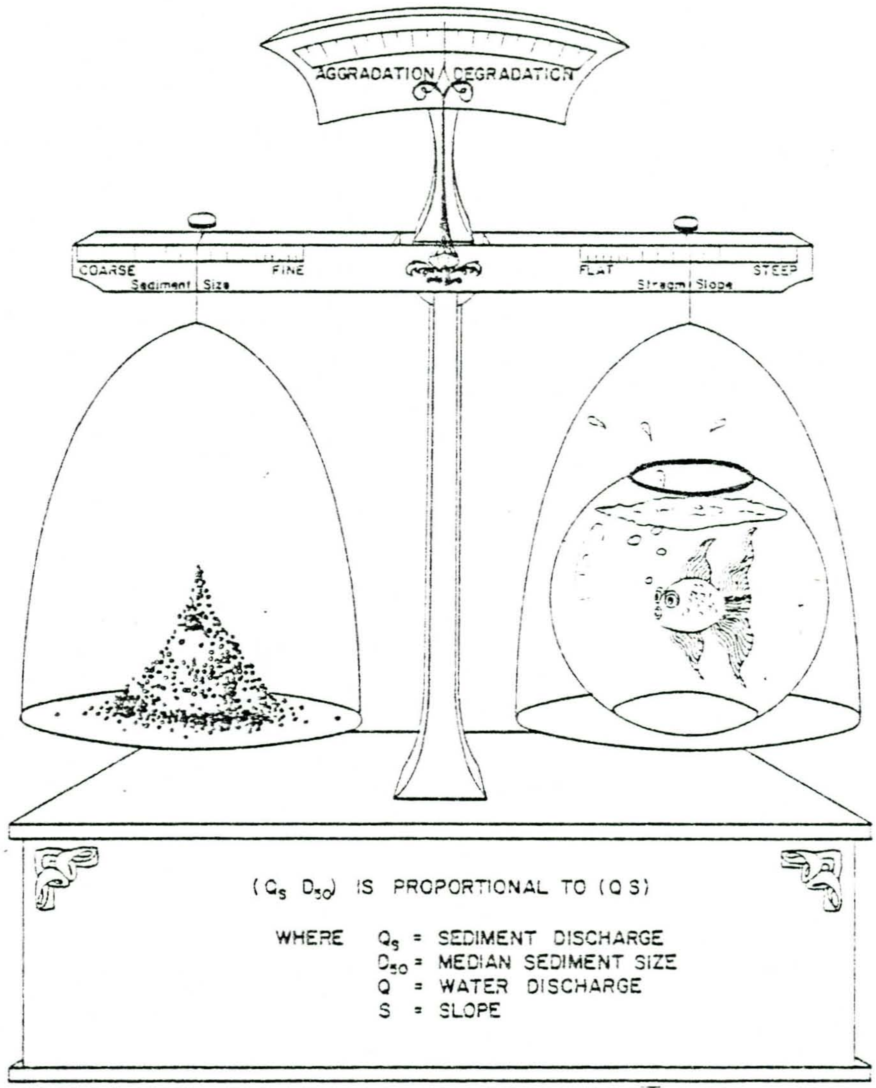


Figure 5.4. Schematic of the Lane relationship for qualitative analysis.

where b_1 is channel width, d_1 is depth, λ is meander wave length, S is channel slope and P is sinuosity. Width/depth ratio, indicated to be directly related to sediment discharge, is implicitly included in Equation 5.2b because both depth and width appear separately.

Investigations have also focused on the relationship between channel characteristics, such as slope and sinuosity, and channel patterns (straight, meandering, braided). Results of Friedkin (1945), Leopold and Wolman (1957), and Lane (1957) suggest that for a given discharge there is a threshold slope separating braided and meandering channels. Figure 5.5 summarizes the various results, which in general can be fitted by equations of the form

$$S\,Q^{\alpha} = K \tag{5.3}$$

where S is the channel slope, Q is the discharge, α is a coefficient and K is a constant. The data used to develop these relationships included both laboratory results and field measurements for predominantly sand-bed channels. Furthermore, the results were derived from perennial channels using either the mean annual discharge (dominant discharge) or the bankfull discharge for analysis. Consequently, a strict application of these relationships to the ephemeral streams typical of the Southwest is impossible; however, they can be used in a qualitative sense to develop an understanding of possible channel response.

Figure 5.6 illustrates a relationship between sinusoity, slope, and channel pattern (after Kahn, 1971). This figure also illustrates that any natural or artificial process which alters channel slope can result in modifications to the existing river pattern. Similar to the slope-discharge relations, strict application of Figure 5.6 is not feasible to ephemeral channels, since it was developed from limited laboratory results; however, application in a qualitative sense can be beneficial.

Example - A series of grade-control structures has been proposed that will reduce channel slope from 0.1 percent to 0.065 percent for an arroyo with a bankfull discharge of 2,500 cfs.

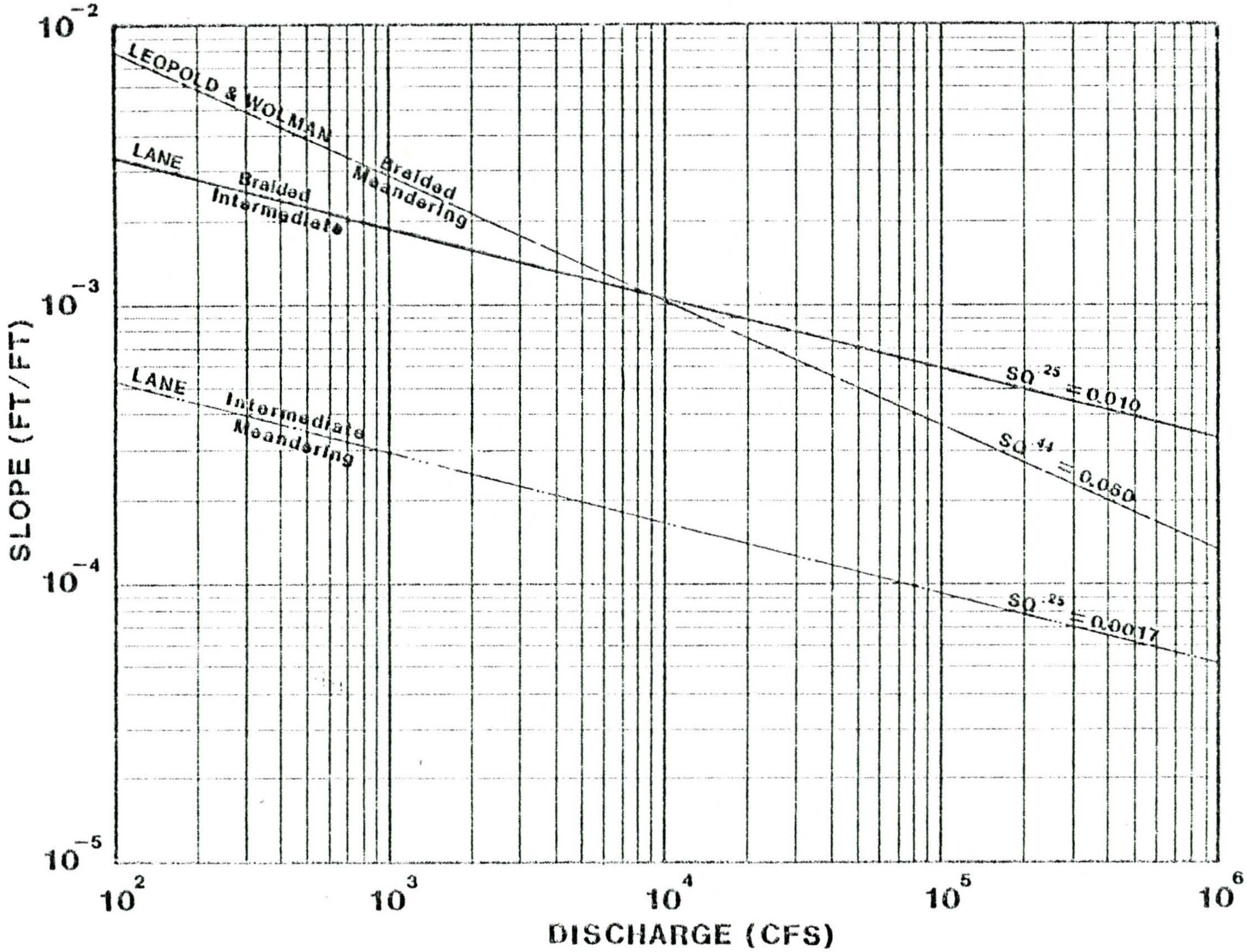


Figure 5.5. Slope-discharge relations.

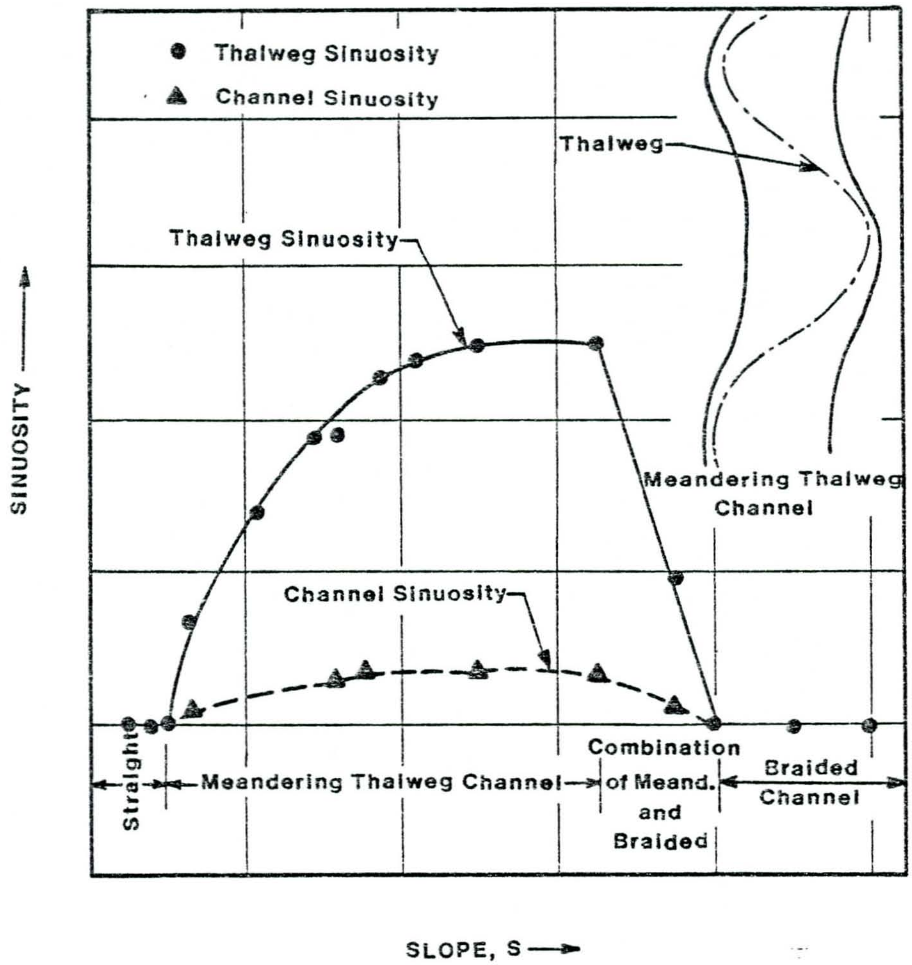


Figure 5.6. Channel pattern versus slope and simuosity (Kahn, 1971)

Assuming water discharge and D_{50} sediment size remain constant, the Lane relation (Equation 5.1) indicates that the sediment discharge must decrease. That is,

$$Q^{\circ}S^{-} \propto Q_s^{-} \; D_{50}{}^{\circ}$$

(Note that if we had more than one dependent variable, for example, if the D_{50} size was not assumed constant, it might not be possible to predict the direction of change in Q_s.) Application of the Schumm equilibrium equations (Equations 5.2a and 5.2b) provides a similar result.

According to the slope-discharge relation (Figure 5.5), a decrease in slope will produce a change in the direction towards a meandering channel. Using the bankfull discharge of 2,500 cfs suggests that the grade-control structures will not significantly change the channel pattern from an intermediate or mildly meandering characteristic; however, since we are applying ephemeral channel data to a relationship derived for perennial channels, it is impossible to be conclusive.

The Kahn relationship (Figure 5.6) suggests that even a small decrease in slope from an intermediate or mildly meandering channel will promote significant thalweg sinuosity. As with the slope-discharge relation, it is not possible to be conclusive; however, the application of these relationships together promotes the idea of a transition to a more stable, meandering channel after installation of grade-control structures.

5.2.3 Aerial Photograph Interpretation

Discussion - Maps and aerial photographs supplement each other and provide more information when used together than either does alone. For example, a topographic map provides quantitative information on land surface characteristics; however, due to the time since it was compiled, parts of the map may be obsolete. A recent aerial photograph will show changes that have occurred since the map was compiled and allows accurate assessment of present conditions.

There are two major types of aerial photography: vertical and oblique. A vertical photograph is taken with the optical axis of the camera held essentially vertically. Vertical photographs are used in most planimetric and topographic mapping, construction of mosaics, and orthophoto production.

Oblique photography is accomplished by purposely tilting the optical axis a sizeable angle from the vertical. A high oblique is a photograph taken with the camera inclined so that the apparent horizon appears. A low oblique is taken with the camera axis tilted but not to the degree that the horizon appears. Due to the greater ground coverage of obliques, high obliques are often used in the preparation of small-scale planimetric maps and charts. When taken as convergent photography, low obliques can be utilized in the compilation of accurate topographic maps.

Application - Aerial photographs provide information valuable to the qualitative and quantitative analysis of river hydraulics and channel geometry problems. Utilization of aerial photographs over a span of many years will provide a time-sequenced documentation of historical trends and changes in the river. Assessments made from aerial photographs are dependent largely on the quality and scale of the photos. Properly applied, photographic interpretation can provide an abundance of accurate and useful information.

Evidence of land-use changes, bank cutting, shifting of the thalweg, meander tendencies, lateral migration, vegetation changes, and sediment deposition can be documented by studying photographs for different years. If time-sequenced aerial photography is available for an area, it is a relatively simple procedure to trace or freehand a composite sketch showing morphologic evolution, or to document changes in channel width, sinuosity, etc. through direct measurements.

It should be noted that an aerial photograph is a perspective projection of the ground surface onto the focal plane of the camera. Thus, points in a plane closer to the camera at the time of exposure will have larger images than points located farther from the camera. For this reason, the scale can vary in different portions of a vertical photograph depending on topographic relief. Generally, the scale given for a set of aerial photographs is the average scale based on the difference between the average ground-surface elevation and flying height for all photographs taken during the flight. An average scale can be applied to a scaled distance to give a reasonable estimate of corresponding ground length so long as relief is not extremely variable. In areas of highly variable relief, scaling errors will result from use of an average scale and limit the accuracy and reliability of any quantitative measurements.

Time can be a limiting factor in acquisition of aerial photographs. Orders often require four to six weeks to be processed. Time delays are often increased by the fact that many agencies hesitate to select photos for you because of their uncertainty as to what is wanted. Therefore, unless you have access to indexes retained at the agencies, allow another four to six weeks to obtain copies of indexes from which you will designate preferred photos.

Photos are indexed by geographic coordinates, but are further referenced by codes representing the various flights making up the index mosaics. As flight paths tend to be straight, while rivers tend to meander, the necessity for careful identification of desired photos becomes more understandable.

Aerial photos come in a standard 9" x 9" size, usually costing $5.00 to $6.00 each. Often, however, these may be enlarged two, three, or even four times (two-times enlargements--18" x 18"--run $25.00 to $30.00). Note: flight elevations do vary, and thus scales will also vary. At a scale of 1:24,000, one inch on the photograph depicts 2,000 feet. This 1:24,000 scale photo then covers approximately 12 square miles. A 1:63,360 scale photo covers about 81 square miles. Be prepared to compensate accordingly.

The U.S. Geological Survey National Cartographic Information Center (NCIC) provides assistance in locating and acquiring maps, aerial photographs, satellite images, and other cartographic products. NCIC offers direct access to most of the nation's domestic aerial photographs (including some historical material) and satellite images available to the public. Important other sources also exist and NCIC will assist you in contacting them when appropriate. These sources include federal agencies and some private firms that retain the originals of photographs or that produce highly specialized products.

NCIC works in conjunction with the Earth Resources Observation Systems (EROS) Data Center in Sioux Falls, South Dakota. Both NCIC and the EROS Data Center research requests for information about photos and take orders for aerial and space photographs and space images. For photographs prior to 1941, the National Archives must be contacted. Addresses for these agencies are provided in Table 5.1.

Example - For an erosion-sedimentation analysis of Arroyo de las Calabacillas in New Mexico, three sets of aerial photographs covering a time period of 45 years were obtained. A 1935 soil conservation photograph was

Table 5.1. Agencies with Information on Aerial Photographs.

EROS Data Center U.S. Geological Survey EROS Data Center User Services Section Sioux Falls, South Dakota 57198 Telephone: 605/594-6151	National Cartographic Information Center U.S. Geological Survey National Space Technology Laboratories NSTL Station, Mississippi 39529 Telephone: 601/688-3544
NCIC Headquarters National Cartographic Information Center U.S. Geological Survey 507 National Center Reston, Virginia 22092 Telephone: 703/860-6045	Rocky Mountain Mapping Center--NCIC U.S. Geological Survey Box 25046, Stop 504 Federal Center Denver, Colorado 80225 Telephone: 303/234-2326
NCIC Offices Eastern Mapping Center--NCIC U.S. Geological Survey 536 National Center Reston, Virginia 22092 Telephone: 703/860-6336	Western Mapping Center--NCIC U.S. Geological Survey 345 Middlefield Road Menlo Park, California 94025 Telephone: 415/323-8111, ext. 2427
Mid-Continent Mapping Center--NCIC U.S. Geological Survey 1400 Independence Road Rolla, Missouri 65401 Telephone: 314/341-0851	National Archives Cartographic Division Attn: Richard Spurr 841 South Pickett Street Alexandria, Virginia 22304 Telephone: 703/756-6704

obtained from the National Archives with a four-times enlargement of its original 1:35,000 scale. A 3' x 3' mosaic was obtained based on 1967 photography available from NASA. The original photographs had a scale of 1:26,000 and those selected for the mosaic were enlarged four times. A 1980 set of 9" x 9" low-altitude photographs (scale 1:10,800) were obtained from a local aerial surveying firm. Part of the analysis of these photographs consisted of preparation of composite sketches illustrating plan form changes over the 45-year period (Figure 5.7). As can be seen from this figure, aerial photography indicates a history of meander development and cutoff in the area of the S bend and increased sinuosity at the horseshoe bend. From 1935 to 1967 several channel shifts occurred; however, from 1967 to 1980 the channel was unchanged. When combined with available information on historical flood occurrences or land-use changes, such qualitative aerial photograph interpretation can provide much valuable information on system response and evolution.

5.2.4 Bed- and Bank-Material Analysis

Discussion - Knowledge of the characteristics of bed and bank material is important to any fluvial systems analysis. Bed and bank material analysis in a qualitative Level I evaluation primarily involves visual observations made during site reconnaissance as well as evaluation of existing data pertaining to soils and geology of the study area. Soils and geologic information are interrelated to the extent that surficial geology influences soil type and development. Additionally, rock outcrops may comprise the channel bed and/or banks in certain reaches, limiting the extent to which degradation or lateral migration can progress. Thus, accurate delineation of geologic control is an integral part of a qualitative assessment of bed and bank materials in a fluvial system.

Application - Visual inspection of bed and bank materials can serve to identify physical conditions or features of significance in a system. For example, the relative cohesiveness of bank materials and their ability to resist erosion by water can readily be assessed by observing the height and steepness of the channel banks. During site reconnaissance observable bank failure areas should be noted. For example, block failure from development of tension cracks can be a significant point source of sediment in a given reach. Although block failures are most common to stratified banks, similar localized

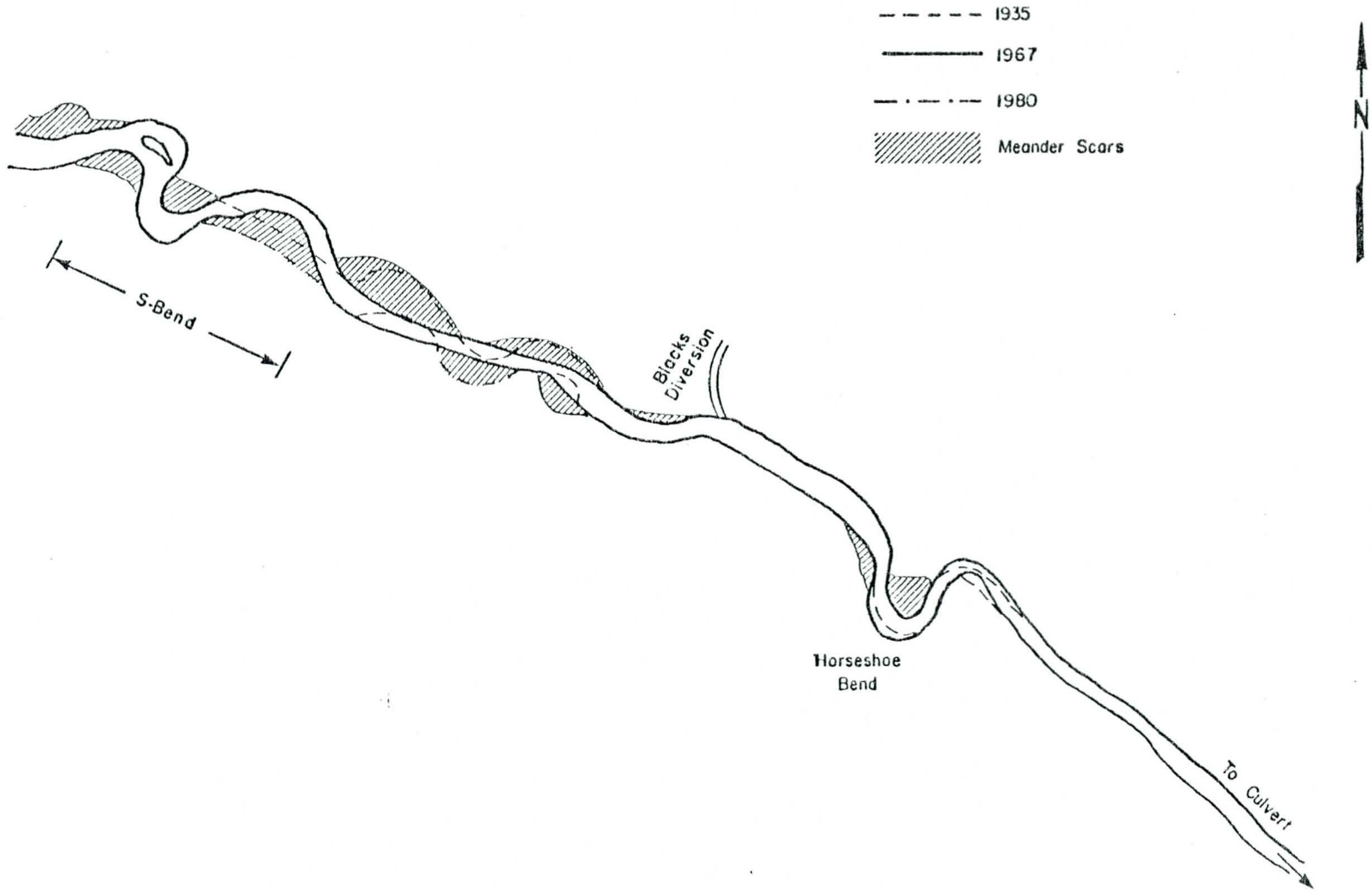


Figure 5.7. Example illustrating qualitative information derived from time sequence analysis of aerial photographs.

mass wasting phenomena occur in noncohesive and cohesive banks from the processes of sloughing and sliding, respectively. In addition to visual observation of bank material and conditions, observation of bed material and bar deposits can tell the observer much about the type of sediments being transported in the system.

Visual techniques can also be employed to assess the textural composition and predominant material sizes (i.e., sand, clay, silt) in the bed and banks. Incised banks should be investigated to determine the level of stratification, presence of clay lenses, and layer thicknesses.

In addition to field observations, information in the literature may be useful in a qualitative assessment of bed and bank material. Possible sources include Soil Conservation Service (SCS) soil survey reports and land-use surveys, and environmental statements.

Example - During a preliminary site visit for an erosion/sedimentation analysis of a sand-bed channel, a 20- to 30-foot high bluff was observed protruding into the channel. Closer inspection found it to be round-stone conglomerate, a relatively stable sedimentary rock outcrop. Results of HEC-2 water-surface profile computation during the quantitative analysis indicated this outcrop was a significant control point influencing channel hydraulics. As a result of field observations, it was known to be a stable formation that would continue to be a significant control, not one expected to erode away quickly.

5.2.5 Land-Use Changes

Discussion - Water and sediment yield from a watershed is a function of land-use practices. Thus, knowledge of the land use and historical changes in land use is essential to understanding the water and sediment sources in a watershed. Relative percentages of forest, agricultural and urban land can provide insight to the quantity and type of water and sediment load produced in a watershed.

The presence or absence of vegetative growth can have a significant influence on the runoff and erosional response of a fluvial system. The root structure of plants, bushes and trees helps to develop and maintain a stable soil structure and serves as an erosion-retarding force. Large-scale changes in vegetation resulting from fire, logging practices, land conversion and urbanization can either increase or decrease the total water and sediment yield from a watershed. For example, fire and logging practices tend to increase water sediment yield, while urbanization promotes increased water yield and decreased sediment yield. In addition to greater runoff volumes, urbanization causes peak flows to occur sooner. Potential damages from floods also increase as the property value subject to damage increases.

Application - Information on land-use history and trends can be found in Federal, state and local government documents and reports (i.e., census information, zoning maps, future development plans, etc.). Additionally, analysis of historical aerial photographs can provide significant insight on land-use changes. For example, the changes in vegetative cover over a given time can be classified into groups, such as "no change," "vegetation increasing," "vegetation damaged," and "vegetation destroyed." Estimates can also be made of bank stability and riparian conditions from aerial photographs.

Example - An analysis of land-use changes along the Salt River was conducted during a hydraulic analysis of the Seventh Street bridge in Phoenix. The main changes that have occurred since 1960 have been induced by man. Photographs of the river in 1960 show a wide braided channel with scattered vegetation. The braided portion of the channel extends laterally nearly 3,000 feet at some points. Since this time, gravel mining activities, construction of roads and bridges, and development along the river have eliminated the vegetation and in many places channelized and contained the river so that it is no longer braided. This development has caused an increase in flow velocities accompanied by an increase in sediment transport rate and potential degradation in the channel bed. The effects of the increased sediment transport rate and degradation have been curtailed by the river's ability to form an armor layer of large cobbles and boulders. This layer exists through most of the study reach. When the armor layer is ruptured, the sediment transport will increase, degrading the channel until enough large material accumulates on the surface of the channel bed to re-form an effective armor layer. In recent years the increase in construction and gravel mining has disturbed the armor layer, and the bed profile of the channel has changed due to degradation, mining, and the reworking of the channel bed.

5.2.6 Flood History and Rainfall-Runoff Relations

Discussion - Consideration of flood history is an integral step in attempting to characterize watershed system response and morphologic evolution. Analysis of flood history is of particular importance to an understanding of dryland stream characteristics. Many dryland streams flow only during the spring and immediately after major storms. For example, Leopold, et al. (1966) found that arroyos near Santa Fe, New Mexico, flow only about three times a year. As a consequence, dryland stream response can be considered to be more hydrologically dependent than streams located in a humid environment. Whereas the simple passage of time may be sufficient to cause change in a stream located in a humid environment, time alone, at least in the short term, may not necessarily cause change in a dryland system due to the infrequency of hydrologically significant events. Thus, the absence of significant morphological changes in a dryland stream or river, even over a period of years, should not necessarily be construed as indicative of system stability.

Although the occurrence of single large storms can often be directly related to system change, this is not always the case. In particular, the succession of morphologic change in arid to semiarid regions may be linked to the concept of geomorphic thresholds as proposed by Schumm (1977). Under this concept, although a single major storm may trigger an erosional event in a system, the occurrence of such an event may be the result of a cumulative process leading to an incipiently unstable geomorphic condition.

Application - Where available, the study of flood records and corresponding system responses, as indicated by time-sequenced aerial photography or other physical information, may help the investigator determine the relationship between morphological change and flood magnitude and frequency. Evaluation of wet-dry cycles can also be beneficial to an understanding of historical system response. Observable historical change may be found to be better correlated with the occurrence of a sequence of events during a period of above-average rainfall and runoff than with the single large event. The study of historical wet-dry trends may explain certain aspects of system response. For example, a large storm preceded by a period of above-average precipitation may result in less erosion (due to better vegetative stabilization of the channel banks and watershed) than a comparable storm occurring under dry antecedent conditions; however, runoff volumes might be greater due to saturated soil conditions. A good method to evaluate wet-dry cycles is to plot annual rainfall amounts, runoff volumes and maximum annual mean daily discharge for the period of record. A comparison of these graphs will provide insight to wet-dry cycles and flood occurrences. Additionally, a plot of the ratio of rainfall to runoff is a good indicator of watershed characteristics and historical changes in watershed condition.

Example - Analysis of the rainfall and runoff records for the Santa Margarita watershed in southern California has been completed. Figure 5.8 provides the precipitation record since 1877, and Figure 5.9 the maximum annual mean daily discharge. From Figure 5.9 it is apparent that 1938, 1943, 1969, 1978 and 1980 were years of significant flooding. Additionally, analysis of historical documents indicates that 1884 was also a significant flood year. From Figure 5.8, total precipitation in the 1884 flood year was second only to that of 1978. Both of these years were preceded by very dry years. In comparison, the flooding of 1916 resulted from significantly less rainfall, but was preceded by a wet year in 1915. Other years with rainfall totals similar to 1916 but preceded by dry years did not produce floods of record. The 1938 flooding occurred after a significantly wetter year in 1937. It can be concluded that antecedent soil moisture is a significant factor in the extent of flooding resulting from a given precipitation event in the Santa Margarita watershed.

The runoff-rainfall ratio for the period 1924 to 1982 is plotted in Figure 5.10. Rough estimation of average values for 10-year periods have been superimposed on the data. These estimates indicate periods of high runoff production from 1935 to 1945 and from 1975 to 1982, and extremely low production for the period in between (i.e., 1945 to 1975).

5.3 Level II Analysis

5.3.1 Watershed Sediment Yield

Discussion - The determination of erosion from natural and disturbed lands has great significance to water-resources planning and development. Erosion of the land surface affects not only the nature of the land itself, but also the erosion and sedimentation process in the receiving river system. Sediment eroded from the land surface can cause silting problems in reservoirs and channels, resulting in increased flood stages and damage. Conversely,

ANNUAL RAINFALL AT LAKE O'NEILL

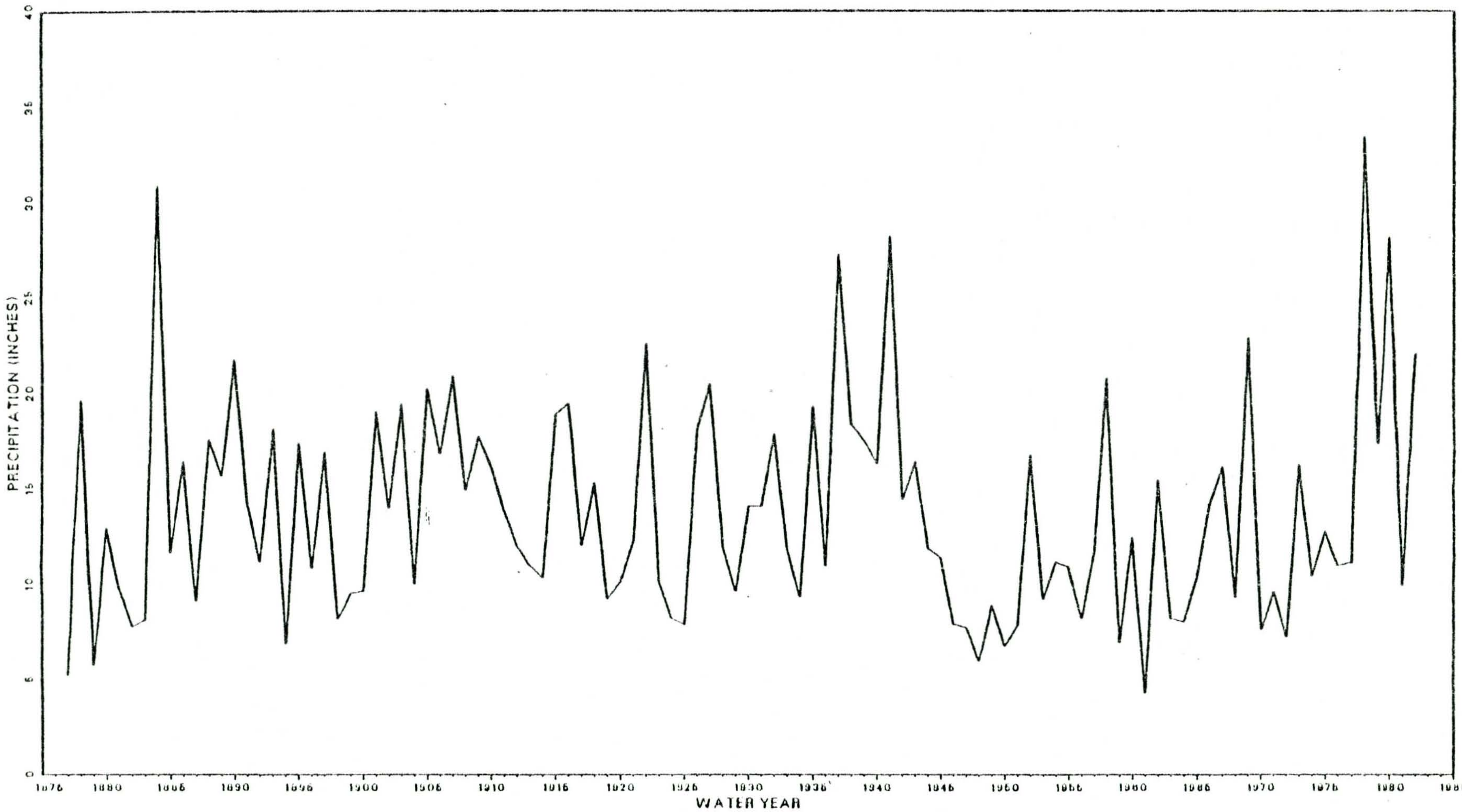


Figure 5.8. Annual rainfall for Santa Margarita Watershed.

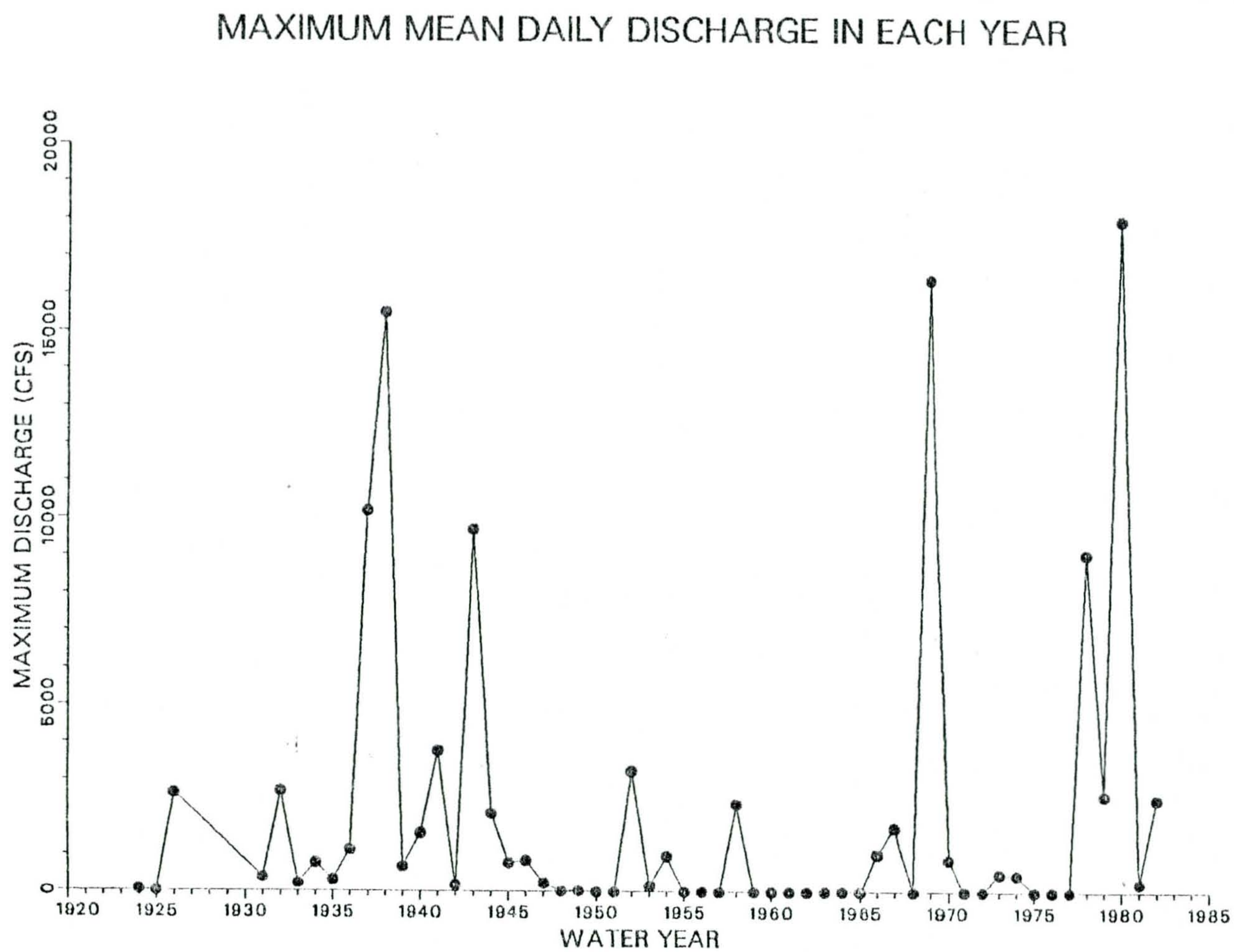


Figure 5.9. Maximum annual mean daily discharge for Santa Margarita River.

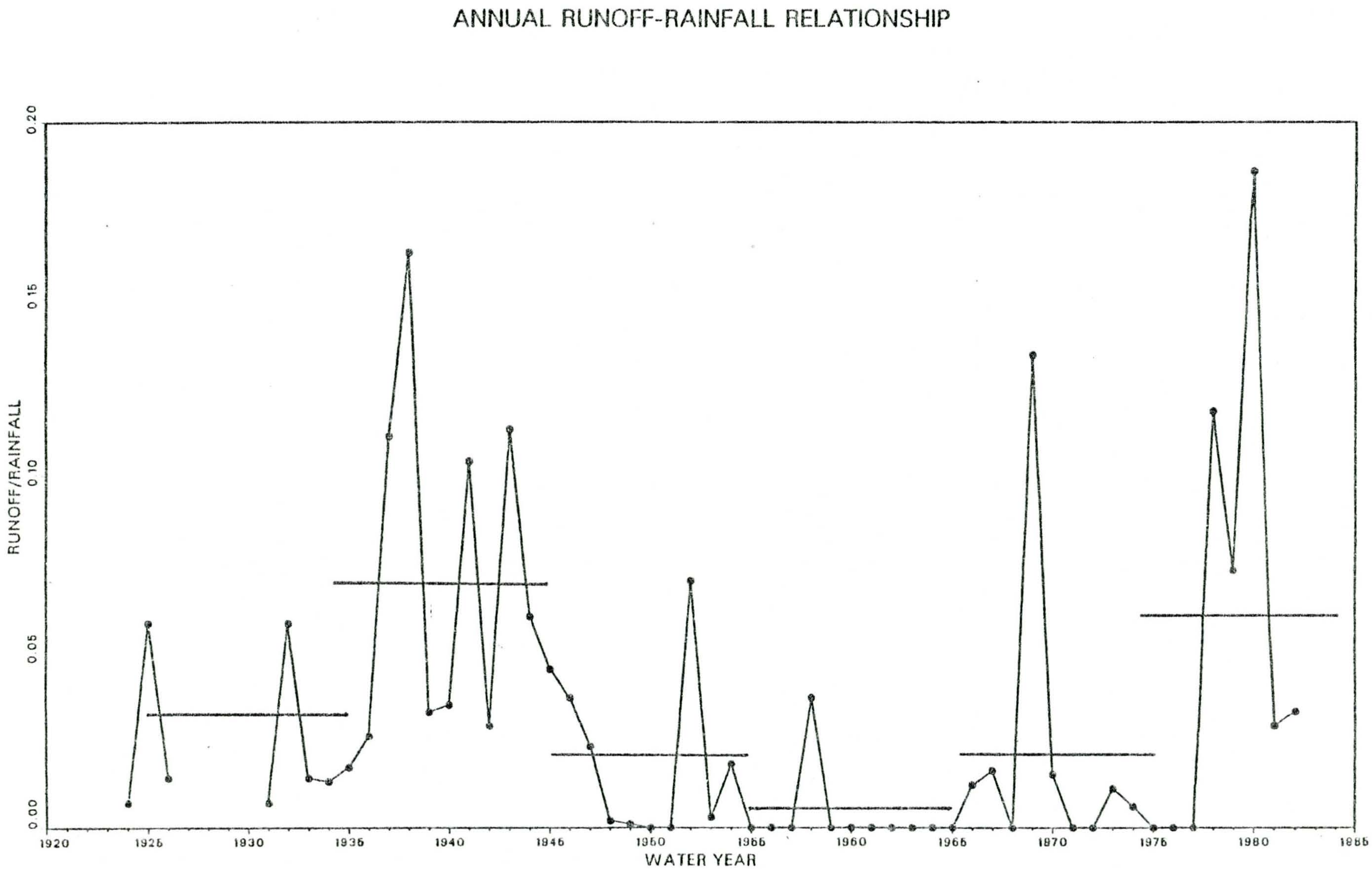


Figure 5.10. Annual runoff-rainfall ratio for Santa Margarita watershed.

reduction in erosion can also cause adverse impacts to river systems by reducing the supply of incoming sediment, thus promoting channel degradation and headcutting.

The wash load of the total sediment load in an alluvial channel is determined by the supply available in the watershed. Limited quantities of fine material moving as wash load usually will not pose direct problems for development in the riverine environment. It is usually assumed, unless there are detention structures that could effectively trap wash load, that such material does not come out of suspension and will pass through the system. A reduction in wash load can prevent the natural sealing of river banks induced by deposition of fine sediment, causing increased water loss and bank instability. Large concentrations of wash load, however, can influence the capacity of a stream to transport bed material through its influence on fluid viscosity and density, bank stability, growth of aquatic plants, and the biomass of the channel.

Formation of wash load is largely a function of raindrop detachment and transport by overland flow, which in turn, is inversely related to the level of surface cover and stabilization by vegetation. Precipitation generating erosion in dryland landscapes of the western states usually results from small storm cells that may be limited in areal extent, but can produce high-intensity and rainfall energy. This type of storm produces "flashy" runoff with a pronounced capacity for sediment removal and transportation. Thus, streams in the western states often carry large suspended sediment loads reflecting the sparsity (paucity) of vegetal cover and high transport capacity of rainfall runoff. This condition contrasts the low suspended sediment loads normally carried by streams in a humid environment due to well-developed soils and vegetative stabilization.

Application - Assessment of watershed sediment yield first requires a qualitative evaluation of sediment sources in the watershed and the types of erosion that are most prevalent. The physical processes causing erosion can be classified as sheet wash, rilling, gullying, and fluvial processes causing erosion of the stream bed and banks. Other types of erosional processes are classified under the category of mass movement, e.g., soil creep, earthflows, and landslides. Data from Soil Conservation Service (SCS) publications and maps, water-well log reports, reservoir records, climate records, and other site-specific information can be utilized along with field observations to evaluate the area of interest.

One approach providing an approximate rating of sediment yield from a watershed was developed by the Pacific Southwest Interagency Committee (PSIAC, 1968). This method designed as an aid for broad planning purposes only, consists of a numerical rating of nine factors affecting sediment production in a watershed. This rating, in turn, is correlated with ranges of annual sediment yield in acre-feet per square mile. The nine factors are surficial geology, soil, climate, runoff, topography, ground cover, land use, upland erosion, and channel erosion and transport.

A strong correlation between PSIAC estimated annual sediment yield and actual annual sediment yield has been demonstrated by Shown (1970) and Renard (1980). Both workers tested the PSIAC method against actual annual sediment yield measured in ponds and dams in the Southwest. The comparisons were done on watersheds less than about 20 square miles in area, and PSIAC results agreed with or were slightly lower than actual measurements. Appendix A briefly describes application of the PSIAC methodology.

Another approach to determine sediment yield from natural or disturbed land surfaces is based on regression equations as typified by the Universal Soil Loss Equation (USLE). The USLE, an empirical formula for predicting soil loss due to sheet and rill erosion, is probably the most widely used method for predicting soil erosion. The equation was developed from over 10,000 plot-years of runoff and soil-loss data, collected on experimental plots of agricultural land in 23 states by the U.S. Department of Agriculture. The USLE approach relates annual soil loss due to sheet and rill erosion to the product of six major factors describing rainfall energy, soil erodibility, cropping and management, supplemental erosion-control practices such as contouring or terracing, and slope steepness and length, which are usually combined to form a topographic factor. Wischmeier and Smith (1978) provide detailed descriptions of this equation and its terms.

Although widely used, the USLE approach has some important limitations, particularly in the arid regions of the West. The data base used in developing the USLE was collected east of the Rocky Mountains. Extrapolation to western areas can introduce significant error. Many arid regions of the West get a large percentage of rainfall in the form of high-intensity, short-duration thunderstorms. As this is not the case in the central and eastern

United States, the effect of this type of rainfall cannot be totally incorporated. In addition, the weathering process caused by the wind and sun on the soil between rainstorms is much more severe in arid areas. Weathering creates an additional supply of easily eroded material that can increase the erodibility factor significantly.

Williams and Berndt (1972) recognized that application of the USLE is limited to soil loss, and developed another procedure, the Modified Universal Soil Loss Equation (MUSLE), for computing sediment yields from watersheds. This method determines sediment yield based on single storms. They introduced a runoff factor instead of rainfall energy into the USLE to estimate soil loss. This make the MUSLE more applicable to the arid regions of the West, since the effect of short-duration, high-intensity events can be more adequately represented. Appendix B briefly reviews application of the MUSLE methodology.

If the sediment yield from the land surface on an annual basis rather than from a single storm is desired, the MUSLE can also be used. This application is accomplished by determining the soil loss for events of varying return periods. Recommended return period are 2, 10, 25, 50, and 100 years. The sediment yields are then weighted according to their incremental probability, resulting in a weighted storm average.

The USLE, MUSLE, and PSIAC methods are generally applicable as predictors of wash load. Total sediment load in a fluvial system is estimated as the sum of wash load (computed from the USLE, MUSLE, or another comparable method) and bed-material load (computed according to Section 5.3.6). The substitution of the MUSLE for the USLE provides a methodology that is more applicable to western conditions, especially in arid regions.

Example - Examples illustrating application of the PSIAC and MUSLE methodologies are given in Appendices A and B.

5.3.2 Detailed Analysis of Bed and Bank Material

Discussion - Bed material is the sediment mixture of which the streambed is composed. Bed material ranges in size from huge boulders many feet in diameter to fine clay particles. The erodibility or stability of a channel largely depends on the size of the particles in the bed. It is often insufficient to know only the median bed-material size (D_{50}) in determining the potential for degradation; knowledge of the bed-material size distribution is also important. Furthermore, the potential for or existence of an armor layer also needs to be addressed (see Section 5.3.7). Armoring potential differentiates a gravel- or cobble-bed stream or river from a sand-bed river. "Whereas the bed surface of a sand-bed stream typically appears to represent a random cut through the sandy bed material, gravel beds commonly consist of two separate populations, the surface layer and the underlying deposit" (Kellerhalls and Bray, 1971). As water flows over the bed of a gravel-bed stream, smaller particles that are more easily transported are carried away, while larger particles remain, armoring the surface layer of the bed. This armor layer can serve as a control unless a flow of sufficiently large magnitude occurs.

Bank material usually consists of particles of the same size as, or smaller than, bed particles. Thus, banks are often more easily eroded than the bed unless protected by vegetation, cohesion, or some type of man-made protection. River banks can be classified according to stability by vegetation, soil cohesion, amount of protection, lateral migration tendencies of the stream, etc.

Sediments are broadly classified as cohesive and noncohesive. With cohesive sediment the resistance to erosion depends on the strength of the cohesive bond binding the particles. Cohesion may far outweigh the influences of the physical characteristics of the individual particles. However, once erosion has taken place, cohesive material may become noncohesive with respect to transport.

Of the various sediment properties, size has the greatest significance to the hydraulic engineer, not only because size is important and 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 has been found to sufficiently describe the sediment particle for many practical purposes.

Size may be measured by calipers, optical methods, photographic methods, sieving, or sedimentation methods. The size of an individual particle is not of primary importance in stream mechanics or sedimentation studies, but the size distribution of the sediment that forms the bed and banks of a stream or reservoir is of great importance.

Application - The most commonly used method to determine size frequency is a volumetric sample that is laboratory-analyzed by mechanical or sieve analysis, supplemented by analysis with a hydrometer, pipette or bottom withdrawal (BW) tube when significant fine sediments are present. The VA tube technique is also utilized, particularly for samples that consist primarily of sands. Table 5.2 provides guidelines for application of the different techniques for particle size analysis. Detailed discussion of specific laboratory procedures is provided in several governmental publications (i.e., COE, 1970; USGS, 1969; ARS, 1979). In general, the results are presented as cumulative size-frequency curves. The fraction or percentage by weight of a sediment that is smaller or larger than a given size is plotted against particle size. A useful parameter describing the shape of a gradation curve is the gradation coefficient:

$$G = \frac{1}{2} \left(\frac{D_{84.1}}{D_{50}} + \frac{D_{50}}{D_{15.9}}\right) \tag{5.4}$$

where $D_{84.1}$, D_{50} and $D_{15.9}$ are based on a percent finer (by dry weight) analysis. This equation is only applicable to S-shaped, particle size-distribution curves.

The size of the bed or bank material sample required for sieve analysis will depend on the maximum particle size in the sample and the requirement that the sample be representative of the material to be tested. Within the constraints of obtaining a representative sample, bed and bank material samples should be limited in weight to facilitate handling. Corps of Engineers guidelines for obtaining a minimum weight sample for sieve analysis are presented in Table 5.3. As Table 5.3 indicates, for bed and bank materials that have maximum particle sizes in the coarse gravel to cobble range, the sample size required to ensure accurate representation becomes fairly weighty (i.e., 13 pounds for 3-inch maximum particle sizes). For a sample collection program that entails gathering numerous bed and bank material samples, the collective sample weights can become burdensome.

Another consideration pertaining to bed material sample collection on gravel-or cobble-bed streams is the potential existence of a two-layer system consisting of (1) a thin surface layer of coarser materials created by hydraulic sorting, and (2) undisturbed subsurface material. Samples containing materials from both layers would contain materials from two popula-

Table 5.2. Recommended Size Range, Analysis Concentration, and Quantity of Sediment for Commonly Used Methods of Particle Size Analysis (after ARS, 1979).

Method of Particle Size Analysis	Analysis Recommended for Particles in This Size Range	Quantity of Sediment Required for Analysis	Desirable Range in Analysis Concentration
	mm	g	Mg/l
Sieves	0.062 - 32	see Table 5.3	-----
VA tube	0.062 - 2.0	0.05 - 15.0	-----
Pipette	0.002 - 0.062	1.0 - 5.0	2,000 - 5,000
BW tube[1]	0.002 - 0.062	0.5 - 1.8	1,000 - 3,500
Hydrometer[2]	0.002 - 0.062	20 - 200	25,000 - 50,000

[1]If necessary, may be expanded to include sands up to 0.35 mm, the accuracy decreasing with increasing size--the concentration and size increased accordingly

[2]Quantity depends on size of settling container--a 1,000 ml cylinder has about the minimum diameter for most hydrometers

Table 5.3. Minimum Recommended Sample Weights for Sieve Analysis (COE, 1970).

Maximum Particle Size	Minimum Weight of Sample g	Minimum Weight of Sample lb
3-in.	6,000	13
2-in.	4,000	9
1-in.	2,000	4
1/2-in.	1,000	2
Finer than No. 4 sieve	200	0.5
Finer than No. 10 sieve	100	0.25

tions in unknown proportions. Alternatively, the thin surface layer could be removed and subsurface materials sampled by normal volumetric methods. The importance of sampling surface and/or subsurface materials in a gravel-cobble bed system is dependent largely on the objectives of the study. If study objectives focus on hydraulic friction or initiation of bed movement, then the surface layer is of interest. Conversely, for analysis of bed-material transport, sampling efforts should focus on the underlying bed materials. Quite often it may be appropriate to consider both bed layers in a sample collection program, since the disruption of an armor layer during a flood and subsequent transport of underlying bed material may be of interest.

Kellerhalls and Bray (1971) note that standard volumetric sampling methods are not appropriate for evaluating material composition of thin surface layers in river beds composed of coarse fluvial sediments. Weight limitations presented in Table 5.3 also discourage use of volumetric methods to sample coarse bed and bank material. Kellerhalls and Bray discuss the advantages and disadvantages of various methodologies for sampling coarse fluvial sediments. In addition to volulmetric sampling, other methodologies are (1) grid sampling, (2) areal sampling, and (3) transect sampling. A principal concern with use of alternative methods is the equivalence of results to standard sieve-by-weight results so that all material compositions will be referenced to a common datum. Kellerhalls and Bray present a discussion of the various bed-material sampling methodologies and the weighting factors for conversion of sampling procedures to standard sieve-by-weight methods.

A sampling and analysis procedure not considered by Kellerhalls and Bray is the area-by-area approach. Following the methodology presented by Kellerhalls and Bray, it can be shown that this approach is equivalent to standard sieve-by-weight procedures. A common way of utilizing this approach entails superposing a 2' x 2' grid subdivided into 0.1' x 0.1' squares over a randomly selected area. In this application the grid is not used to identify discrete sampling points, as in standard grid sampling procedures, but rather to provide a convenient method of determining particle surface area. A slide photograph of the grid is taken with a 35 mm camera from above (vertical to the grid). A sample identification number or location can be included in the photograph by placing a placard at one edge of the grid.

Particle size analysis of the sample defined by the grid is accomplished by projecting developed slides onto a screen and determining the area (as a percentage of total area) occupied by particles in specific size ranges. Since the grid is broken into 0.1-foot-square blocks, it is not possible to accurately differentiate particle sizes less than about 0.05 foot in diameter using this method.

Constructing a grid is relatively simple and consists of no more than some type of framework (aluminum angle, plastic pipe, etc.) with a grid pattern made of nylon twine. Grids can also be fabricated from flexible, clean plastic sheets with the grid pattern inked on; however, some grid squares may be distorted in photographs due to flexibility of the plastic. Another option, especially helpful when a grid is not immediately available, or perhaps not practical, involves taking a picture of the area of interest with a ruler placed in the center. Using this method, the photographic image can be projected onto a grid and the image size adjusted by moving the projector.

Important factors to consider in determining where and how many bed and bank material samples to collect include (1) size and complexity of the study area, (2) number, lengths and drainage areas of tributaries, (3) evidence of or potential for armoring, (4) structural features that can impact or be significantly impacted by sediment transport, (5) bank failure areas, (6) high bank areas, and (7) areas exhibiting significant sediment movement or deposition (i.e., bars in channels). For a large-scale study (i.e., five or more river miles) it is recommended as a minimum that sampling be conducted once every mile. At each sampling location a bed, bank and flood-plain sample would typically be taken. Occasional sampling at more frequent intervals may be required to characterize unique situations. It is especially important to adequately address tributary sediment characteristics, since a single major tributary and tributary source area could be the prominent supplier of sediment to a system. Samples might typically be taken 500 feet above and below the tributary on the main channel and at some location near the mouth of the tributary to completely characterize conditions.

The depth of bed material sampling depends on the homogeneity of surface and subsurface materials. When possible, it is desirable to dig down some distance to establish bed-material characteristics. If stratification of bed material is found, it is important to sample the material and note the depth at which it occurred. In homogeneous bed material, samples are typically taken near the surface, i.e., in the upper 12 inches of sand. Bank samples may be taken anywhere, if bank composition is homogeneous. For stratified banks, several samples may be required.

For purposes of Level II erosion and sedimentation analysis, it is usually desirable to define a single representative bed-material size distribution that can be utilized in evaluating sediment transport. For a more complex system, such as a system where an appreciable change in bed-material characteristics occurs, it may be necessary to use different representative gradation curves for different channel reaches. The criteria for selection of a bed-material gradation is that it adequately represents the range and distribution of bed material in the majority of the study area and should provide somewhat conservative estimates of sediment transport capacity.

Example - During a site reconnaissance 50 sediment samples were collected consisting of 16 bed samples (taken at depths of 0 to 12 inches), 12 bank samples, 6 tributary samples, 13 watershed samples and 3 flood-plain samples. Laboratory evaluation of these samples consisted of dry sieve analysis supplemented with hydrometer analysis where appreciable silt-clay percentages were encountered. Particle gradation curves were developed for the samples based on this analysis and plotted by reach.

Considering bed particle size gradation curves representative of sediment characteristics in the surface layer, a noticeable shift towards finer material occurred downstream of a small drainage entering from the right bank. A sample of alluvial fan material deposited by the small drainage documented this channel as the source of the fine material. Figure 5.11a illustrates particle size gradation curves of four samples collected upstream of the tributary, while Figure 5.11b depicts the representative gradation curve for this reach, as determined by overlaying and eye fitting (the representative curve could also be determined mathematically).

5.3.3 Profile Analysis

Discussion - Comparison of thalweg profiles over time can provide valuable insight to and understanding of aggradation/degradation patterns in a channel. This information is useful both by itself and as verification of mathematical modeling results. The amount and quality of information derived from this analysis is largely dependent on the number of years of data and the total record length. Changes in profile generally occur over many years; furthermore, in arid and semiarid regions these changes are hydrologically

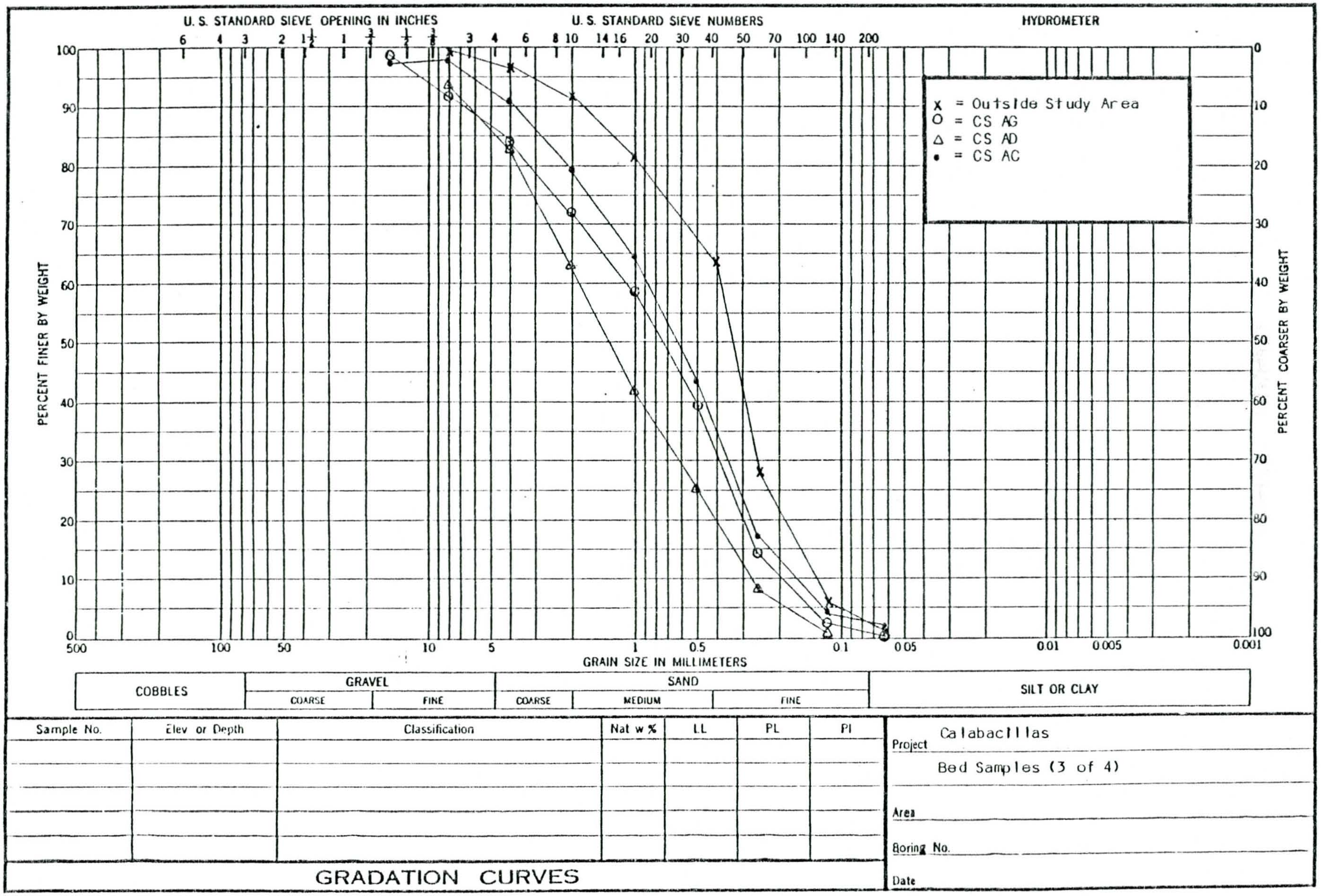
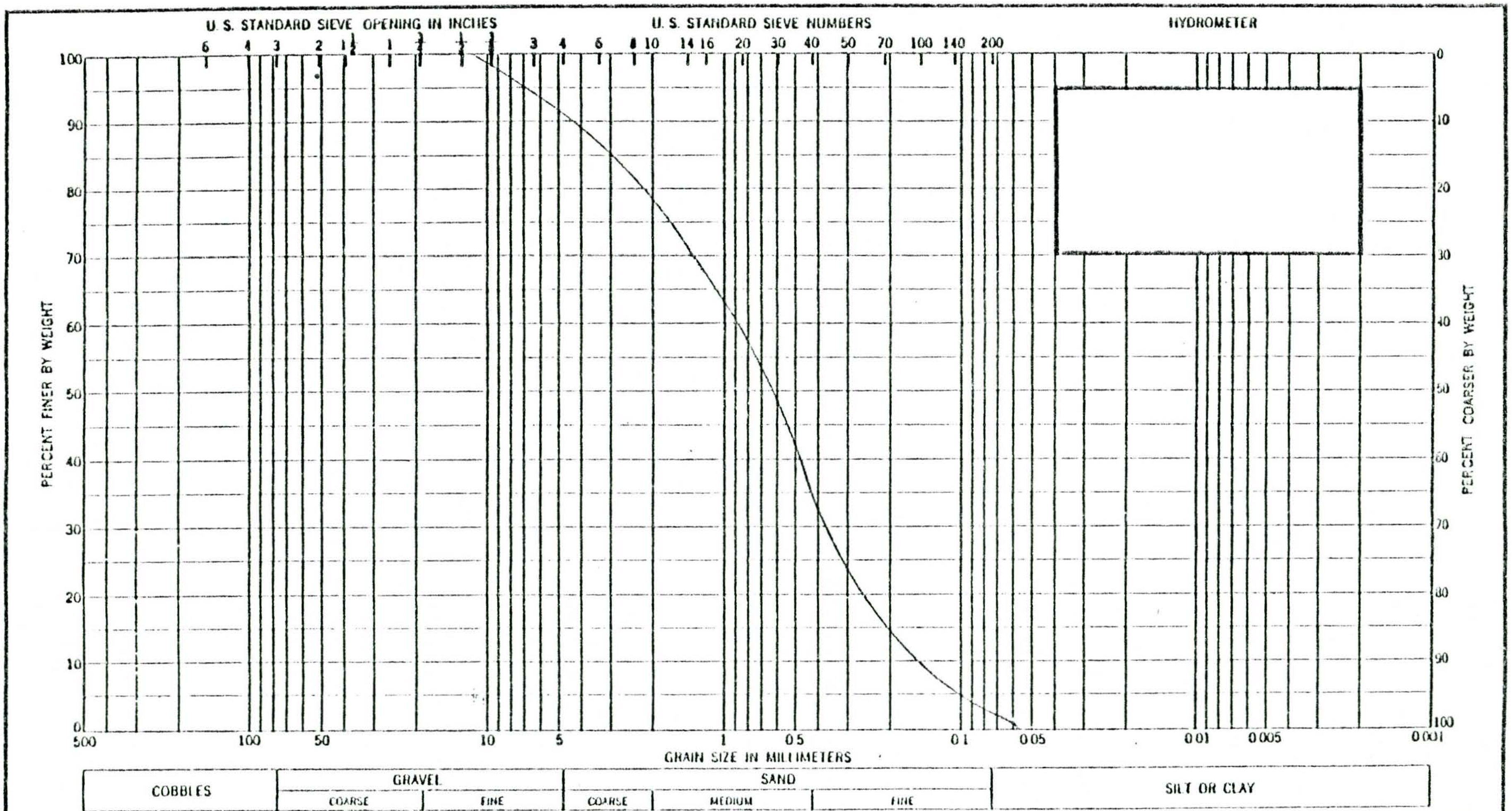


Figure 5.11a. Selected particle size gradations.



Sample No	Elev or Depth	Classification	Nat w %	LL	PL	PI

Project

Area

Boring No.

Date

GRADATION CURVES

Figure 5.11b. Representative gradation curve.

dependent. If there have been no significant floods in the period of record, then little change would be expected in the channel profile.

Application - Channel profile data can be developed from a variety of sources. Topographic mapping, for example, USGS 7.5-minute quadrangle sheets, is a readily available source, particularly for analyses involving a relatively long study reach (for short study reaches, the scale and contour interval of a 7.5-minute map may not provide sufficiently accurate information). Other sources of topographic mapping include county and city agencies and private parties who prepare mapping for development purposes, as well as for flood-plain mapping. Similarly, HEC-2 input data prepared for flood insurance studies can be a valuable source of data.

Less detailed data, both temporally and spatially, are often available from elevation data of pipeline crossings, railroad and highway bridges, diversion structures, and grade-control structures. With knowledge of the elevations of these structures, it is relatively simple to make field measurements of present bed elevations. Additionally, when available, the construction plans for these structures can provide valuable historical insight. The invert elevations at the time of construction are usually provided on the plans or can be deduced from the given information.

Finally, a field survey of the thalweg is valuable when time and/or budget constraints permit this level of effort. Surveying just the thalweg profile is relatively quick, compared to cross-section surveying, and is a good way to see the study reach in detail.

Example - During a relatively small flood (2-year flood) a bridge failure occurred, causing loss of life. Litigation resulted, and in support of the defense, a comprehensive engineering investigation of the failure was conducted. A profile analysis was part of the investigation and provided a substantial amount of information. Extensive data of the channel profile were first published in a Soil Conservation Service (SCS) flood-plain information report, based on a 1967 survey. Cross-section data collected by the Corps of Engineers (COE) were used to establish a 1972 profile. A previous analysis by an engineering firm provided a 1976 profile, based primarily on soundings from bridges. A COE General Design Memorandum (GDM) provided a 1978 profile. An additional data point for 1958 was derived from county bridge construction plans. The recorded top of pile elevation, pile length of 40 feet, and reported pile penetration of 23 feet into the existing creek bed placed the elevation of the bed at approximately 246 feet NGVD. While similar "as-built" data on other bridges in the study reach would have extended the profile for 1958, such data could not be obtained.

When the data for all these years were plotted, they provided a time-sequenced picture of profile changes. The comparison of these profiles established a strong system-wide degradation trend in the study reach. Combined with results from qualitative analysis, it was determined that the degradational trend had resulted from land-use changes (urbanization) that produced higher runoff volumes, and from extensive channelization beginning in the 1930's to straighten the system. From these and other results, it was concluded that the bridge failure at this location was imminent and could have occurred during any reasonable flow condition. Inspection of other bridges in the study reach by county maintenance crews led to extensive revetment and grade stabilization structures at all bridge crossings.

5.3.4 Incipient Motion Analysis

Discussion - An evaluation of relative channel stability can be made by evaluating incipient motion parameters. The definition of incipient motion is based on the critical or threshold condition where hydrodynamic forces acting on a grain of sediment have reached a value that, if increased even slightly, will move the grain. Under critical conditions, or at the point of incipient motion, the hydrodynamic forces acting on the grain are just balanced by the resisting forces of the particle. For given hydrodynamic forces, or equivalently for a given discharge, incipient motion conditions will exist for a single particle size. Particles smaller than this will be transported downstream and particles equal to or larger than this will remain in place.

Application - The Shields diagram (Figure 5.12) may be used to evaluate the particle size at incipient motion for a given discharge. The Shields diagram was developed through measurements of bed-load transport for various values of $\tau/(\gamma_s-\gamma)D$ at least twice as large as the critical value, and then extrapolated to the point of vanishing bed load. In the turbulent range, where most flows of practical engineering interest occur, Figure 5.12 suggests the parameter $\tau/(\gamma_s-\gamma)D$ is independent of flow conditions and the following relationship is established:

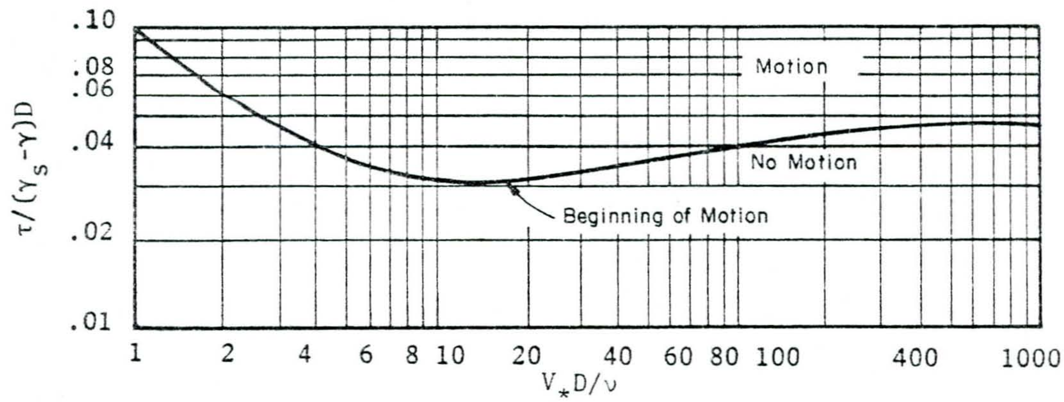


Figure 5.12. Shields' relation for beginning of motion (adapted from Gessler, 1971).

$$D_c = \frac{\tau}{0.047\ (\gamma_s - \gamma)} \qquad (5.5)$$

where D_c is the diameter of the sediment particle for conditions of incipient motion, τ is boundary shear stress, γ_s and γ are the specific weights of sediment and water, respectively, and 0.047 is a dimensionless coefficient referred to as the Shields parameter. [As originally proposed by Shields (1936), 0.060 was the parameter value in the turbulent range. The value of 0.047 was suggested by Meyer-Peter and Muller (1948), and further supported by Gessler (1971).] Any consistent set of units may be used with this equation.

The concept of incipient motion is of fundamental importance to sediment transport. Additionally, direct application of incipient motion concepts through Equation 5.5 are used in armor analysis and can provide useful insight for other Level II analyses. For example, given a discharge, hydraulic calculations can be used to determine information necessary to evaluate the boundary shear stress (Equation 4.7a or b) at various locations in a study reach. Using either computed or assumed standard values for water and sediment specific weights, the incipient motion particle size can then be evaluated for this discharge. This calculation may be repeated for other discharges characteristic of a given flood to determine what particle sizes would be in motion at various times during the flood. Results from this evaluation of incipient motion also indicate the total time during which various particle sizes would be in motion, as well as the percentage of time, relative to the total storm duration, that incipient motion conditions would be equaled or exceeded for each particle size.

Long-term incipient motion characteristics can be assessed in a similar fashion based on the annual hydrograph (i.e., annual record of mean daily or mean monthly discharge), instead of a single flood hydrograph. Such assessments are semi-quantitative since it must be assumed that the hydraulic properties at a point of interest have not changed appreciably over the long term. Additionally, results of any incipient motion analysis are generally more useful for analysis of gravel- or cobble-bed systems than for sand-bed systems. When applied to a sand-bed system, incipient motion results usually indicate that all particles in the bed material are capable of being moved (exceeding incipient motion conditions) for even very small discharges.

Example - Using results of a multiple-profile HEC-2 analysis, the hydraulic properties of an arroyo were known for a series of discharges characteristic of a 1980 flood. For each discharge the boundary shear stress was computed from Equation 4.7b and the incipient motion particle size from Equation 5.5. Results of this calculation are summarized on Figure 5.13.

Table 5.4 indicates the total time during which the various particle sizes of Figure 5.13 would be in motion. Also indicated in Table 5.4 is the percentage of time, relative to the total storm duration, that incipient motion conditions would be equaled or exceeded for each of these sizes. This type of information is useful in developing a Level II understanding of sediment transport characteristics, particularly in establishing the duration of significant transport during a flood.

5.3.5 Armoring Potential

Discussion - The armoring process begins as the non-moving coarser particles segregate from the finer material in transport. The coarser particles are gradually worked down into the bed, where they accumulate in a sublayer. Fine bed material is leached up through this coarse sublayer to augment the material in transport. As movement continues and degradation progresses, an increasing number of non-moving particles accumulate in the sublayer. This accumulation interferes with the leaching of fine material so that the rate of transport over the sublayer is not maintained at its former intensity. Eventually, enough coarse particles accumulate to shield, or "armor," the entire bed surface (Figure 5.14). When fines can no longer be leached from the underlying bed, degradation is arrested.

Examination of typical armor layers reveals several important characteristics:

- Less than a single complete covering layer of larger gravel particles seems to suffice for a total armoring effect for a particular discharge.

- A natural "filter" apparently develops between the larger surface particles and the subsurface material to prevent leaching of the underlying fines.

- The shingled arrangement of surface particles is not restricted to the larger material, but seems evident throughout the gravel gradation.

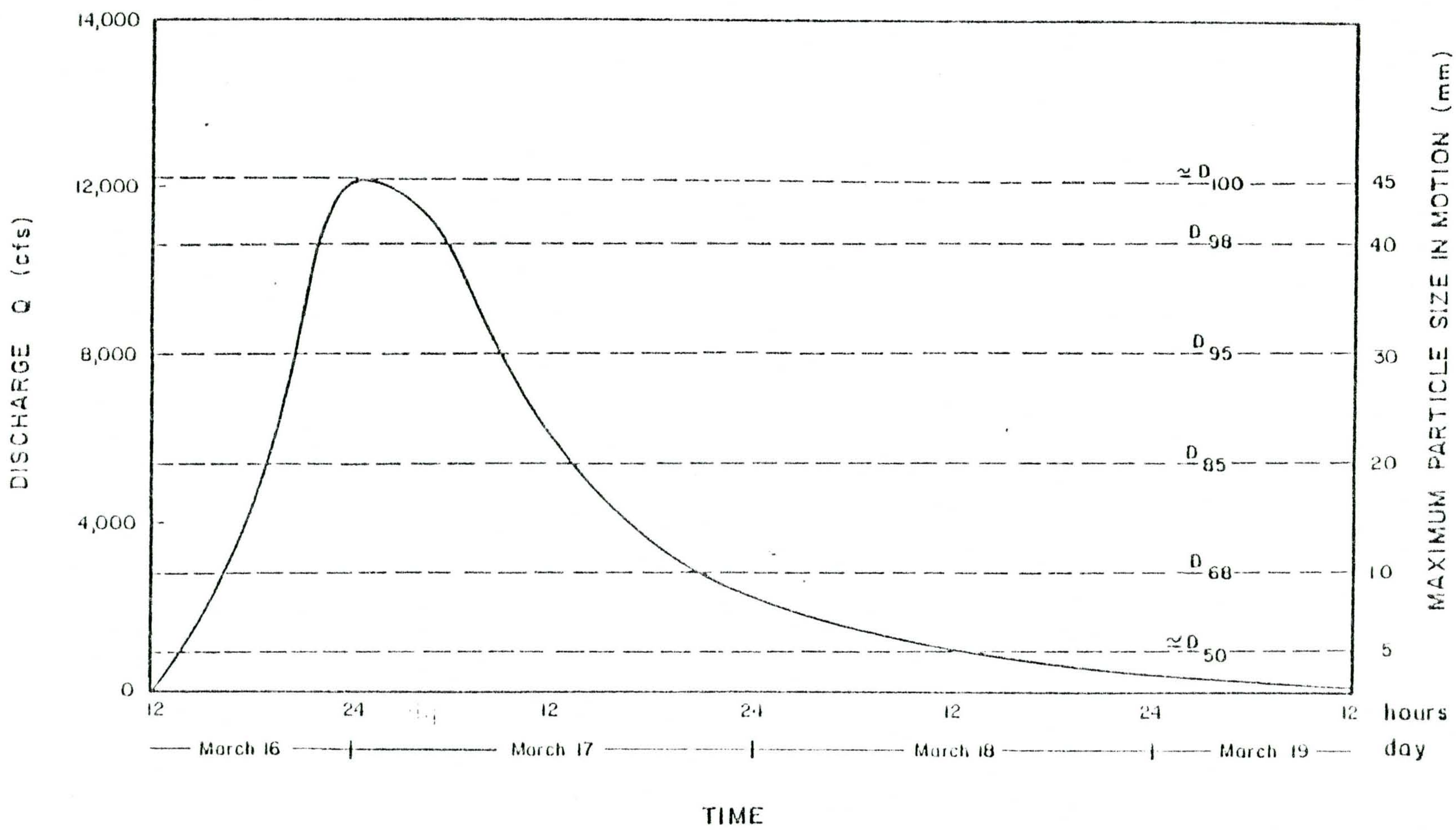
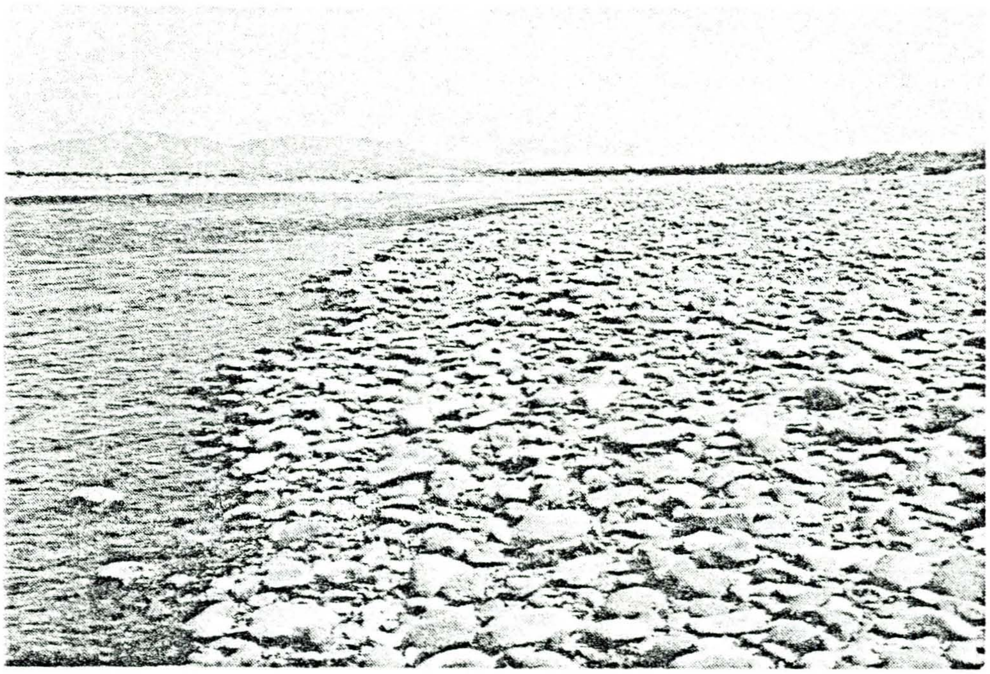


Figure 5.13. Frequency of motion for various particle sizes.

Table 5.4. Incipient Motion Characteristics.

Particle Size (mm)	Time in Motion (hrs)	Percentage of Total Storm Duration in Motion
5	56	78
10	34	47
20	22	30
30	15	20
40	9	13



Armored bed of Salt River upstream of Gilbert Road near Mesa, Arizona

Excavation through armor layer of the Salt River near Mesa, Arizona. Tape length shown in photograph is 24 inches.

Figure 5.14

An armor layer sufficient to protect the bed against moderate discharges can be disrupted during high flow, but may be restored as flows diminish. However, in a cobble-bed system the armoring condition is usually stable enough that the channel bed can be considered rigid, i.e., bed form conditions will not develop (see Figure 4.2). It is evident that an armor layer will tend to accumulate in areas of natural scour in the river, such as on the upstream ends of islands and bars. However, caution should be used in eliminating scour protection along the toe of levee or channel embankments under the assumption that an armor layer will be created uniformly along the toe. If a uniform armor layer is not present, or if one fails to develop at a predicted depth during a design flow, the levee toe could be undermined by scouring action, thus leading to failure.

Application - Potential for development of an armor layer can be assessed using Shields' criteria for incipient motion (see Section 5.3.4) and a representative bed-material composition. In this case a representative bed-material composition is that which is typical of the depth of anticipated degradation. Using Equation 5.5 the incipient-motion particle size can be computed for a given set of hydraulic conditions. If no sediment of the computed size or larger is present in significant quantities in the bed, armoring will not occur. The D_{90} to D_{95} size of the representative bed material is frequently found to be the size "paving the channels" when scouring is arrested. Within practical limits of planning and design, the D_{95} size is considered to be about the maximum size for pavement formation (SCS, 1977). Therefore, armoring is probable when the particle size computed from Equation 5.5 is equal to or smaller than the D_{95} size.

By observing the percentage of the bed material equal to or larger than the armor particle size (D_a) the depth of scour necessary to establish an armor layer (ΔZ_a) can be calculated from (USBR, 1984):

$$\Delta Z_a = y_a \left(\frac{1}{P_c} - 1\right) \qquad (5.6)$$

where y_a is the thickness of the armoring layer and P_c is the decimal fraction of material coarser than the armoring size. The thickness of the armoring layer (y_a) ranges from one to three times the armor particle size (D_a), depending on the value of D_a. Field observations suggest that a relatively stable armoring condition requires a minimum of two layers of armoring particles.

Example - As an example, consider the case where Equation 5.5 indicates that the critical particle size equals 1.5 inches and a representative bed-material gradation curve shows that this is the D_{90} size. Thus, the depth to formation of an armor layer would equal

$$\Delta Z_a = y_a \left(\frac{1}{P_c} - 1\right) = 2\ (1.5)\ \left(\frac{1}{0.1} - 1\right) = 27 \text{ inches}$$

It should be recognized that development of an armor layer does not occur uniformly across a channel bed, but rather tends to begin along the thalweg and at other points of natural scour in the channel.

5.3.6 Sediment Transport Capacity

Discussion - Sediment transport equations are used to determine the sediment transport capacity for a specific set of flow conditions. Knowledge of sediment transport capacity is required for many fluvial systems analyses, including evaluation of aggradation/degradation, general scour/deposition, and lateral migration. The first step in evaluating sediment transport capacity is to select one or more of the available equations for use in solving the given problem. Selection of an appropriate sediment transport relation is predicated on an understanding of the system being studied. For example, some formulas were developed from data collected in sand-bed streams where most of the sediment was transported as suspended load. Conversely, other equations pertain to conditions where bed-load transport is dominant. Study objectives also determine what portion of the sediment transport needs to be estimated and the level of accuracy required in such an estimate. If it is desirable to know the relative contributions of bed load and suspended load to the bed-material discharge, then formulas for each are available. Other formulas provide direct determination of bed-material discharge. A common feature of bed-material discharge sediment transport equations is that wash load is not included; however, there are methodologies that incorporate sediment sampling data, such as the modified Einstein procedure, that can be used to estimate total sediment transport rate (including wash load).

Available sediment transport equations range from theoretical or empirical methods to methods that require measured suspended sediment loads and/or other normal stream flow measurements. Table 5.5 summarizes some of the most commonly used sediment transport relations and their applications. As a result of the complexity of the Einstein bed load and suspended load methodologies, they will not be presented; however, it is important to note that the power relationships presented in this section were developed from a joint application of the MPM bed load and the Einstein suspended load equations. Similarly, the modified Einstein procedure, presented by Colby and Hembree (1955), will not be presented; however, the application of this procedure should be considered for evaluation of total sediment load when measured water and suspended sediment discharge data are available.

In using any sediment transport methodology, consideration should be given to solution by size fraction. Different transport capacities can be expected for different sediment sizes and some loss in accuracy may result from a calculation based on a single representative grain size (i.e., D_{50} size). Solution of the total bed-material discharge by size fraction analysis is based on a weighted average of the sediment transport for the geometric mean particle size representing various intervals of the sediment gradation curve. The number of intervals required depends on the accuracy desired and the characteristics of the gradation curve; however, adequate results are usually obtained using four to six intervals. As a final note, with any methodology it is desirable to verify results against measured data whenever possible and adjust equation parameters accordingly to obtain suitable results.

Application - Meyer-Peter, Muller Equation. Based on experiments with sand particles of uniform sizes, sand particles of mixed sizes, natural gravel, lignite, and baryta, Meyer-Peter and Muller (1948) developed a formula for estimating total bed-load transport. Most of the data used in developing the Meyer-Peter, Muller (MPM) equation were obtained in flows with little or no suspended sediment load. A common form of the MPM equation derived for a wide channel with plane-bed conditions is:

$$q_b = \frac{12.85}{\sqrt{\rho}\ \gamma_s} (\tau_o - \tau_c)^{1.5} \qquad (5.7a)$$

Table 5.5. Sediment Transport Calculation Procedures.

	Calculation				Application	
Procedure	Bed Load	Suspended Bed-Material Load	Bed-Material Load	Total[1] Sediment Load	Sand Bed	Cobble Bed
Meyer-Peter, Muller Equation (MPM)	X				X[2]	X
Einstein Bed Load Equation	X				X	X
Einstein Suspended Load Methodology		X			X	X
Power Relationships			X		X	
Colby Methodology			X		X	
Modified Einstein				X	X	X

[1] Includes wash load

[2] Must be supplemented with Einstein suspended load methodology to get suspended bed-material load component

where q_b is the bed-load transport rate in cubic feet per second (cfs) per unit width for a specific size of sediment, τ_0 is the tractive force (boundary shear stress), τ_c is the critical tractive force, ρ is the density of water and γ_s is the specific weight of dry sediment. The critical tractive force is defined by the Shields parameter (see Section 5.3.4). The tractive force or boundary shear stress acting under the given flow conditions is most often defined by Equation 4.7b. The use of Equation 5.7a is not recommended if dunes or antidunes are expected due to the plane bed assumption in its derivation. Other more complex forms of the equation are available for use under these circumstances (see USBR, 1960). Any application of the MPM relationships provides an estimate of bed-load transport only and should be supplemented by other methods if appreciable suspended bed-material transport is suspected.

A general form of the MPM equation was presented by Shen (1971) as

$$q_b = a_4 (\tau_0 - \tau_c)^{b_4} \tag{5.7b}$$

in which a_4 and b_4 are constants. When the constants in this equation are calibrated with field data, good results are usually obtained.

A complete discussion of Meyer-Peter's formulas for beginning of motion and sediment transport is provided by Meyer-Peter and Muller (1948).

Empirical Power Relationships. Using a computer-generated solution of the Meyer-Peter, Muller bed-load transport equation combined with Einstein's integration of the suspended bed-material discharge, a procedure has been developed for estimating total bed-material discharge in sand-bed channels from power relationships of the form (Simons, Li and Fullerton, 1981)

$$q_s = a \; Y_h^b \; V^c \tag{5.8a}$$

where q_s is the bed-material discharge in cfs per unit width, Y_h is hydraulic depth, V is the average velocity and a, b, and c are regression coefficients. Using a computer-generated data base, representative values for coefficients a, b, and c were determined for various sediment sizes, gradations and bed slopes. Results of this analysis are presented in Tables 5.6a and 5.6b. For evaluation of transport capacity at a sediment size or gradient coefficient not tabulated, interpolation between q_s values for sediment sizes and gradation coefficients bracketing the given size is required. The curves resulting from a plot of D_{50} or G versus a, b, or c are not linear relationships. Therefore, prior to attempting a linear mathematical interpolation between these coefficients and exponents, the user may want to plot D_{50} or G versus the tabulated values for a, b, and c and use the resulting curves for a visual interpolation of these values.

As Table 5.6 indicates, sediment transport rates are highly dependent on velocity, and to a lesser degree on depth. Sediment transport for some sediment sizes is directly proportional to depth, whereas transport of other sizes is inversely proportional to depth. Transport of smaller sediment sizes is generally proportionally dependent on depth since the smaller material is more easily suspended and the resulting sediment concentrations are more uniform. Thus, the larger the depth, the greater the amount of sediment that will be suspended for a given velocity. Larger sediment particles, on the other hand, are more difficult to suspend and keep in suspension. For a given velocity, as depth increases the intensity of turbulent transfer properties for these larger sizes decreases. The increase in area available for suspended sediment associated with the increased depth does not totally counterbalance the reduced turbulent transfer characteristics, resulting in an inverse relationship between transport and depth for larger particles. Sediment sizes exhibiting little dependence on depth fall between these extremes.

As an alternative to Equation 5.8a and Tables 5.6a and 5.6b, a single relationship was later developed (Zeller and Fullerton, 1983):

$$q_s = 0.0064 \frac{n^{1.77} V^{4.32} G^{0.45}}{Y_h^{0.30} D_{50}^{0.61}} \qquad (5.8b)$$

where n is Manning's roughness coefficient (based on bed forms and grain size roughness), V is the mean velocity, G is the gradation coefficient, Y_h is the hydraulic depth, and D_{50} is the median diameter. In this equation all units are in the ft-lb-sec system, except D_{50}, which is in millimeters.

Table 5.7 lists the range of parameters considered in the development of the sediment transport relations given in Tables 5.6a and 5.6b and in development of Equation 5.8b. The applicability of either methodology to any specific set of conditions should be checked in Table 5.7. It should be noted

Table 5.6a. Results of Regression Analysis.

$(0.001 \leq S_o \leq 0.01; \quad q_s = a \; Y_h^b \; V^c)$

D_{50} =	0.1 mm	0.25 mm	0.5 mm	1.0 mm	2.0 mm	3.0 mm	4.0 mm	5.0 mm
				G = 1.0				
a	2.90×10^{-4}	1.81×10^{-5}	3.19×10^{-6}	2.06×10^{-6}	3.45×10^{-6}	5.05×10^{-6}	6.15×10^{-6}	6.35×10^{-6}
b	0.505	0.0446	-0.363	-0.628	-0.693	-0.672	-0.652	-0.639
c	3.43	4.43	5.01	5.03	4.60	4.30	4.13	4.06
				G = 2.0				
a		6.80×10^{-5}	1.48×10^{-5}	3.54×10^{-6}	2.46×10^{-6}	2.81×10^{-6}	3.14×10^{-6}	
b		0.315	0.0501	-0.324	-0.587	-0.649	-0.644	
c		3.83	4.31	4.78	4.79	4.62	4.49	
				G = 3.0				
a			5.25×10^{-5}	1.61×10^{-5}	3.71×10^{-6}			
b			0.317	0.112	-0.260			
c			3.76	4.11	4.61			
				G = 4.0				
a				4.31×10^{-5}				
b				0.324				
c				3.70				

S_o = bed slope

q_s = sediment transport rate in cfs (unbulked)

Y_h = hydraulic depth in feet (area/top width)

V = average velocity in fps

G = gradation coefficient

Table 5.6b. Results of Regression Analysis.

$(0.01 < S_o \leq 0.04; \quad q_s = a \ Y^b \ V^c)$

D_{50} =	0.1 mm	0.25 mm	0.5 mm	1.0 mm	2.0 mm	3.0 mm	4.0 mm	5.0 mm
				G = 1.0				
a	4.74×10^{-4}	7.45×10^{-5}	1.66×10^{-5}	5.80×10^{-6}	3.58×10^{-6}	3.62×10^{-6}	4.03×10^{-6}	4.50×10^{-6}
b	0.557	0.305	0.0530	-0.198	-0.427	-0.532	-0.587	-0.615
c	3.22	3.76	4.17	4.42	4.45	4.37	4.27	4.18
				G = 2.0				
a		1.27×10^{-4}	3.81×10^{-5}	1.16×10^{-5}	5.25×10^{-6}	4.20×10^{-6}	3.89×10^{-6}	
b		0.383	0.199	-0.0318	-0.264	-0.385	-0.459	
c		3.56	3.88	4.18	4.33	4.34	4.31	
				G = 3.0				
a			7.40×10^{-5}	3.02×10^{-5}	1.08×10^{-5}			
b			0.310	0.161	-0.0502			
c			3.65	3.86	4.10			
				G = 4.0				
a				5.30×10^{-5}				
b				0.264				
c				3.67				

S_o = bed slope

q_s = sediment transport rate in cfs (unbulked)

Yh = hydraulic depth in feet (area/top width)

V = average velocity in fps

G = gradation coefficient

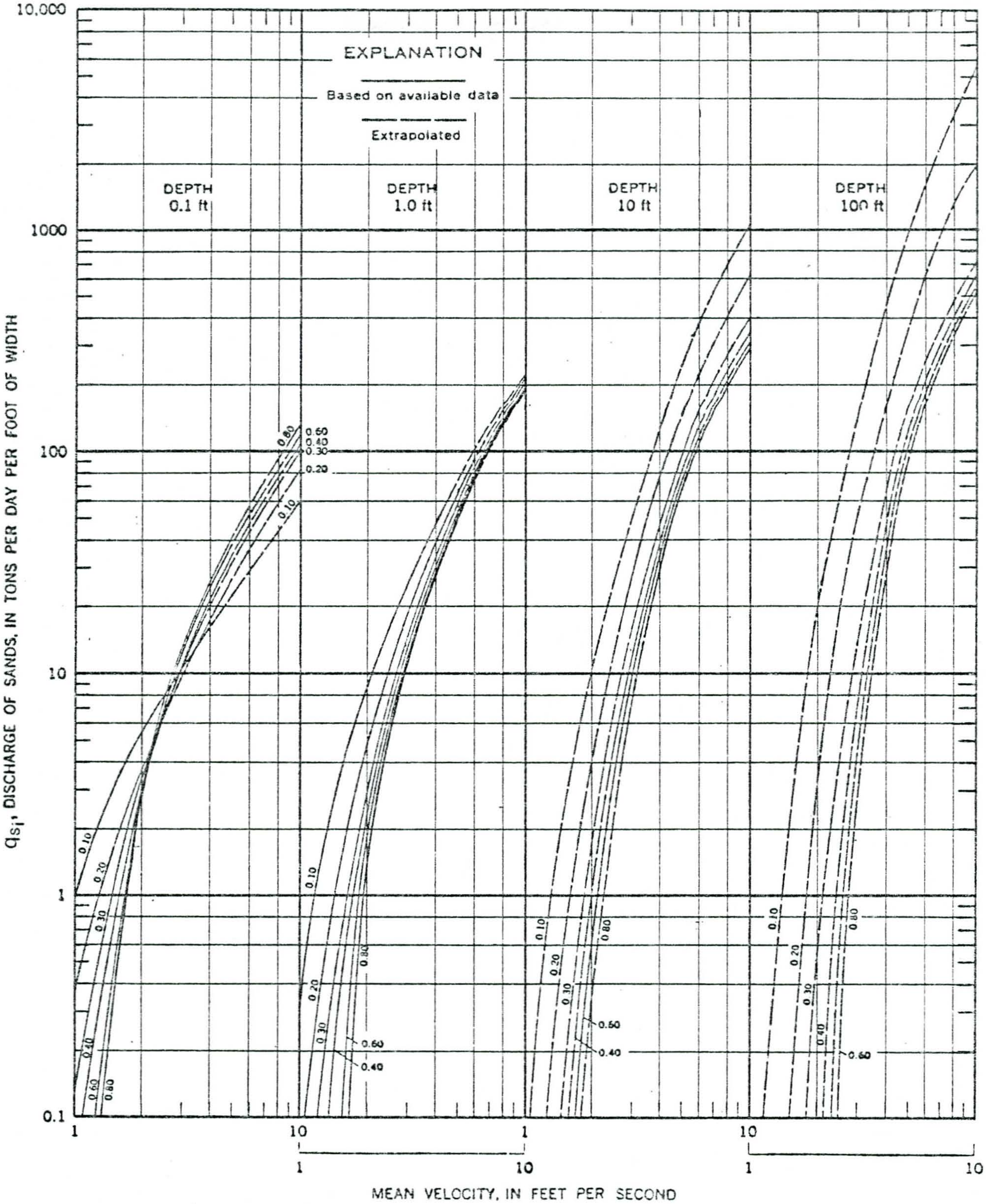
Table 5.7. Range of Parameters Examined for Power Relationships.

Parameter	Value Range: Equation 5.8a when used with Tables 5.6a and 5.6b	Value Range: Equation 5.8b
Froude No.	<4	unlimited
Velocity	5-26 (ft/sec)	3-30 (ft/sec)
Manning's n	0.025	0.018-0.035
Bed Slope	0.001-0.040 (ft/ft)	0.001-0.040 (ft/ft)
Unit Discharge	5-200 (cfs/ft)	10-200 (cfs/ft)
Particle Size	$D_5 \geq 0.062$ mm	0.5 mm $\leq D_{50} \leq$ 10 mm
	$D_{90} \leq 15$ mm	
Depth	Unlimited	1-20 ft
Gradation Coefficient	1-4	2-5

that these equations are based on the assumption that all sediment sizes present in the bed are transportable by the flow. If armoring is a possibility (see Section 5.3.5), the regression relations are not valid. Since the equations were developed for sand-bed channels, they do not apply to conditions where the bank material has cohesive properties. Transport rates would be overpredicted for a cohesive channel condition. For conditions meeting the criteria of Table 5.7, as well as other criteria mentioned, either equation should provide results within ten percent of the theoretical values computed with the Meyer-Peter, Muller bed load and Einstein suspended bed-material load methodologies that were used to develop the regression equations.

Colby's Approach. Colby (1964) developed the graphical procedure shown in Figures 5.15 and 5.16 for determining bed-material discharge (tons/day of dry sediment) in sand-bed channels. In developing his computational curves Colby was guided by Einstein's bed-load function (Einstein, 1950) and an immense amount of data from streams and flumes (Simons and Richardson, 1966). However, it should be understood that all curves for the 100-foot depth, most curves of the ten-foot depth, and some of the curves of 1.0-foot and 0.1-foot depths (Figure 5.15) are not based entirely on data but are developed from limited data and theory.

In utilizing Figures 5.15 and 5.16 to compute the bed-material discharge, the following procedure is required: (1) the required data are mean velocity V, depth (typically hydraulic depth), Y_h, median size of bed material D_{50}, water temperature, and fine-sediment concentration; (2) then the uncorrected sediment discharge q_{s_i} for the given V, Y_h and D_{50} can be found from Figure 5.15 for the two depths that bracket the desired depth. A logarithmic scale of depth versus q_{s_i} is used to interpolate in order to determine the bed-material discharge per unit width for the actual Y_h, V and D_{50}; (3) two correction factors, k_1 and k_2, shown in Figures 5.16a and 5.16b, respectively, 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 5.16c is applied to correct for sediment size effect. True sediment discharge q_s corrected for water temperature effect, presence of fine suspended sediment, and sediment size is given by



Note: The curves on this chart represent particle size in mm.

Figure 5.15. 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 (Colby, 1964).

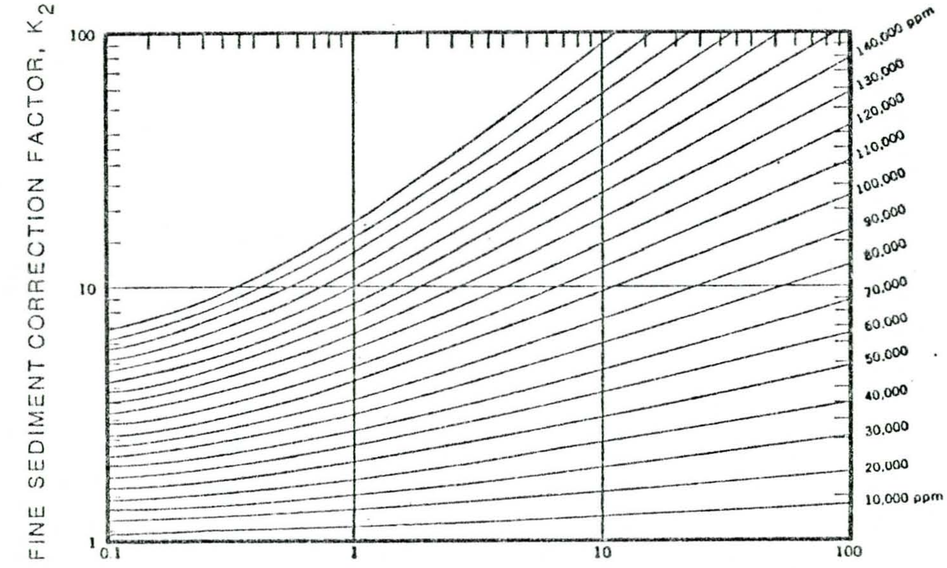


FIGURE 5.16b

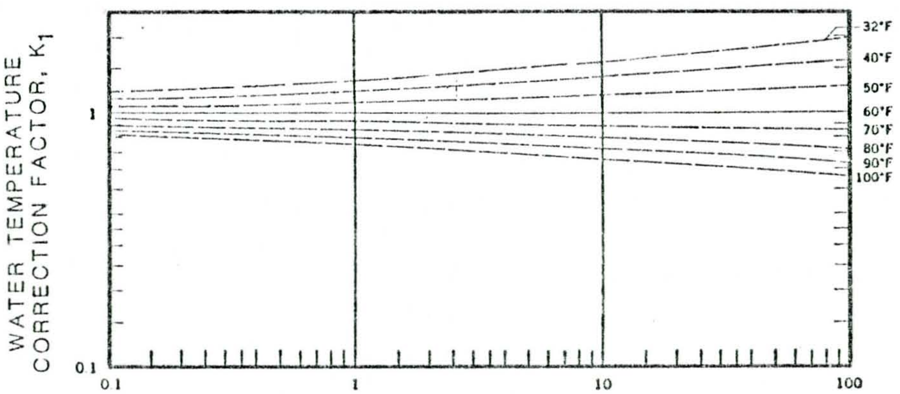


FIGURE 5.16a

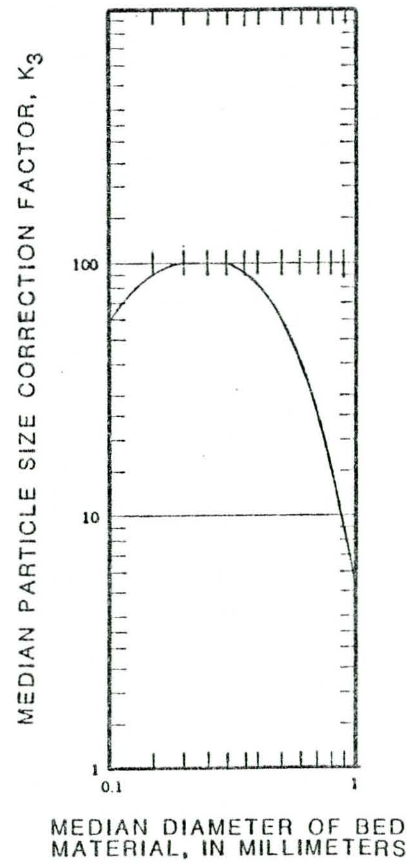


FIGURE 5.16c

Figure 5.16. Colby's correcti[illegible] curves for temperature and fine sediment (Colby, 1964)

$$q_s = [1 + (k_1 k_2 - 1)\ 0.01\ k_3]\ q_{s_i} \tag{5.9}$$

As Figure 5.16 shows, $k_1 = 1$ when the temperature is 60° F, $k_2 = 1$ when the concentration of fine sediment is negligible, and $k_3 = 100$ when D_{50} lies between 0.2 mm and 0.3 mm.

In spite of many inaccuracies in the available data and uncertainties in the graphs, Colby found

> "...about 75 percent of the sand discharges that were used to define the relationships were less than twice or more than half of the discharges that were computed from the graphs of average relationship. The agreement of computed and observed discharges of sands for sediment stations whose records were not used to define the graphs seemed to be about as good as that for stations whose records were used."

Example - Calculation of Sediment-Transport Rates Using:

A. Meyer-Peter, Muller (MPM) Bed-load Function

B. Colby Method

Before beginning the examples, the reader should remember that all sediment transport equations do not compute the same component of total sediment load. Table 5.5 was developed as an easy reference to make this distinction. In the following examples the Meyer-Peter, Muller equation is used to compute the bed-load transport rate. Since this equation was derived from flume experiments using flows with little or no suspended sediment load, it is not recommended for applications where suspended bed-material load is estimated to be a major component of the total sediment load. In contrast, the power relationships and the Colby Method were developed on the basis of predicting total bed-material transport rate.

Because of this difference between transport equations, the following examples will employ the MPM equation to evaluate the bed-load discharge for a gravel-cobble bed stream which would be expected to have very little suspended bed-material load, while the Colby Method will be applied to a sand-bed channel having both suspended bed-material load and bed load components.

Due to the simplicity of the power relationships (Equations 5.8a and 5.8b), no numerical examples will be presented.

Part A, Meyer-Peter, Muller Equation: The gradation curve for the bed material from a gravel-cobble bed stream was divided into three size fractions. The geometric mean particle size and weight of each fraction is listed below:

Fraction #1 (33 1/3% of total sample weight): $D_G = D_{25} = 0.05$ ft

Fraction #2 (33 1/3% of total sample weight): $D_G = D_{50} = 0.10$ ft

Fraction #3 (33 1/3% of total sample weight): $D_G = D_{75} = 0.15$ ft

This reach of the stream is further defined by the following parameters:

For $Q = 5{,}000$ cfs, mean channel velocity, $\overline{V} = 8$ fps.

Darcy-Weisbach friction factor, $f = 0.06$

Specific weight of sediment, $\gamma_s = 165.4\ \mathrm{lb/ft^3}$

Density of water, $\rho = 1.9\ \mathrm{lb\text{-}sec^2/ft^4}$

Using the MPM Equation (Equation 5.7a), the following steps are required to compute bed-load discharge:

1. The boundary shear stress, τ_o, is computed as follows:

$$\tau_o = \frac{1}{8}\rho f V^2$$

$$\tau_o = \frac{1}{8}(1.9)(0.06)(8)^2$$

$$\tau_o = 0.91\ \mathrm{lb/ft^2}$$

2. The critical shear stress, τ_c, is found using Shields' relation:

$$\tau_c = 0.047\ D_c\ (\gamma_s - \gamma)$$

for $D_c = D_{25}$, $\tau_c = (0.047)(0.05)(165.4 - 62.4) = 0.24\ \mathrm{lb/ft^2}$

for $D_c = D_{50}$, $\tau_c = (0.047)(0.10)(165.4 - 62.4) = 0.48\ \mathrm{lb/ft^2}$

for $D_c = D_{75}$, $\tau_c = (0.047)(0.15)(165.4 - 62.4) = 0.73\ \mathrm{lb/ft^2}$

3. The MPM equation can now be used to compute the bed-load transport rate for each of the three sediment size fractions.

for D_{25}, $q_{b_{25}} = \dfrac{12.85}{\gamma_s\sqrt{\rho}}(\tau_o - \tau_c)^{1.5} = \dfrac{12.85}{(165.4)\sqrt{1.9}}(0.91 - 0.24)^{1.5}$

$$q_{b_{25}} = 0.031\ \mathrm{cfs/ft}$$

for D_{50}, $q_{b_{50}} = \dfrac{12.85}{(165.4)\sqrt{1.9}} (0.91 - 0.48)^{1.5}$

$$q_{b_{50}} = 0.016 \text{ cfs/ft}$$

for D_{75}, $q_{b_{75}} = \dfrac{12.85}{(165.4)\sqrt{1.9}} (0.91 - 0.73)^{1.5}$

$$q_{b_{75}} = 0.004 \text{ cfs/ft}$$

4. The total unit bed-load transport rate can now be computed as the weighted average of the transport rates for each of the selected sediment size fractions. This procedure is accomplished as follows:

$$\text{total } q_b = \frac{\Sigma\,(q_{b_i} \times \%\text{ total weight})}{\Sigma\,(\%\text{ total weight})}$$

$$q_b = \frac{(0.031)(33\ 1/3\%) + (0.016)(33\ 1/3\%) + (0.004)(33\ 1/3\%)}{33\ 1/3\% + 33\ 1/3\% + 33\ 1/3\%}$$

$$q_b = 0.017 \text{ cfs/ft}$$

Part B, Colby Method: Colby calculations will be made using the single median bed particle size as well as the sediment size fraction approach. Water temperature and fine sediment concentration are assumed equal to 70°F and 10,000 ppm, respectively, for the example calculations.

a. Calculations Using Single Bed Particle Size. The calculation will be made for a dicharge of 410 cfs, for which $Y_h = 1.36$ ft, $V = 2.93$ fps, and $b = 103$ ft. From Figure 5.17, the median bed particle size D_{50} is 0.225 mm. The solution involves the following steps:

1. Enter Figure 5.15 with a velocity of 2.93 fps for a depth of 1.0 ft and 10.0 ft and read the following values of q_{s_i} for $D_{50} = 0.225$ mm:

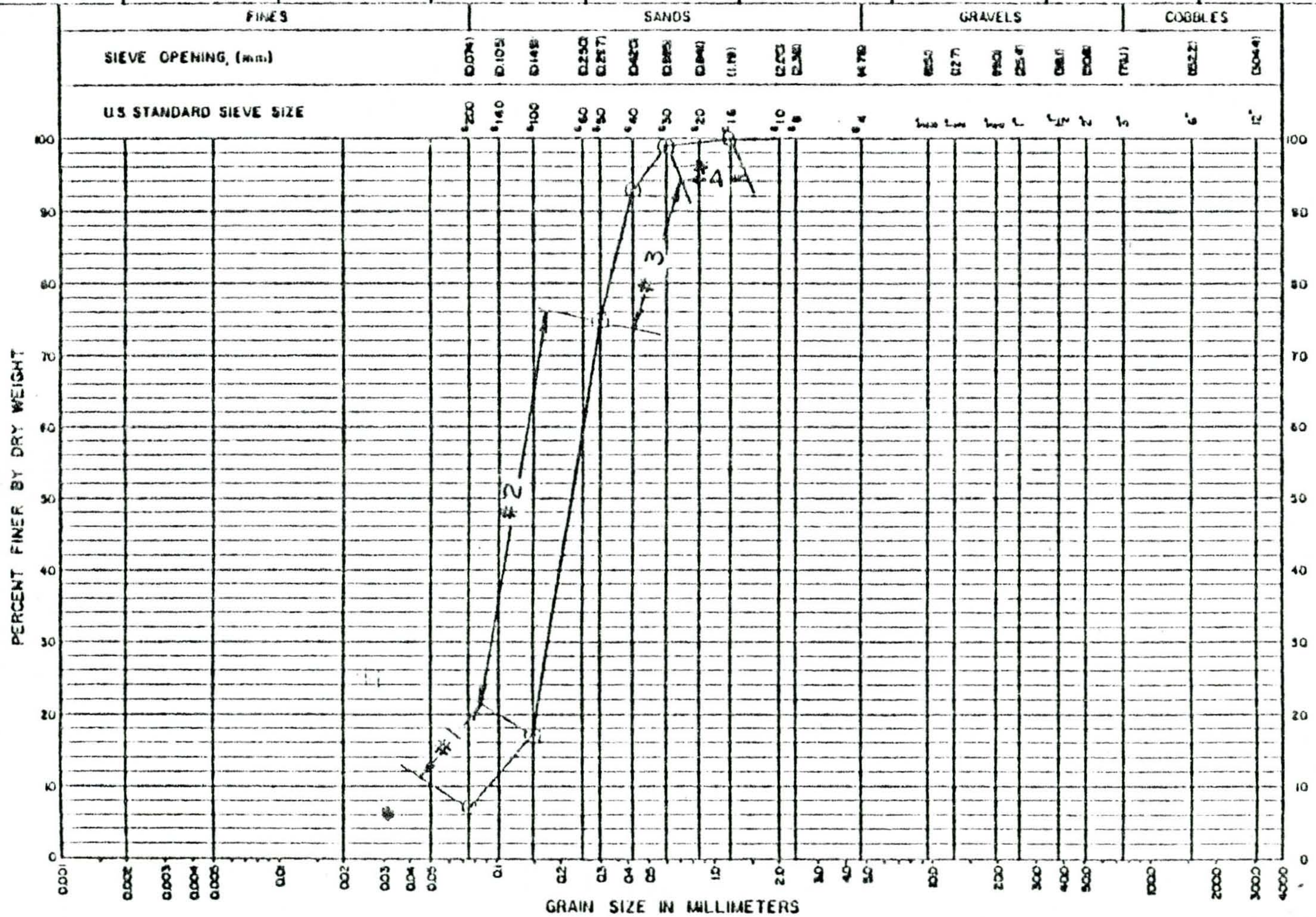
   Depth = 1.0 ft; q_{s_i} = 15.5 tons/day/ft of width

   Depth = 10.0 ft; q_{s_i} = 21.5 tons/day/ft of width

2. The depth and q_{s_i} values determined in Step 1 are plotted on log-log paper in order to interpolate a value of q_{s_i} for the given depth of 1.36 ft. This plot, which is shown in Figure 5.18, yields the following result:

GRAIN SIZE DISTRIBUTION

D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	D_{MAX}	C_u	C_c
0.09	0.13	0.17	0.225	0.25	0.35	2.0	2.8	1.3



SPECIFIC GRAVITY (G_s)		ATTERBERG LIMITS						SOLUBLE SALTS	SHRINKAGE LIMIT	UNDISTURBED CONDITION	
		NATURAL MOISTURE		AIR DRY		OVEN DRY				MOISTURE	DRY UNIT WEIGHT
(-) #4	(+) #4	LL	PI	LL	PI NP	LL	PI	%	%	19.3 %	g/cc pcf

REMARKS:

SYMBOL: SP-SM

DESCRIPTION: Poorly Graded Silty Fine Sand

Figure 5.17. Bed material gradation curve.

5.64

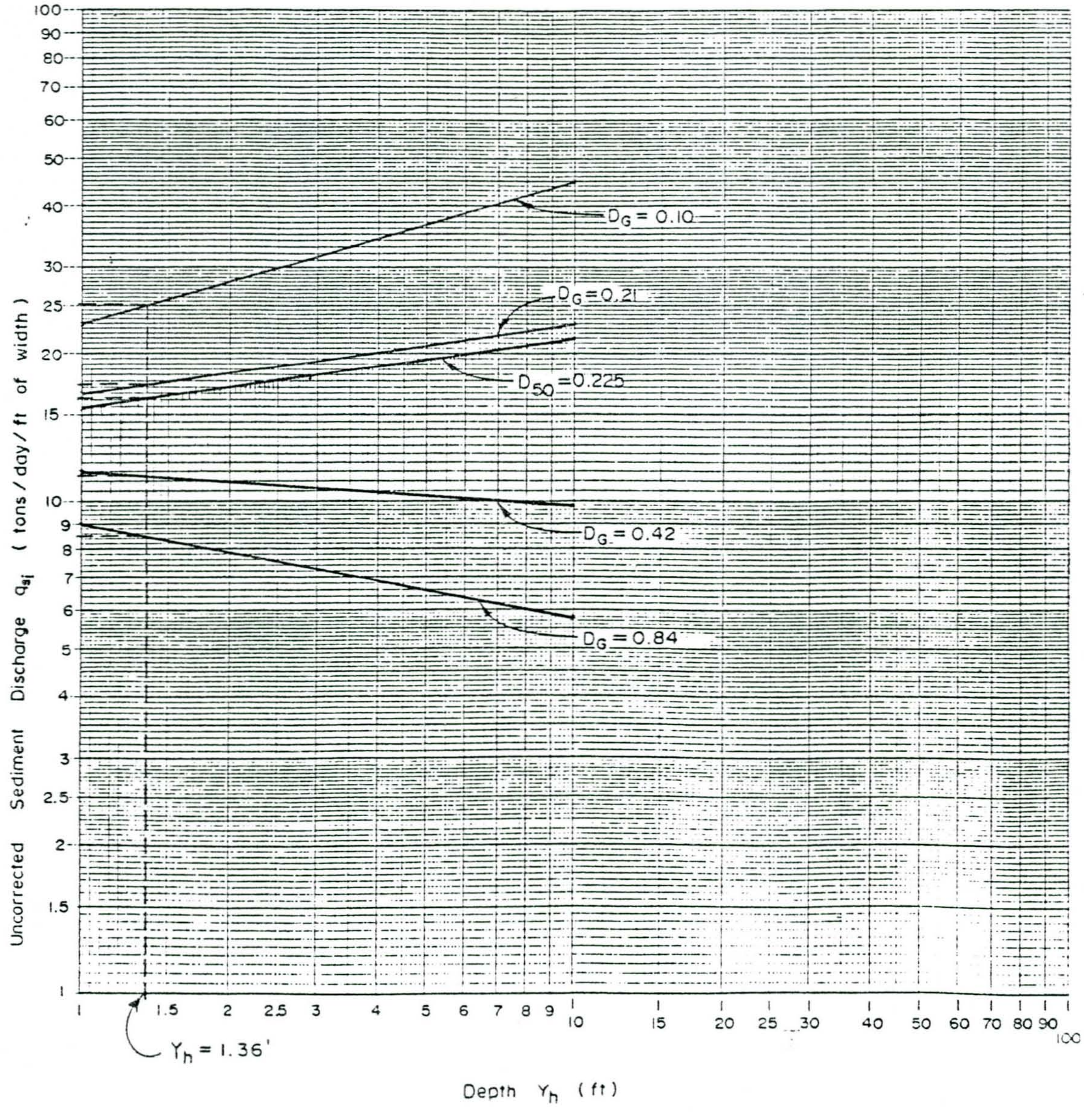


Figure 5.18. Log - log plot for uncorrected sediment discharge (q_{s_i}) versus hydraulic depth (Y_h).

Depth = 1.36 ft; q_{s_i} = 16.2 tons/day/ft of width

3. Water temperature correction: Since the information in Figure 5.15 is based on a water temperature of 60°F and the given water temperature is 70°F, an adjustment must be made to compensate for the difference. This correction is made by entering Figure 5.16a with a depth of 1.36 ft and proceeding to the line for 70°F. A correction value, K_1, is then read as 0.92.

4. Figure 5.16b is now used to determine the correction factor for the fine sediment concentration of 10,000 ppm. Enter this curve with a depth of 1.36 ft and proceed to the curve for 10,000. A correction value, K_2, is then read as 1.2.

5. Sediment size adjustment. Since the D_{50} bed particle size (0.225 mm) falls within the 0.2 to 0.3 mm range, a correction for sediment size is not necessary. For this condition, the K_3 correction factor = 100. This can be verified by entering Figure 5.16c with a median sediment size of 0.225 mm.

6. The true sediment transport, q_s, corrected for water temperature effect, presence of fine suspended sediment, and sediment size, is now computed as:

$$q_s = [1 + (K_1K_2 - 1)\ 0.01\ K_3]\ q_{s_i}$$

$$q_s = [1 + (0.92 \times 1.2 - 1)(0.01)(100)]\ 16.2$$

$$q_s = 14.52 \text{ tons/day/ft of width}$$

7. For the given channel width of 103 ft, the total bed-material transport rate, Q_s, for the cross section is $Q_s = q_s \times b = (14.52)(103) =$ 1,495.6 tons/day.

8. The sediment concentration by weight, c, is computed as follows:

$$c = \frac{Q_s\ \gamma_s}{Q\gamma + Q_s\ \gamma_s} = \frac{Q_s\ (G_s)}{Q + Q_s\ (G_s)}$$

where G_s = specific gravity of sediment (γ_s/γ).

Since the Colby Method gives sediment transport in tons/day, a conversion to cfs must be made before the above formula can be used. This conversion is made as follows:

$$Q_s = \frac{1{,}495.6 \text{ tons}}{\text{day}} \times \frac{1 \text{ day}}{86{,}400 \text{ sec}} \times \frac{1 \text{ ft}^3}{165 \text{ lb of sediment}} \times \frac{2{,}000 \text{ lb}}{1 \text{ ton}}$$

$$Q_s = 0.21 \text{ cfs}$$

Therefore, $c = \frac{(0.21)(2.65)}{410 + (0.21)(2.65)} = 0.0013555$, or 1,355 ppm by weight

b. Calculations Using Sediment Size Fractions. The bed material that was used in the previous example had a mean particle size D_{50} of 0.225 mm. To make the sediment transport calculations more representative of changes in the bed-material gradation curve, solution by size fraction is employed. Using this method, the gradation curve is divided into increments of similar size characteristics. The curve could be broken into soil fractions, e.g., coarse gravel, fine gravel, coarse sand, medium sand, etc., or it could be divided into even increments such as 20 percent by weight intervals. Other methods or criteria could be used as long as the individual size fractions are associated with particle sizes of similar characteristics. The gradation curve for this example (Figure 5.17) was divided into four size fractions, primarily on the basis of noticeable changes in the slope of the curve.

Once the gradation curve has been subdivided, the geometric mean particle size is determined for each grain size interval. The following steps illustrate the Colby Method calculations by size fraction for the same discharge and hydraulic conditions used previously.

1. The bed-material gradation curve (Figure 5.17) is subdivided into four increments and the geometric mean particle size for each increment calculated as given in Table 5.8. The adjustment to the fractional sample weight percentages in Table 5.8 is required to account for the seven percent of the total sample weight that was finer than the #200 sieve. Rather than resort to a hydrometer or similar analysis to grade the seven percent of fine material, this percentage was prorated among the four size fractions. If fine material constituted a significant portion of the total sample weight, a hydrometer analysis might be warranted.

2. Using the hydraulic parameters listed in part (a) of this example, enter Figure 5.15 with a velocity of 2.93 fps for depths of 1.0 ft and 10.0 ft and read values of q_{s_i} for each of the four size fractions (see Table 5.9). It should be noted that an estimate had to be made for the q_{s_i} value for the 0.84 mm size fraction since this value lies slightly outside the range of particle sizes shown in Figure 5.15. This procedure should be used with caution. If the mean size fractions are significantly outside the range of values shown in Figure 5.15, the Colby Method should not be used. In this case, the single non-conforming size frac-

Table 5.8. Geometric Mean Calculations for Colby Example.

Grain Size Interval (mm) D_i to D_j	Geometric Mean $D_G = \sqrt{D_i \times D_j}$ (mm)	Percent of Total Sample Weight	Adjusted % of Total Sample Weight
1.2 - 0.59	$\sqrt{(1.2)(0.59)} = 0.84$	1.0	1.1
0.59 - 0.30	$\sqrt{(0.59)(0.30)} = 0.42$	24.0	25.8
0.30 - 0.145	$\sqrt{(0.30)(0.145)} = 0.21$	58.0	62.4
0.145 - 0.075	$\sqrt{(0.145)(0.075)} = 0.10$	10.0	10.7
	Total:	93.0	100.0

Table 5.9. Uncorrected Sediment Transport Rate, q_{s_i} (tons/day/ft) for Colby Example.

	D_G (mm)			
Depth (feet)	0.84	0.42	0.21	0.10
1.0	9.0	11.5	16.5	23.0
10.0	5.8	9.8	23.0	45.0

tion constitutes only 1.1 percent of the total sample weight. Accordingly, any error induced by this procedure should be minimal.

3. The depth and q_{s_i} values determined in Step 2 are plotted on log-log paper in order to interpolate a value of q_{s_i} for the given depth of 1.36 ft. This plot, which is shown in Figure 5.18, yields the following results:

D_G (mm)	0.84	0.42	0.21	0.10
q_{s_i} (tons/day/ft of width)	8.5	11.3	17.2	25.1

4. The water temperature correction, K_1, and fine sediment concentration correction, K_2, are the same as computed in part (a) of this example, since these factors are not a function of the bed particle gradation curve.

$K_1 = 0.92$

$K_2 = 1.20$

5. A sediment size adjustment factor, K_3, will be required for three of the four size fractions since they lie outside the 0.2 to 0.3 mm range. The correction factors from Figure 5.16c are summarized below.

D_G (mm)	0.84	0.42	0.21	0.10
K_3	12	80	100	60

6. The true sediment transport rate, q_s, corrected for water temperature effect, presence of fine suspended sediment, and sediment size, is now computed for each size fraction using:

$$q_s = [1 + (K_1 K_2 - 1)\ 0.01\ K_3]\ q_{s_i}$$

The results are summarized in the following table.

D_G (mm)	0.84	0.42	0.21	0.10
q_s (tons/day/ft of width)	8.61	12.19	19.04	26.73

7. Once the unit transport rate is computed for each size fraction in Step 6, the actual transport amount of each size fraction within the total bed sample is determined by multiplying the computed transport rates times the percentage of each size fraction in the bed sample (see Table 5.10).

8. Knowing the total unit bed-material discharge from Step 7, the total bed-material discharge from the entire channel cross section can now be calculated by multiplying the sediment discharge rate from Step 7 by the effective channel width.

$$Q_s = b \times q_{s_T}$$

$$= (103)(17.98) = 1{,}851.9 \text{ tons/day}$$

Converting to cubic feet per second yields:

$$Q_s = (1{,}851.9)(1.403 \times 10^{-4})$$

$$Q_s = 0.26 \text{ cfs}$$

9. The sediment concentration is now computed.

$$C = \frac{Q_s\,(G_s)}{Q + Q_s\,(G_s)}$$

$$C = \frac{(0.26)(2.65)}{410 + (0.26)(2.65)}$$

$$C = 0.0016776 \quad \text{or} \quad 1{,}678 \text{ ppm by weight}$$

10. The results of the size fraction technique can be compared to the single bed particle size analysis presented in part (a) of this example for a discharge of 410 cfs. The single size technique produced a bed-material discharge of 0.21 cfs and a concentration of 1,355 ppm, while the size fraction analysis gave a discharge of 0.26 cfs and a concentration of 1,678 ppm. The calculation by size fraction is summarized in Table 5.11. Additionally, calculations for two other discharges are given and illustrate the relationships between Q_s and C as discharge increases, particularly the leveling off of concentration.

Table 5.10. Total Sediment Transport Rate for Colby Example.

Size Fraction	Percent of Total Sample	Unit Transport Rate	Actual Bed Material Discharge
0.84 mm	$1.1 \times \frac{1}{100} \times$	8.61	= 0.09 tons/day/ft.
0.42	$25.8 \times \frac{1}{100} \times$	11.48	= 3.15
0.21	$62.4 \times \frac{1}{100} \times$	19.04	= 11.88
0.10	$10.7 \times \frac{1}{100} \times$	26.73	= 2.86
	q_{s_t}	Total Unit Discharge For Q = 410 cfs	17.98 tons/day/ft.

Table 5.11. Bed-Material Discharge Calculations for Colby Method Example Using Sediment Size Fractions.

Q (cfs)	D_g (mm)	i_b (% of Total Sample Weight)	Y_h (ft)	V (fps)	b (ft)	q_{s_i} $\frac{\text{tons/day}}{\text{ft}}$	K_1	K_2	K_3	q_s	$\frac{i_b q_s}{100}$ tons/day/ft	q_{s_t}	Q_s tons/day	Q_s (cfs)	C (ppm)
410	0.84	1.1	1.36	2.93	103	8.5	0.92	1.20	12	8.61	0.09	17.98	1,851.9	0.26	1,678
	0.42	25.8				11.3			80	12.19	3.15				
	0.21	62.4				17.2			100	19.04	11.88				
	0.10	10.7				25.1			60	26.73	2.86				
2,820	0.84	1.1	2.50	6.63	170	99.6	0.91	1.22	12	100.88	1.11	161.82	27,509.4	3.86	3,614
	0.42	25.8				115.4			80	125.61	32.41				
	0.21	62.4				152.0			100	168.77	105.3				
	0.10	10.7				201.4			60	214.69	23.0				
9,620	0.84	1.1	4.14	9.92	234	238.9	0.90	1.25	12	242.49	2.67	425.59	99,588.1	13.97	3,834
	0.42	25.8				263.5			80	289.89	74.79				
	0.21	62.4				401.4			100	451.52	281.75				
	0.10	10.7				577.1			60	620.34	66.38				

5.3.7 Equilibrium Slope

Discussion - The equilibrium slope is defined as the slope at which the channel sediment transporting capacity is equal to the incoming sediment supply. Mathematically, this concept is expressed as

$$Q_{s_{in}} = Q_{s_{out}} \tag{5.10}$$

where $Q_{s_{in}}$ is the supply rate of sediment into the channel and $Q_{s_{out}}$ is the sediment transport rate out of the channel. When this relationship is satisfied, the channel neither aggrades or degrades, i.e., it is in equilibrium. If the sediment transport rate in a given reach is less than sediment supply, the slope of that reach will have to increase to achieve equilibrium conditions. Conversely, if the transport rate is greater than supply, the slope will need to decrease.

Application - The equilibrium slope methodology is utilized to evaluate long-term channel response (aggradation/degradation), specifically, the slope the channel ultimately wants to achieve. Short-term response during a single flood (general scour/deposition) must be evaluated by other methods (see Section 5.3.8). An equilibrium slope analysis should begin with a study of historic bed profiles through the project area. If trends towards aggradation or degradation can be detected, they should be traced to a cause. Cases may arise in a relatively undisturbed watershed that show no significant change in bed profiles over long periods of time. The length of time necessary to establish stability in bed profiles is hydrologically dependent (i.e., a function of historical climatology and hydrology). However, in any case, the longer the record of available data, generally the more confident one can be in determining the stability of the bed. Watersheds that are considered in equilibrium may not require equilibrium slope analysis unless the proposed flood plain improvements alter the sediment supply or transport capacity.

If historic bed profiles or field inspections indicate the system is attempting to adjust to an equilibrium condition, an analysis should be completed to determine what equilibrium condition is being sought and how any proposed flood plain improvements might cause an alteration in the equilibrium adjustment. The results of this analysis can then be incorporated into the project design.

Since the analysis is utilized to evaluate long-term conditions, the appropriate discharge for calculation purposes is the dominant discharge, which is that discharge predominantly responsible for channel characteristics. The dominant discharge is typically between a two- and a five-year event for perennial channels, and a five- and a ten-year event for ephemeral channels. Often the dominant discharge is equal to the bankfull discharge. Since equilibrium slope calculations do not have much physical significance or importance in the overbank area, bankfull discharge can be considered an upper limit for equilibrium slope calculations.

The first and perhaps most critical step in an equilibrium slope analysis is to determine the sediment supply from the upstream reach for the dominant discharge. In the absence of actual sediment supply data (i.e. measured data or analytically calculated watershed sediment yield), the sediment supply is most often evaluated from computation of the transport capacity (see Section 5.3.6) of the upstream reach, under the assumption that it is in equilibrium. For natural, undisturbed channels and/or watersheds, this is a reasonable assumption that can often be verified through examination of historical data (such as profile analysis or aerial photographs). However, for disturbed channels, e.g. in an urbanizing area, calculation of sediment supply is more complicated. After urbanization, the transport capacity of the selected supply reach does not necessarily reflect sediment supply. Since urbanization generally reduces sediment supply, the calculated transport capacity will typically be larger than the actual sediment supply. Additionally, if channelization has occurred, the transport capacity of the existing channel may not be similar to that for the channel that existed in the natural, undisturbed watershed. Therefore, to properly establish the sediment supply to which the channel is adjusting, it may be necessary to look at historical conditions to estimate the natural channel characteristics. The calculated transport capacity of this channel is then reduced to reflect the effects of urbanization. The calculation is obviously subjective and relies on extensive engineering judgment and experience.

After establishing the upstream sediment supply rate, the transport capacity of the study reach is evaluated. The sediment transport capacity of the study reach (or each subreach therein) is computed on the basis of the same water discharge (i.e., dominant discharge) that was used for the assumed equilibrium sediment supply reach. If the calculated transport capacity does not equal the supply, the slope of the study reach is adjusted and the transport capacity re-evaluated. This procedure is continued until the resulting sediment transport capacity equals the incoming supply, at which point the equilibrium slope will have been found. The equilibrium slope can be calculated for any number of reaches below the supply reach, not just the reach immediately below. When conducting this calculation, it is important to realize that the appropriate sediment supply, or inflow, to any subsequent reach is always the value computed from the supply reach.

An expedient way of determining hydraulic conditions necessary for evaluation of sediment transport capacity is to assume that uniform flow prevails. Manning's equation can then be used to calculate velocity, depth and top width for a given channel slope. This can be done for any channel shape by trial and error and can be adapted easily to hand-held programmable calculators. However, if significant backwater effects exist from a bridge or reservoir, the hydraulic conditions should be computed assuming gradually varied flow.

The selection of the proper channel geometry is important in equilibrium slope analysis. The sediment transport is proportional to some power of velocity (usually between 3.5 and 4.5 for sand bed channels) and is directly proportional to the flow width. This makes the equilibrium slope very sensitive to these parameters. The accurate determination of area, wetted perimeter, and top width as a function of depth are easy to develop and are usually sufficiently accurate below the bankfull level. Using power relationships, normal depth can be determined directly. Developed channel sections are usually trapezoidal and hydraulic conditions can be determined using hand-held programmable calculators.

When assumption of a wide channel is reasonable (i.e., width-to-depth ratio greater than 10), calculation of the equilibrium slope is simplified to a simple function of unit discharge. The equation is

$$S = \left(\frac{a}{q_s}\right)^{\frac{10}{3(b-c)}} q^{\frac{2(2c+3b)}{3(b-c)}} \left(\frac{n}{1.49}\right)^2 \qquad (5.11)$$

where a, b and c are the coefficients of the power relationship describing sediment transport (i.e., $q_s = a\,Y_h^b\,V^c$, see Tables 5.6a and 5.6b), n is the Manning n, S is the slope in ft/ft, q is the unit water discharge for the reach under consideration, and q_s is the unit sediment discharge for the supply reach. The derivation of this equation is provided by Simons, Li & Associates, Inc. (1982); however, the form of this equation is slightly different due to the definition of the exponents in the power relationship describing sediment transport. Furthermore, Equation 5.11 should only be used if the restrictions listed in Table 5.7 for Equation 5.8a (as used with Tables 5.6a and 5.6b) are met. Otherwise, a set of regression coefficients specific to the site under investigation should be developed.

For calculation of equilibrium slope in several reaches below the supply reach, the calculation simplifies even further if Manning's n, channel geometry and total discharge (Q) are the same in each reach. For this case, the only variable is bed slope, S_{ex}. This condition typically exists for channelized conditions where channel geometry is constant and there is no tributary inflow. The equation is

$$S = S_{ex} \left(\frac{Q_s \text{ supply}}{Q_s \text{ capacity}}\right)^{\left(\frac{2}{c-x}\right)} \qquad (5.12)$$

where $x = (3/5)\,(2/3\,c + b)$ and S_{ex} is the existing channel slope (ft/ft) in a given reach. From this equation it can be qualitatively established that the equilibrium slope will be less than the existing slope when sediment supply is less than transport capacity, i.e., an equilibrium slope less than the existing slope indicates a degradational condition.

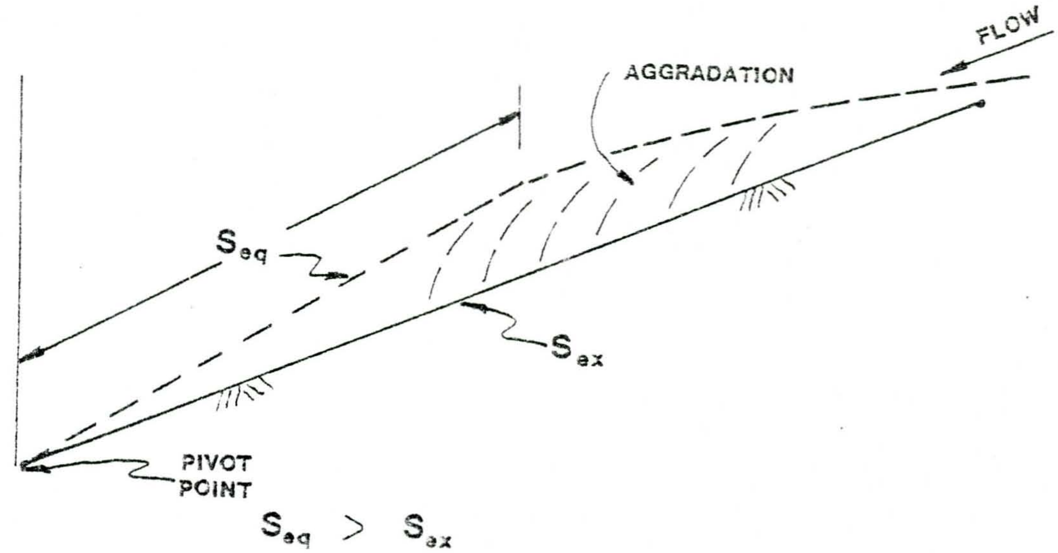
Results of equilibrium slope calculations are used to predict long-term changes to the bed profile of a river system. These changes normally will not occur as the result of a single flood. Usually, equilibrium slope conditions will evolve in response to the occurrence of many floods over a period of time. There is no accurate way to predict how long it will take such slope adjustments to occur. Large-scale, man-made changes to a river system may induce a complete equilibrium response within 10 to 100 years or even less, while natural changes on an undisturbed river may require a much longer time frame, perhaps on the order of 100 to 1,000 years.

A further complicating factor in the application of equilibrium slope calculations focuses on the location of a point from which the computed equilibrium slope can pivot. If natural geologic controls such as rock outcroppings or man-made grade control structures are present, these features can serve as pivot points. For a given river reach with such controls, the slope adjustment will always pivot about the downstream control point, such that if the equilibrium slope is less than the existing slope, degradation will occur, while if the inverse is true, aggradation will occur. Figure 5.19 illustrates how this will occur for the two cases of equilibrium slope being both greater than and less than the existing bed slope.

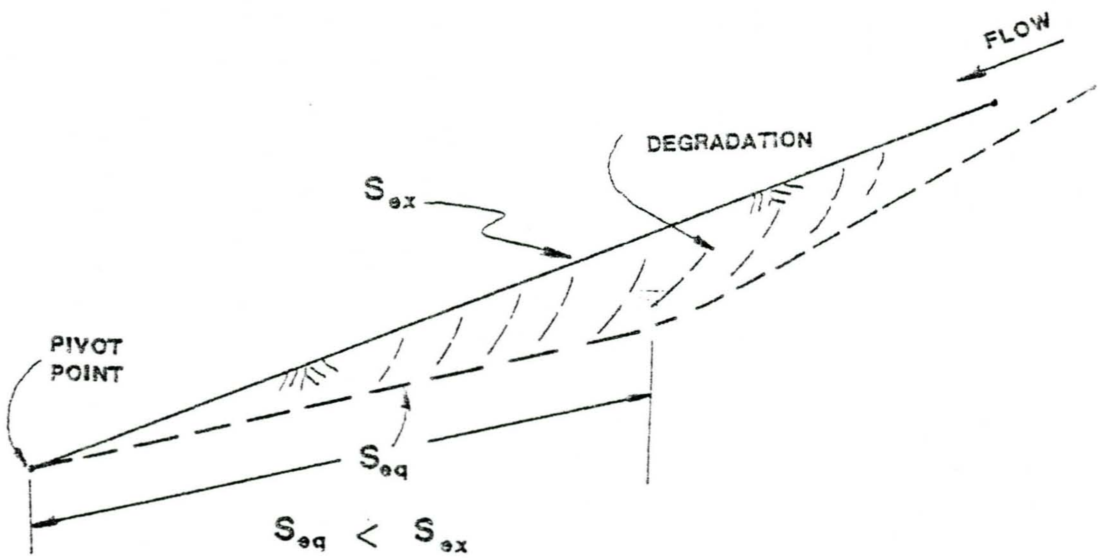
Identification of pivot points is critical to equilibrium slope analysis and relies heavily on engineering judgment and interpretation. For example, at large horizontal distances from a pivot point, the vertical distance between the existing bed slope and the equilibrium slope may become unrealistic. In these cases the engineer must re-evaluate the selection of pivot points to insure that no potential control points have been missed. If no control points can be located, the amount of long-term degradation may be controlled by the channel bank height. Unless a channel is formed through rock or strongly cemented materials, there is usually a maximum vertical height at which a channel bank will no longer be stable. When this limit is reached, bank sloughing will begin to occur which causes the channel to adjust horizontally rather than through continued vertical cutting. As the channel widens, the velocity of flow will decrease, resulting in a decrease in sediment transport capacity. This type of channel widening will continue until the transport capacity is brought into equilibrium with the sediment supply to the reach.

In addition to stable bank heights being a potential control for the equilibrium slope, a check should also be made to determine if channel armoring will be a factor in limiting the amount of degradation to a value less than that predicted by the equilibrium slope analysis. This may reveal that armoring will arrest the vertical channel movement before the predicted equilibrium slope can be attained.

Due to the complex interaction of variables that determine long-term aggradation/degradation and the simplifying assumptions that must be made in analyzing such long-term changes, the numerical results from an equilibrium slope analysis must be carefully evaluated to ensure they are reasonable. Overall, the results of this type of analysis can be very subjective and in many cases may only be useful in a qualitative sense rather than quantitative.



In this case, the sediment supply exceeds the sediment transport capacity of the reach. Under this condition, the bed slope must increase in order to increase the transport rate to match the supply rate. The initial excess of sediment supply will cause aggradation at the upstream end of the reach until the downstream portion of the bed slope is steep enough to transport all the incoming sediment.



In this case, the incoming sediment supply is less than the sediment transport capacity of the reach. This sediment deficit will be satisfied by a removal of bed material through the reach until the bed slope is flattened enough to reduce the transport capacity to the point that it matches the incoming sediment supply.

Figure 5.19 Relationship between equilibrium slope and channel bed controls.

A summary of the equilibrium slope procedure is presented as follows:

1. Select upstream equilibrium supply reach and obtain the following data:

   a. channel geometry
   b. channel slope
   c. sediment size distribution
   d. channel resistance (Manning's n)

2. Determine dominant discharge.

3. Divide the segment of the river system under analysis into separate reaches of similar hydraulic characteristics and identify control points.

4. Obtain the same information as in Step 1 for each downstream reach that is to be analyzed.

5. Select an appropriate sediment transport equation (i.e., from Table 5.5 if applicable).

6. Establish the sediment supply provided by the upstream supply reach. This rate will be the sediment supply for all downstream reaches unless significant tributary flow is encountered downstream of the supply reach. If equilibrium conditions can be assumed in the supply reach, the sediment supply will equal the transport capacity of the supply reach.

7. Compute the sediment transport rate for each of the downstream reaches by varying the slope through each reach until a transport rate is found which matches the sediment supply. This establishes an equilibrium slope for each reach.

8. Pivot equilibrium slopes about control points (if any were identified) to determine long-term bed adjustment.

9. Check any degradation dimensions determined from Step 8 to see if the maximum stable bank height or armoring will control the amount of bed adjustment possible.

Example: Prior to the establishment of a strict flood plain management program, residential development was allowed to encroach into the flood plain of a desert wash located in a rural area of Arizona.

In order to resolve the flooding problem for this community, a levee/channelization project has been proposed. The channel improvements will consist of clearing vegetation from the exiting channel in order to lower the n value to 0.025 as well as widening the channel to 200 ft.

An equilibrium slope analysis is to be performed to determine the long-term aggradation/degradation that may be induced by these channel improvements. The results of this analysis will be incorporated into the design of the bank stabilization system for the proposed levee.

1. A field inspection of the site indicated the wash had very uniform characteristics both up and downstream of the reach for proposed channel improvements. Accordingly, the following channel parameters were considered representative of both the upstream sediment supply reach and the existing downstream reach which is to undergo channelization.

   a. Existing channel bottom width is approximately 150 ft.
   b. Existing bed slope = 0.007 ft/ft.
   c. D_{50} (bed material) = 0.5 mm.
   c. G (gradation coefficient) = 2.0.
   e. Average channel depth is about 4 ft.
   f. Existing channel n value was estimated at 0.04.

2. A hydrologic analysis of the upstream drainage area indicated the 10-year event has a peak discharge of 3,000 cfs. When Manning's Equation was applied to the supply section channel geometry with a discharge of 3,000 cfs, the depth of flow was found to be about 3 ft. Since this is within a foot of being bankfull, 3,000 cfs was chosen as the dominant discharge.

3. The proposed channelization only extends along a 1,500-foot reach of the wash so the equilibrium slope analysis will be confined to this length. Since the existing channel conditions and proposed channel improvements are uniform throughout this length, only one downstream reach will be used for the analysis.

   An "at-grade" soil cement road crossing was discovered near the downstream end of the study reach. It was assumed this crossing would withstand the 100-year design flood, therefore it was to be a stable control point for the equilibrium slope analysis.

4. The existing channel conditions through the study reach are listed in Step 1.

5. Referring to Tables 5.5 and 5.6a, it was determined that a power relationship of the form $q_s = a\,Y_h^b\,V^c$ would be the most efficient way to analyze the sediment supply and transport capacities through the reach.

   From Table 5.6a, the following coefficients and exponents were obtained using the data from Step 1:

   $$a = 1.48 \times 10^{-5}$$

   $$b = 0.0501$$

   $$c = 4.31$$

   Accordingly, the sediment transport equation is:

   $$q_s = 1.48 \times 10^{-5}\,Y_h^{0.0501}\,V^{4.31}$$

$G = \left(\frac{d_{84}}{d_{50}} + \frac{d_{50}}{d_{16}}\right)$

(gradation coefficient)

6. Manning's Equation was used to compute the velocity and hydraulic depth for the upstream sediment supply section. The following parameters from Step 1 were used:

$n = 0.04$

$b = 150$ ft

side slopes = 1:1

bed slope = 0.007

$Q = 3{,}000$ cfs

The calculation yields $V = 6.40$ fps
$y_h = 3.00$ ft

The sediment supply is now calculated as the transport capacity of the upstream supply section:

$$q_s = 1.48 \times 10^{-5} \, (3.00)^{0.0501} \, (6.40)^{4.31}$$

$$q_s = 0.047 \text{ cfs/ft}$$

eq. = 5.8A
= 5.8B

page 5.54 + 5.55

$Q_s = q_s W$

$q_s = a d^b V^c$

$a = 1.48 \times 10^{-5}$

$b = 0.0501$

$c = 9.31$

Total sediment supply = q_s x average flow width

$$Q_{s_{in}} = (0.047)(153)$$

lw

$$Q_{s_{in}} = 7.19 \text{ cfs}$$

7. The transport capacity of the improved channel reach will now be computed with different bed slopes until one is found which will yield a transport capacity equal to the incoming supply rate. The proposed channel parameters are as follows:

$n = 0.025$

$b = 200$ ft

side slopes = 1:1

Using the dominant discharge of 3,000 cfs, Manning's Equation is first used to calculate the velocity and hydraulic depth for substitution into the sediment transport equation. The calculations, which employ a trial-and-error sequence, are summarized in the following table:

Slope (ft/ft)	Y_h (ft)	V (fps)	q_s (cfs/ft)	Flow Width (ft)	Q_s (cfs)
0.007	1.92	7.65	0.098	204	19.99
0.0025	2.61	5.60	0.026	205	5.33
0.003	2.47	5.91	0.033	205	6.77
0.00315	2.44	6.00	0.035	205	7.18

A slope of 0.00315 yields a transport rate of 7.18 cfs which is approximately equal to the incoming supply rate of 7.19 cfs. Accordingly, this can be taken as the equilibrium slope.

8. By pivoting the equilibrium slope around the downstream control point (soil cement road crossing), it is determined that up to 5.8 feet of degradation is possible at the upstream end of the channelized reach (see Figure 5.20).

   This long-term degradation should be added to any other anticipated erosion or scour to get a total toedown depth necessary to protect the levee from undermining. Additionally, this long-term degradation may initiate a headcut upstream of the channelized reach. For this reason, consideration should be given to placing a grade-control/drop structure at the upstream end of the channel.

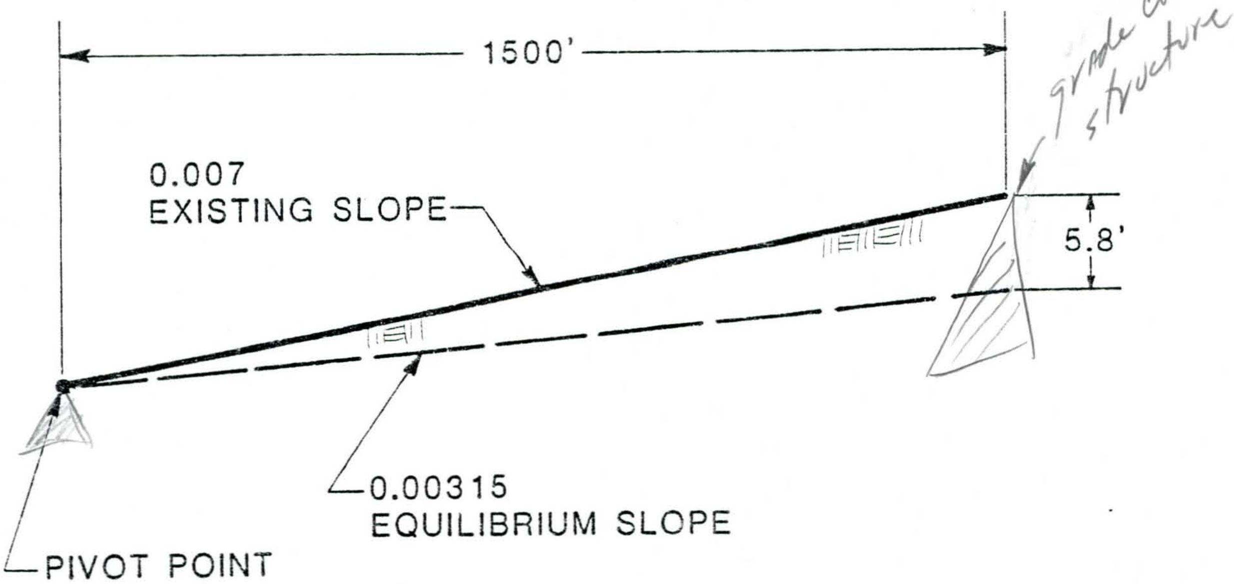
9. Backhoe pits were excavated to a depth of 8 feet at two locations in the existing channel. No bed material was encountered of a size large enough to form an armor layer. Accordingly, armoring will not limit the predicted amount of long-term degradation.

5.3.8 Sediment Continuity Analysis

Discussion - The sediment continuity principle applied to a given channel reach states that the sediment inflow minus the sediment outflow equals the time rate of change in sediment storage. Mathematically, this can be presented as

$$Q_{s_{in}} - Q_{s_{out}} = \frac{dVol}{dt} \qquad (5.\ 13)$$

$Q_s = q_s \frac{W}{3}$



NOT TO SCALE

Figure 5.20. Existing and Equilibrium Slope Profiles for Example Problem.

For a given discharge acting for a given time, the volume of sediment deposited or eroded in a channel reach is simply the difference between the upstream sediment supply rate ($Q_{s_{in}}$) and the channel sediment transport rate ($Q_{s_{out}}$). If the supply rate is greater than the transport rate, the reach is depositional, while if transport is greater than supply, general scour will occur. (The basic principle of the equilibrium slope analysis is no change in volume, i.e., $dVol/dt = 0$ in Equation 5.10.)

The sediment continuity principle can be applied to analyze conditions during a single discharge (e.g. peak discharge of a 100-year flood) or during the hydrograph of a single flood. Either of these applications provides information on the short-term erosion/sedimentation occurring on a reach-by-reach basis.

Application - The first step in sediment continuity analysis is delineation of the study reach into a number of subreaches. Delineation of subreaches is based on (1) physical characteristics of the channel, such as top width and slope, (2) hydraulic parameters, such as depth and, particularly, velocity, (3) bed-material sediment characteristics, (4) areas of particular interest to study objectives, such as bridges or locations of proposed channel improvements, and (5) the desire to maintain reach lengths as uniform as possible throughout the system. Items 1, 2 and 3 are generally selected to provide consistency within the subreach, so that representative average conditions may be determined. For example, consistency in top width and channel length influence the utilization of sediment continuity results in evaluation of vertical channel response. As discussed in Section 5.3.9, erosion/sedimentation volumes from sediment continuity are often linearly distributed through the reach to determine vertical channel adjustments. Therefore, if an upstream reach length is 2,500 feet and the downstream reach is only 1,500 feet, the vertical adjustment of the channel bed responding to the imbalance in sediment supply and transport capacity between reaches will be much different from that had the downstream reach been dimensioned as 2,500 feet.

Furthermore, uniform channel lengths are important in maintaining the integrity of sediment continuity analysis. Sediment continuity analysis does not address the time or channel length that it takes for the difference between sediment supply and transport rate to achieve a balance. It is assumed that a balance will be achieved within the reach regardless of its length.

This is not necessarily correct. For example, in a very short depositional reach, particle settling times may not permit the calculated sedimentation to occur. For this reason it is recommended that reach lengths be kept as uniform as possible to avoid the introduction of an additional variable to the analysis that could bias or otherwise create unrealistic results.

After subreach delineation, characteristic geometric and hydraulic information must be developed for each subreach for the discharge(s) under consideration. This information may be computed manually through uniform flow or gradually varied flow calculations, or through computer programs such as HEC-2. For example, if HEC-2 output data are available, the required velocity, depth and top width data at various cross sections within the study reach will be provided. Within a given subreach these data can be averaged to define values representative of conditions in that reach for the given discharge.

After establishing representative hydraulic characteristics in each subreach for the given discharge(s), the sediment transport capacity of each subreach is calculated using an appropriate method (see Section 5.3.6). The sediment continuity principle is then applied by comparing transport capacity on a reach-by-reach basis, under the assumption that the sediment supply to any given subreach is equal to the transport capacity of the adjacent upstream reach. The comparison begins at the upstream end of the study reach by designating the first subreach as a supply reach, which initiates the calculation in Subreach 2.

Application of sediment continuity analysis to a flood hydrograph requires discretizing the hydrograph into a series of discrete discharges, as described in Section 3.5. The reach-by-reach comparison is then completed for each discharge and the total volume of erosion or deposition occurring in any given reach during the flood is computed as $VOL_i = \Sigma\,(Q_s\,\Delta T)$ where VOL_i is the net volume change during the flood for subreach i, Q_s is the excess transport capacity or supply in subreach i for the given discharge (i.e. supply minus transport capacity), and ΔT is the time interval corresponding to that discharge from the discretized hydrograph. It is important to note that this procedure yields a net volume of erosion or deposition that occurs in response to passage of the complete flood hydrograph, i.e., we are looking at the <u>net change</u> in volume at the <u>end</u> of the hydrograph. There may be time intervals <u>within</u> the hydrograph where the volume change for that specific interval would exceed the net volume change for the entire hydrograph. This is important to remember when using the sediment continuity procedure to compute general scour, since an analysis of net changes at the end of a hydrograph may under-estimate a transitory scour condition that might occur during a critical time interval within the hydrograph.

To expedite the calculation procedure when evaluating several hydrographs, the following analysis procedure is suggested. First, identify five to ten discharges adequate to span the discharge range of all hydrographs. After computing the average hydraulic characteristics in each subreach for each discharge, compute the corresponding sediment transport capacities. Then, for each subreach, develop a relationship of the form $Q_s = a\,Q^b$ where Q_s is the sediment transport capacity in cfs, Q is the water discharge in cfs, and a and b are regression coefficients. The analysis of the discretized hydrographs then proceeds as outlined above, with the sediment transport capacity for any given discharge in any given reach obtained by using the appropriate regression relationship.

It is important to note that the sediment continuity analysis described herein is based on the assumption of rigid-boundary conditions. For example, during evaluation of a flood hydrograph, the channel geometry is assumed to remain unchanged throughout the flood. A more accurate analysis technique is to update the channel cross sections for each discharge level of the flood to account for the computed erosion/sedimentation changes. This concept is referred to as quasi-dynamic routing, and is the basis of Level III analysis where computer models such as HEC-2SR are applied. However, for many practical engineering analysis and design problems the application of the sediment continuity procedure is adequate and more cost efficient.

Example - As part of a channel stability study of the Agua Fria River near Phoenix, Arizona, a sediment continuity analysis was conducted for the peak discharge of the 10- and 100-year floods. This application of the sediment continuity procedure provided insight to the short-term response of the channel. The approximate 30-mile study reach was divided into 10 reaches. Average hydraulic and geometric characteristics for the 10 reaches were established from HEC-2 analysis. For the 10-year flood peak the main channel velocities ranged from 5 to 7 feet per second (fps) and for the 100-year, 7 to 10 fps. Sediment transport capacity was estimated by the Meyer-Peter, Muller bed-load equation in combination with the Einstein suspended load procedure. Reach 1 was utilized as the supply reach from which the sediment continuity calculation began. Table 5.12 gives the results from the analysis and indicates the general scour/deposition condition of each reach.

5.3.9 Quantification of Vertical and Horizontal Channel Response

Discussion - Sediment continuity results provide the rate and/or volume of erosion/sedimentation expected in each subreach. More meaningful results are obtained when these values are converted to represent vertical and/or horizontal changes in each subreach. Using the definitions established in Section 5.1.2, sediment continuity results from a single flood would be useful in quantifying general scour/deposition or short-term lateral migration.

Application - In the absence of significant controls the erosion/sedimentation amounts can be assumed uniformly distributed in the streamwise direction for any given subreach. If the cross-streamwise distribution is also assumed uniform, then with knowledge of the reach length and by assuming a representative channel width, typically top width, the uniform depth of vertical adjustment can be evaluated. As an alternative, the cross-streamwise distribution can be done according to flow conveyance; however, this can become a tedious calculation by hand calculator.

In making the distribution, the computed sediment volumes must be corrected for porosity. The sediment transport equations used in the sediment continuity analysis give answers in unbulked volumes per unit time. A porosity factor must be applied to these values to accurately represent the volume changes that will occur in the channel bed. For sand-bed channels, a typical porosity of $n = 0.4$ can be assumed. The unbulked sediment volumes computed by the transport equations are then corrected as follows:

$$V_t = \frac{V_s}{1-n} \tag{5.14}$$

where V_t is the bulked sediment volume, V_s is the sediment volume computed by transport equations, and n is the porosity.

Evaluation of lateral migration amounts is more difficult and not as subjective to a rigorous analysis procedure as are vertical adjustments. The two basic mechanisms of lateral migration can be related to erosion/sedimentation supply

Table 5.12. Short-Term General Scour/Deposition Response.

of sediment

Reach	10-Year Q_s (cfs)	10-Year ΔQ_s (cfs)	10-Year Response	100-Year Q_s (cfs)	100-Year ΔQ_s (cfs)	100-Year Response
1	105			379		
2	112	-7	Near Equilibrium	254	+125	Deposition
3	187	-75	Scour	446	-192	Scour
4	122	+65	Deposition	541	-95	Scour
5	158	-36	Scour	581	-40	Scour
6	131	+27	Deposition	538	+43	Deposition
7	168	-37	Scour	676	-138	Scour
8	81	+87	Deposition	465	+211	Deposition
9	80	+1	Equilibrium	446	+19	Near Equilibrium
10	106	-26	Scour	492	-46	Scour

Q_s is sediment transport rate, ΔQ_s is general scour (-)/deposition (+) rates of the flood peak.

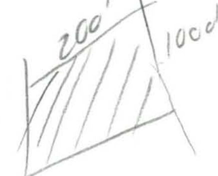
assume 30 min

$-75 \text{ cfs} \cdot \Delta t = \Delta V_s$

$-75(30)60 = 135{,}000 \text{ cfs}$

$\frac{\Delta V_s}{1-n} = \frac{-135{,}000}{.6} = 225{,}000 \text{ ft.}$

V_b



$\frac{V_b}{wL_k} = -1.13 \text{ ft}$

trends in the channel. The first mechanism, associated with channel reaches of large w/d (width/depth) ratio where significant sedimentation is occurring, is that which promotes bank instability and lateral migration as a result of increased velocities and shear stresses along the banks as the local energy gradient increases. The second mechanism, associated with channel reaches of small w/d ratio, typically in an erosional mode, is that which causes increased bank instability from bank failures as a result of development of a narrow, deep channel with steep banks.

There are several variations of the first mechanism involving channels where significant sedimentation is occurring. If sedimentation occurs as isolated sand and gravel bars, the local energy gradient increases due to higher flow velocities that result from a reduction in effective channel area. Additionally, relatively stable sand and gravel bar deposits deflect the flow towards the more erodible banklines. Consequently, severe localized bank failures may occur. However, if deposition occurs more uniformly across the channel, the local energy gradient downstream of the deposition increases due to higher velocities resulting from an increase in channel slope. The absence of current deflection and the more gradual increase in velocities results in less severe bank erosion, but erosion takes place over longer distances. Under either situation, quantifying lateral migration amounts from sediment continuity calculations is difficult. Generally, in these types of reaches the assessment of lateral migration potential must be made from qualitative analysis such as historical evidence, meander scars, meander width, geomorphic relationships, etc.

There are also variations of the second mechanism involving a typically erosional reach of the channel. The mode of bank failure as the channel deepens depends on bank material composition. In a channel with predominately clay banks, failure may be by sloughing due to undercutting by low-flow discharges. In a stratified bank with lenses of erodible material, enough of this material may be removed that the block of bank material above tilts downward, opening a vertical tension crack. Ultimately the bank fails in large blocks. Piping can also promote bank failure in a stratified bank. Quantifying lateral migration amounts for erosional reaches is easier than for reaches where sedimentation is occurring. The volume of erosion computed from sediment continuity analysis can be assumed to come entirely from the channel banks or can be distributed between bed and banks. However, since it is difficult to establish the distribution, and since the direction of lateral migration is not known with certainty, it is sometimes appropriate to assume the required volume first comes entirely from one bank and then the other. The exception to this procedure is when controls inhibit movement in a given direction. Additionally, it may be possible that the entire volume could come from a single location on one bank, for example, a short-radius bend in relatively erodible material.

In both mechanisms of lateral migration, development of saturated banks above the water line can increase bank erosion through local mass wasting. Saturated banks may develop during the rising stage of a flood, during which flow moves into the bank from the river, promoting increased bank stability, particularly in the saturated condition. Flow may also occur from the bank to the river due to a groundwater table that is higher than the river stage. This condition could develop during a wet period as water draining from the watershed saturates the flood plain to a level higher than normal.

Example - A preliminary design for a sewer line in the City of Globe, Arizona, proposed an alignment that followed the Pinal Creek channel for approximately 3 miles. To evaluate the adequacy of the proposed 6-foot burial, an erosion/sedimentation study was conducted. The study included application of the sediment continuity concept to estimate erosion/sedimentation volumes, and then the conversion of these volumes to general scour/deposition estimates.

The analysis was conducted for the 100-year flood (peak discharge 19,500 cfs). For simplicity, the hydrograph was discretized into three discharge levels: one peak and two medium flows. The study reach was divided into eight subreaches and, from a HEC-2 analysis, the average hydraulic conditions for each reach were determined. Sediment transport rates for each of the reaches for all three discharges were then calculated, and the sediment continuity equation (Equation 5.13) applied between reaches to estimate the erosion/sedimentation for each reach. The depth of general scour/deposition was then determined by uniformly distributing the calculated volumes in each reach. Table 5.13 presents the results and indicates as much as 9 feet of general scour is possible; therefore, the 6-foot burial depth was not con-

Table 5.13. Sediment Continuity Results. (100-Year Flood)

Reach	Average Potential Bed Level Change (feet)
1	+ 4.8
2	+ 8.3
3	- 9.4
4	+ 1.2
5	- 5.9
6	- 0.5
7	- 4.3
8	Supply Reach

sidered adequate unless some type of channelization or grade control was to be implemented in Reach 3.

5.3.10 Local Scour Concepts

Discussion - Local scour is observed whenever an abrupt change in the direction of flow occurs, such as at bridge piers or embankments. For example, local scour at bridge piers is a result of vortex systems developed at the pier. Local scour occurs when the capacity of the flow to remove or transport the bed materials is greater than the rate at which replacement material is supplied.

During a flood, an equilibrium condition between sediment supply and transport capacity at a scour hole may never become established. During the rising limb of the hydrograph scour occurs and endangers the hydraulic structure. After the peak has passed (during the falling limb), the scour hole refills as sediments drop out with the lower flows. Therefore, the critical time for structural stability during the storm is near the peak flow (see Figure 5.21). Soundings made of scour holes after the storm do not indicate the potentially dangerous situation that might have existed during the storm.

The depth of scour also varies with time depending upon the presence or absence of bed forms. The time required for dune or antidune motion is much larger than the time required for local scour. Thus, even with steady-state conditions, the depth of scour is likely to fluctuate with time when there are dunes or antidunes traveling on the channel bed. The depth of the scour hole is more variable with larger bed forms. When the crest of the dune or antidune reaches the local scour area, the transport rate into the hole increases, the scour hole fills and the scour depth temporarily decreases. When a trough approaches, there is a smaller sediment supply and the scour depth increases to try to re-establish equilibrium in sediment transport rates. A mean scour depth between these oscillations is referred to as equilibrium scour depth.

Application - A number of formulas are available for predicting local scour around bridge piers. Review of these formulas indicates that each is based on those factors that seem most important in evaluating local scour at bridge piers; however, most of these formulas are based primarily on model study data in sand-bed laboratory flumes with little or no field verification. Therefore, it is generally advisable to utilize several formulas to insure a

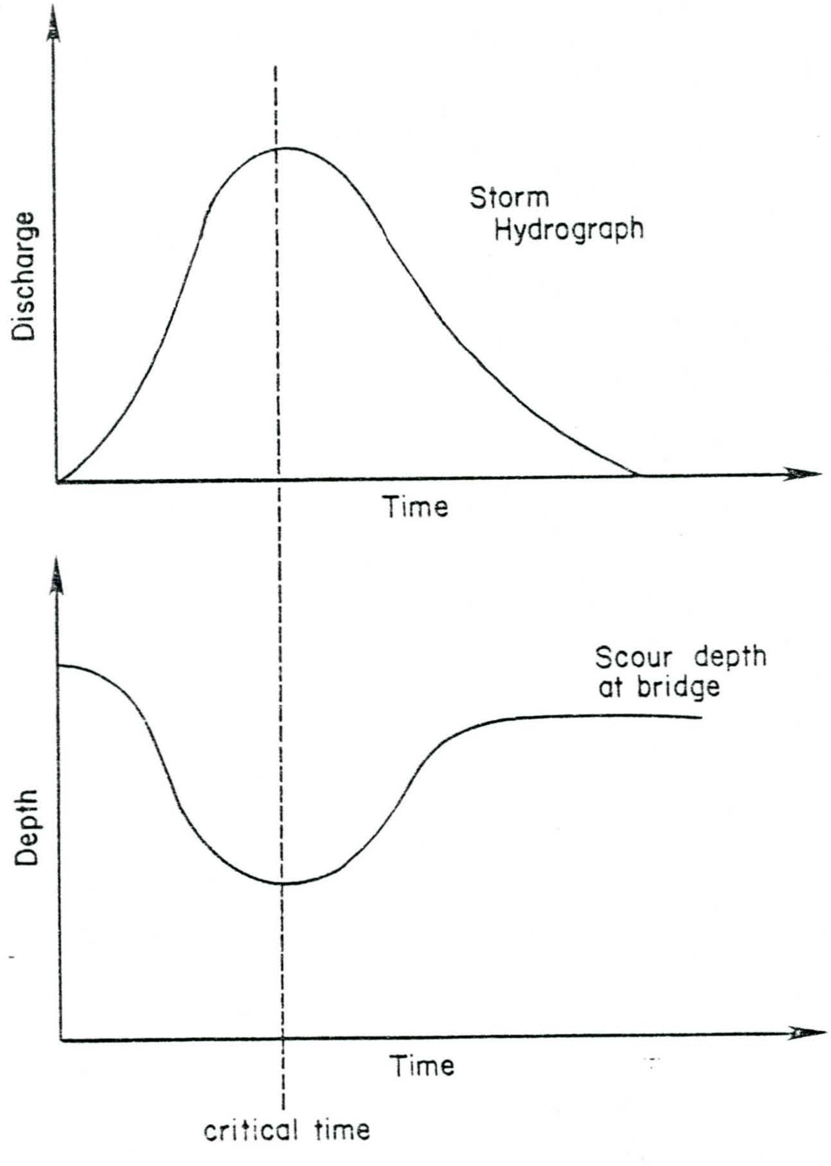


Figure 5.21. Temporal change of scour hole depth during a storm (typical).

reasonably accurate estimate. Several of these formulas have been found to be particularly successful based on previous experience. A relationship for square-nosed piers presented by Richardson, et al. (1975) is

$$\frac{\Delta Z_{\ell s}}{Y} = 2.2 \left(\frac{b_p}{Y}\right)^{0.65} Fr^{0.43} \tag{5.15a}$$

and for a group of circular cylinders

$$\frac{\Delta Z_{\ell s}}{Y} = 2.0 \left(\frac{b_p}{Y}\right)^{0.65} Fr^{0.43} \tag{5.15b}$$

where $\Delta Z_{\ell s}$ is the equilibrium depth of the scour hole, b_p is the pier width (normal to the flow direction), Y is the upstream depth of flow, and Fr is the upstream Froude number ($Fr = V/\sqrt{gY}$ with V the upstream velocity and g the acceleration of gravity).

The equations by Shen et al. (1966, 1969) for circular piers are

$$\Delta Z_{\ell s} = 0.00073\ R_p^{0.619} \tag{5.16a}$$

and

$$\frac{\Delta Z_{\ell s}}{b_p} = 11.0\ Fr_p^2 \qquad \text{for} \quad Fr_p < 0.2 \tag{5.16b}$$

$$\frac{\Delta Z_{\ell s}}{b_p} = 3.4\ Fr_p^{0.67} \qquad \text{for} \quad Fr_p \geq 0.2 \tag{5.16c}$$

respectively, where R_p is the pier Reynolds number ($V\ b_p/\nu$), V is the mean velocity of the undisturbed flow, b_p is the width of pier projected on a plane normal to the undisturbed flow, ν is the kinematic viscosity, and Fr_p (pier Froude number) is $V/\sqrt{gb_p}$.

The shape of the pier is a very significant parameter with respect to scour depth because it reflects the strength of the horseshoe vortex at the base of the pier. A blunt-nose pier causes the deepest scour. Streamlining the front end of the pier reduces the strength of the horseshoe vortex, thus reducing the scour. Streamlining the downstream end of piers reduces the strength of wake vortices. Common shapes of piers are shown in Figure 5.22. The scour depth generally decreases as a consequence of streamlining, while

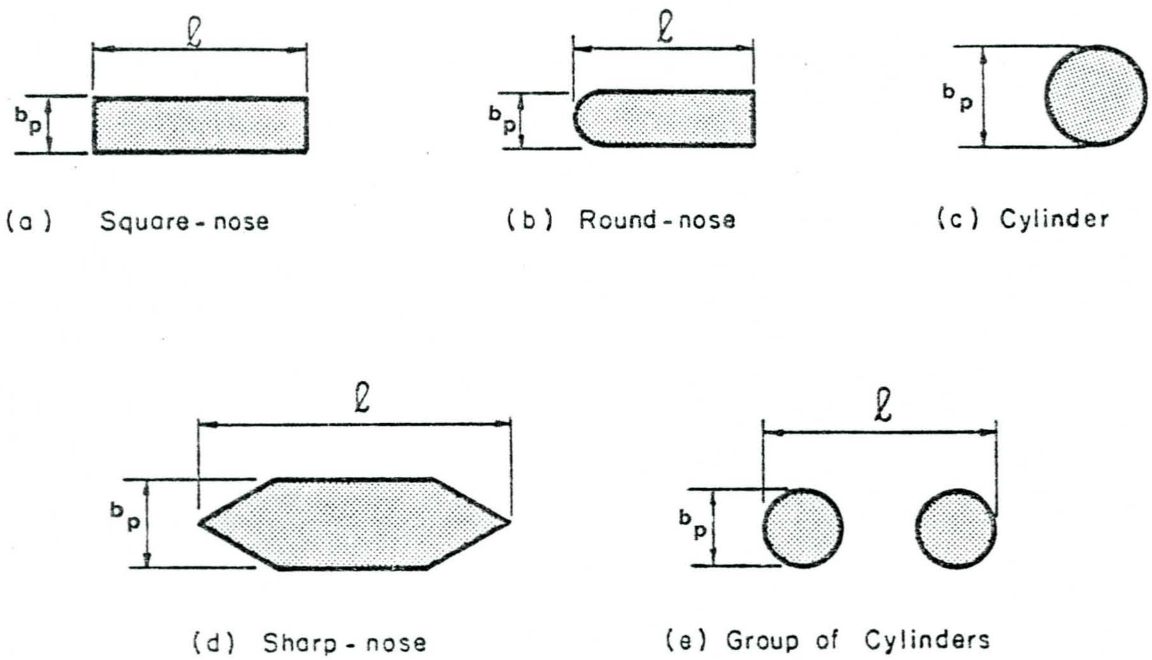


Figure 5.22. Common pier shapes.

skewed pier alignment (pier not parallel with flow direction) will create deeper scour holes. The reduction due to streamlining can be estimated from Table 5.14, while the increase due to skew can be determined from Figure 5.23 (Factor K_{aL}). As previously indicated, the maximum scour depth can be considerably greater than the equilibrium scour depth due to dune bed forms. Richardson, et al. (1975) suggest that scour depths can be up to 30 percent greater as a consequence of bed forms. Therefore, when dune or antidune bed forms are possible, a safety factor of 1.3 is recommended, unless the magnitude of the dune or antidune bed forms is calculated as a separate component.

Another important local scour zone at a bridge crossing occurs at the abutments. Detailed studies of scour around embankments have been made only in laboratories. For example, Liu, et al. (1961) investigated scour around vertical wall embankments for subcritical flow in a rectangular laboratory flume with sand-bed conditions and found

$$\frac{\Delta Z_{\ell s}}{Y} = 2.15 \left(\frac{a}{Y}\right)^{0.4} Fr^{0.33} \tag{5.17a}$$

where Y is the upstream normal flow depth, a is the embankment length (measured normal to the wall of the flume in the model studies), and Fr is the upstream Froude number (using the upstream normal flow depth as length dimension). Liu, et al. also presented limited data for spill-through embankments, where a spill-through embankment has sloping sides (i.e. the more commonly constructed earthen embankment). Analysis of the data presented suggests the equation

$$\frac{\Delta Z_{\ell s}}{Y} = 1.1 \left(\frac{a}{Y}\right)^{0.4} Fr^{0.33} \tag{5.17b}$$

Richardson et al. (1975) suggest that Equation 5.17b be applied only for embankments where a/y is less than 25. For embankments where a/y is greater than 25, the equation

$$\frac{\Delta Z_{\ell s}}{Y} = 4\ Fr^{0.33} \tag{5.18}$$

is recommended. This equation was developed from field measurement of embankment scour at rock dikes on the Mississippi River. It is worthwhile to note that embankment scour equations are also useful for estimating local scour at bank protection, spur dikes and jetties.

Table 5.14. Reduction Factors When Applying Formulas for Square Nose Piers to Other Shapes (assuming equal projected widths of piers).

Type of Pier	Reduction Factor
Square nose	1.0
Cylinder	0.9
Round nose	0.9
Sharp nose	0.8
Group of cylinders	0.9

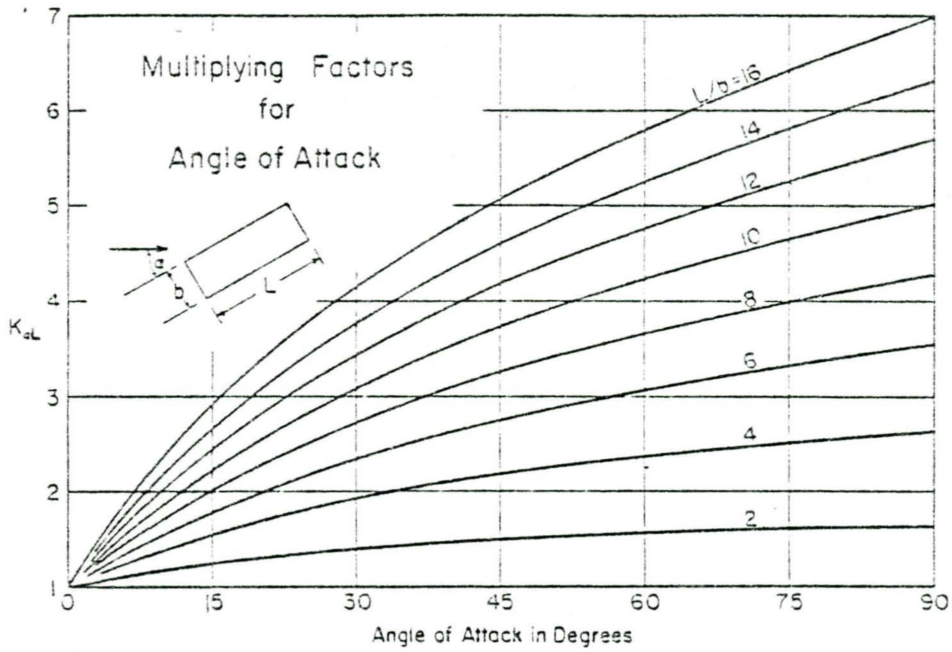


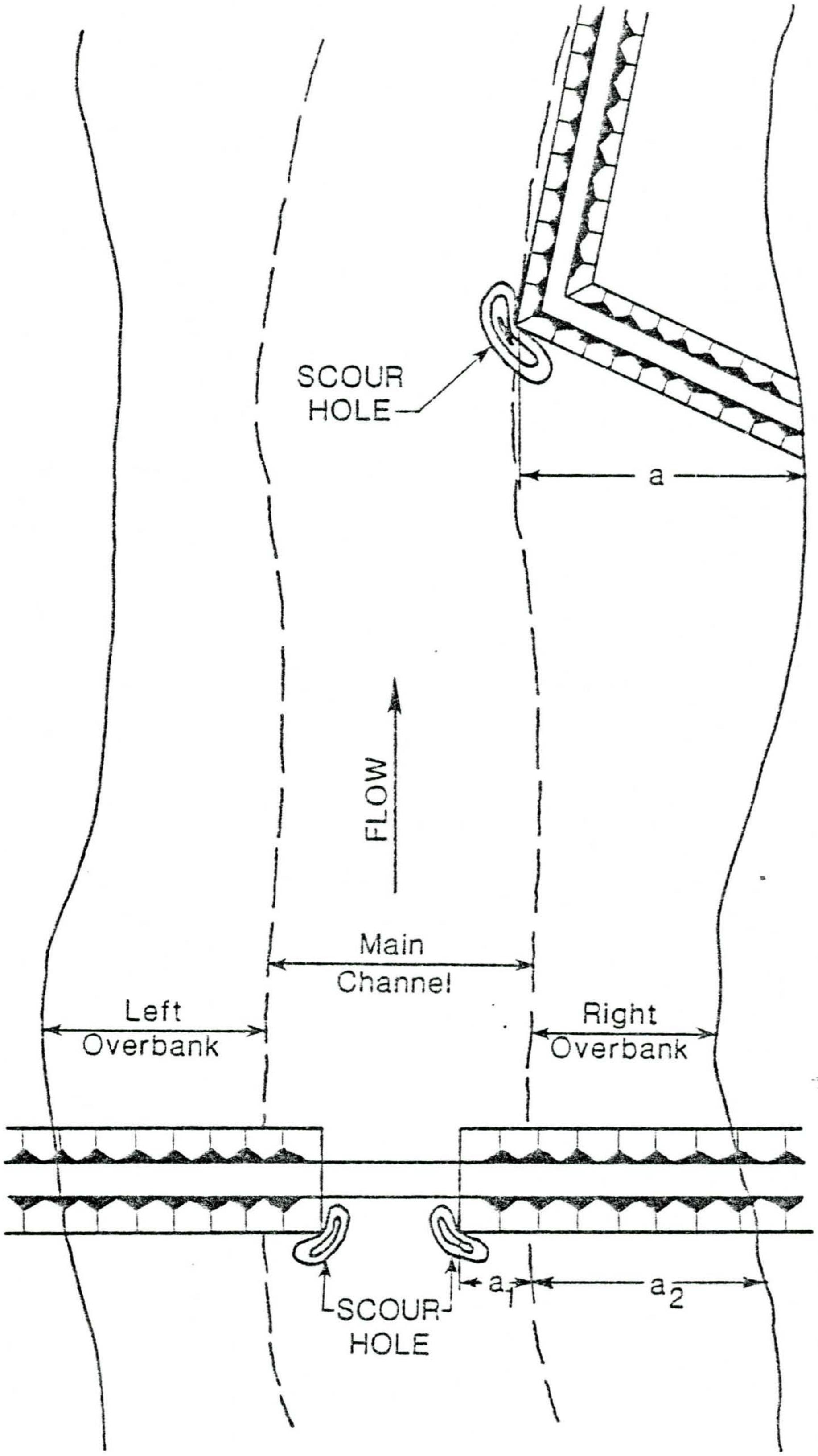
Figure 5.23. Scour increase factor, K_{aL}, with flow (from Lauren and Toch, 1956).

One of the difficulties in applying Equations 5.17b and 5.18 is definition of "embankment length." Model study investigations considered only short embankment lengths in smooth, rectangular flumes. In prototype situations, the embankments may span large distances across the overbank of a wide flood plain while stopping short of, or just slightly protruding into, the main channel. Due to the normally large difference in hydraulic characteristics between main channel and overbank flow, caution must be exercised in defining the embankment length for such cases. Figure 5.24 illustrates a recommended embankment length definition for different cases that may be encountered outside the realm of a rectangular laboratory flume. For Case 2 of Figure 5.24, the engineer should compute embankment scour using main channel hydraulics with the value of a_1, and compare this result to that obtained using overbank hydraulics with the value for a_2. The larger of these two scour depths would be the recommended design value. Due to the sensitivity of Equations 5.17a, 5.17b, and 5.18 to embankment length, engineering judgment should always be applied.

Another difficulty common to any scour calculation is the definition of the base level, and its relation to both flow depths and scour depths. In a nonprismatic natural channel, the upstream normal depth (Y) is generally defined by the hydraulic depth (Y_h) for purposes of scour calculations, while the computed scour amounts are referenced to the thalweg elevation. If dunes exist, the upstream normal depth would generally be referenced near the top of the dunes (in consideration of effective flow area), while the scour amounts should be referenced to the bottom of the dunes. In the presence of degradation and/or general scour, the ultimate bed invert elevation should first be established for these scour components, from which local scour depths are then referenced.

Once the scour depth is accurately established, the lateral extent of the scour hole is nearly always determinable from the depth of scour and the natural angle of repose of the bed material. A safety factor of 2 should be applied to the lateral scour hole dimension to account for nonuniform flow conditions. This can be accomplished by dividing the angle of repose by 2 and using the resulting angle to define the sides of the scour hole.

Example - The design of two bridge crossings on the Canada del Oro Wash near Tucson, Arizona, required the evaluation of local scour around the bridge



CASE 1
Overbank Levee

Upstream depth of flow, Y, and Froude number should be based on hydraulic conditions for right overbank flow.

CASE 2
Bridge Embankment

Upstream depth of flow, Y, and Froude number should be based on hydraulic conditions for main channel flow when using a_1 and overbank flow when using a_2. A comparison of scour calculations using these two definitions of embankment length is recommended.

Figure 5.24. Definition sketch of embankment length "a".

piers. Each bent consists of four piers, aligned parallel to the flow (i.e., no skew), and each pier was a concrete cylinder three feet in diameter. The design conditions are stated as follows:

Q_{100} = 33,000 cfs

Y (average depth of flow) = 6.0 feet

V (average velocity) = 18.2 fps

A review of historical photos taken during flood stage at other bridge locations on the Canada del Oro indicates that two additional feet of debris buildup beyond the normal pier width could be expected during a 100-year event. Accordingly, the effective pier width was set as follows:

b_p = pier diameter + 2 = 3 + 2 = 5

Local scour was computed with Equations 5.15b, 5.16a and 5.16b. The computations are shown as follows:

<u>Equation 5.15b</u>: $\Delta Z_{\ell s} = 2.0\ Y \left(\frac{b_p}{Y}\right)^{0.65} Fr^{0.43}$

Y and b_p are given above.

$$Fr = \frac{V}{\sqrt{gY}} = \frac{18.2}{\sqrt{32.2 \times 6.0}} = 1.31$$

Substituting in Equation 5.15b:

$$\Delta Z_{\ell s} = 2.0\ (6.0)\ \left(\frac{5}{6.0}\right)^{0.65} (1.31)^{0.43} = 12.0 \text{ feet}$$

<u>Equation 5.16a</u>: $\Delta Z_{\ell s} = 0.00073\ R_p^{0.619}$

where $R_p = \frac{Vb_p}{\nu}$

with an assumed water temperature of 70°F, $\nu = 1.059 \times 10^{-5}$ ft^2/sec,

therefore, $R_p = \frac{(18.2)(5)}{1.059 \times 10^{-5}} = 8.5 \times 10^6$

Substituting in Equation 5.16a,

$$\Delta Z_{\ell s} = 0.00073\ (8.5 \times 10^6)^{0.619} = 14.2 \text{ feet}$$

Equation 5.16b: The pier Froude number, Fr_p, must first be calculated to determine which form of Equation 5.16b should be used:

$$Fr_p = \frac{V}{\sqrt{g b_p}} = \frac{18.2}{\sqrt{32.2 \times 5}} = 1.43$$

Since $Fr_p > 0.2$, the following equation is used:

$$\Delta Z_{\ell s} = 3.4\ b_p\ Fr_p^{0.67}$$

$$= (3.4)(5)(1.43)^{0.67} = 21.6 \text{ feet}$$

A summary of the calculation is presented as follows:

	Eq. 5.15b	Eq. 5.16a	Eq. 5.16b
Local equilibrium scour depth	12.0 ft	14.2 ft	21.6 ft
	Average = 15.9 ft		

Considering the average of the three calculations, 16 feet of local scour could be expected during the design flow. However, because of the similarity of two of the three estimates, it is reasonable to assume that the equilibrium scour depth will probably be less than 16 feet.

5.3.11 Contraction Scour

Dicussion - Contraction scour was defined in Section 5.1.2 as a special case of general scour. Scour at a contraction occurs because the flow area becomes smaller than the normal channel and the average velocity and bed shear stress increase, hence there is an increase in stream power (τV) at the contraction and more bed material is transported through the contracted section than is transported into the section. As the bed level is lowered, 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.

Application - Evaluation of contraction scour is by application of the sediment continuity principle for conditions after equilibrium has been achieved. That is,

$$Q_{s_1} = Q_{s_2} \tag{5.19}$$

where Q_{s_1} is the sediment transport capacity at the upstream section and Q_{s_2} is the value at the contraction. When the sediment transport capacity is expressed in the form of power functions (e.g. as given in Tables 5.6a, 5.6b and 5.7), the relationship is

$$a\ Y_1^b\ V_1^c\ W_1 = a\ Y_2^b\ V_2^c\ W_2 \tag{5.20}$$

Through manipulation and simplification of this equation, a relationship for the flow depth Y_2 (after equilibrium is established) can be derived as

$$Y_2 = \left(\frac{q_{s_2}}{a\ q_2^c}\right)^{\frac{1}{b-c}} \tag{5.21}$$

where

$$q_{s_2} = \frac{W_1}{W_2}\, q_{s_1} \qquad (\text{where} \qquad q_{s_1} = a\ Y_1^b\ V_1^c) \tag{5.22}$$

and

$$q_2 = \frac{W_1}{W_2}\, q_1 \tag{5.23}$$

The amount of general scour is then the difference between the pre-scour flow depth and that value from Equation 5.21 after equilibrium had been achieved, i.e.,

$$\Delta Z_{gs} = Y_2 - Y_2' \tag{5.24}$$

where ΔZ_{gs} is the general scour depth and Y'_2 is the original flow depth at the contraction.

If the site under investigation has hydraulic and sediment properties that fall outside of the limits listed in Tables 5.6a, 5.6b and 5.7, a set of regression coefficients (a, b and c) should be developed for the specific conditions at that site. For example, this regression analysis can be performed by using the Meyer-Peter, Mueller bed-load equation in combination with the Einstein suspended-load methodology to compute the unit width bed-material load transport rate for a range of discharges at the site under investigation. Each unit transport rate is then regressed against the corresponding velocity and depth parameters for the given water discharges (e.g. as established from HEC-2 results). The results of this regression analysis yield values for a, b and c (describing the equation $q_s = a\, Y_h^b\, V^c$) which can then be used in the above contractual scour analysis. As a less time consuming and less site-specific alternative to the regression analysis approach, the engineer may opt to utilize the scour equations presented on pages 58 through 62 of Sedimentation Engineering, ASCE Manuals and Reports on Engineering Practice No. 54 (1975).

Example - Construction of a bridge will result in a reduction in channel width from 320 to 240 feet. Water-surface profile analysis with the bridge in place established velocity and depth in the reach upstream of the proposed bridge as 8.6 fps and 10 feet, respectively, for a peak discharge of 27,500 cfs. Similarily, at the bridge site the velocity and depth were computed as 10.2 fps and 11.2 feet, respectively.

Considering the bed-material characteristics, the appropriate empirical power relationship for sediment transport (Table 5.6a) is

$$q_s = 3.45 \times 10^{-6}\, Y_h^{-0.693}\, V^{4.60}$$

Therefore, the unit sediment discharge upstream of the bridge is

$$q_{s_1} = 3.45 \times 10^{-6}\, (10)^{-0.693}\, (8.6)^{4.60}$$

$$= 0.014 \text{ cfs/ft}$$

and at the bridge site

$$q_{s_2} = \frac{320}{240}\, 0.014\, \frac{\text{cfs}}{\text{ft}}$$

$$= 0.019$$

The unit water discharge at the bridge site is

$$q_2 = \frac{320}{240}\left(\frac{27,500}{320}\right) = 114.6$$

The flow depth at the bridge site after equilibrium is

$$Y_2 = \left(\frac{0.019}{3.45\text{x}10^{-6}\ (114.6)^{4.60}}\right)^{\frac{1}{-0.693-4.60}}$$

$$= 12.1 \text{ ft}$$

The amount of scour is then

$$\Delta Z_{gs} = 12.1 - 11.2 = 0.9 \text{ ft}$$

5.3.12 Bend Scour

Discussion

The bends associated with meandering channels will induce transverse or "secondary" currents which will scour sediment from the ouside of a bend and cause it to be deposited along the inside of the bend. It is important to note that this scouring mechanism is caused by the spiral pattern of secondary flow, and is not due to a shift of the maximum longitudinal velocity filament against the outer bank. Channel bends will cause a shift in this velocity filament, but through the bend the maximum longitudinal velocity is normally moved nearer to the inside bank, whereas the shift to the outer bank occurs downstream of the bend. It is at these downstream locations that the shift in longitudinal velocity patterns will most likely cause lateral erosion of a channel bank.

The discussion presented in this manual will address the vertical scour potential in a channel bend. A review of technical literature will reveal the existance of several theoretical relationships that have been developed to predict the amount of scour through a river bend. To date, there is no known procedure which consistently yields an accurate prediction of bend scour through a wide range of hydraulic and geometric conditions. Based on the assumption of constant stream power through the channel bend, Zeller (1981) developed the following relationship for estimating the maximum scour component resulting from channel curvature in sand-bed channels:

$$\Delta Z_{bs} = \frac{0.0685\ YV^{0.8}}{Y_h^{0.4}\ S_e^{0.3}} \left[2.1 \left(\frac{\sin^2 \frac{\alpha}{2}}{\cos \alpha}\right)^{0.2} - 1\right] \quad (5.25)$$

where ΔZ_{bs} = bend scour component of total scour depth (feet)

V = mean velocity of upstream flow (fps)

Y = maximum depth of upstream flow (feet)

Y_h = hydraulic depth of upstream flow (feet)

S_e = upstream energy slope (bed slope for uniform flow conditions, feet/feet)

α = angle formed by the projection of the channel centerline from the point of curvature to a point which meets a line tangent to the outer bank of the channel (degrees, see Figure 5.25)

Mathematically, it can be shown that, for a simple circular curve, the following relationship exists between α and the ratio of radius of curvature to channel topwidth.

$$\frac{r_c}{W} = \frac{\cos \alpha}{4 \sin^2(\frac{\alpha}{2})} \quad (5.26)$$

where r_c = radius of curvature to centerline of channel (feet)

W = channel topwidth (feet)

If the bend under evaluation deviates significantly from a simple circular curve, the engineer should consider dividing the bend into a series of circular curves and analyzing the bend as a compound curve. Under this procedure, there would be a different value of α determined for each segment of the compund curve. A scour depth would then be computed for each segment of the curve using the α determined for that segment.

Application

Equation 5.25 can be applied to natural river bends to get an approximation of the scour depth that can be expected in the bend during a specific water discharge. The impact that other simultaneously occurring phenomena such as sand waves, local scour, long-term degradation, etc., might have on bend scour is not known for certain. In order that the maximum scour in a

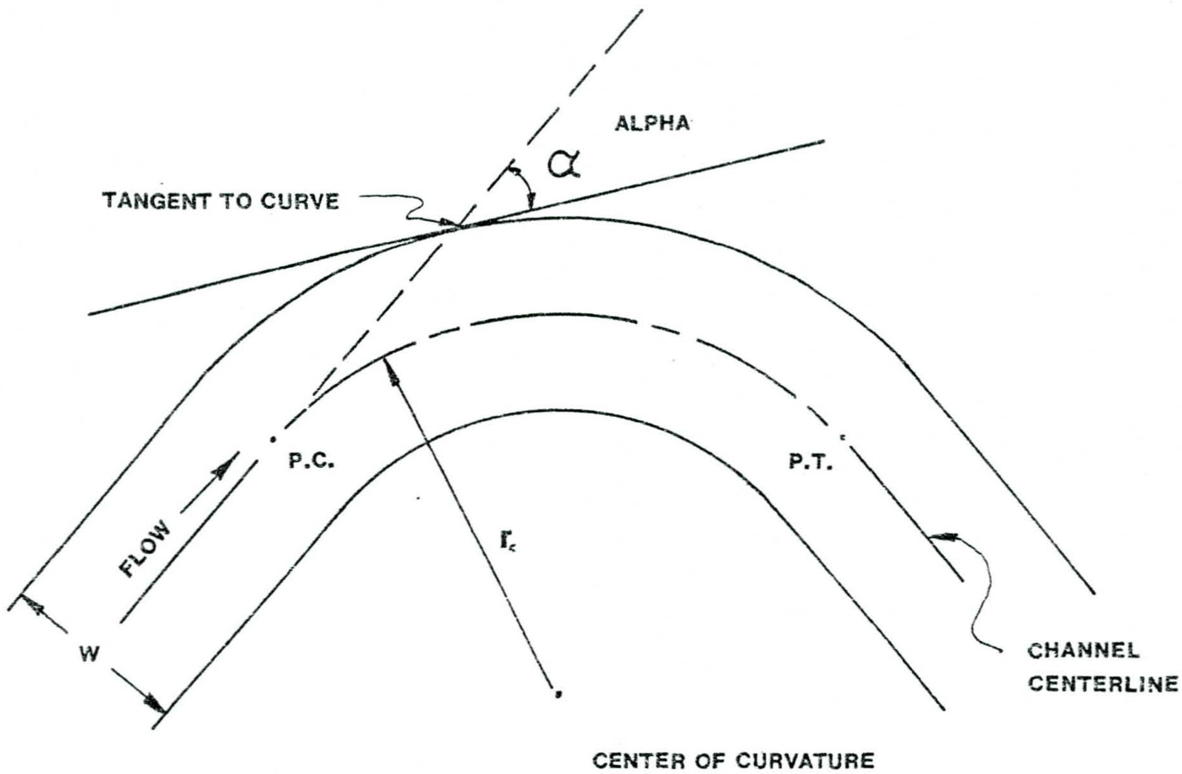


Figure 5.25 Illustration of terminology for bend scour calculations.

bend not be underestimated, it is recommended that bend scour be considered as an independent channel adjustment that should be added to those adjustments computed for long-term degradation, general scour, and sand wave troughs. Whether or not bend scour should be added on top of local scour would depend on the type of obstruction creating the local scour. For isolated structures, such as transmission towers, that would not appreciably disrupt the secondary flow pattern responsible for bend scour, it would be recommended that bend scour and local scour be computed separately and added together. For the case of a series of armored spur dikes placed along the outside bank of a bend, the spiral flow pattern may be disrupted to the point that significant bend scour would not occur. Engineering judgement would have to be exercised in such cases when computing the total vertical scour that might occur in the channel bed.

The longitudinal extent of the bend scour component is as difficult to quantify as the vertical extent. Rozovskii (1961) developed an expression for predicting the distance from the end of a bend at which the secondary currents will have decayed to a negligible magnitude. This relationship is:

$$X = 2.3 \left(\frac{C}{\sqrt{g}}\right) Y \tag{5.27}$$

where X = distance from the end of channel curvature (point of tangency, P.T.) to the downstream point at which secondary currents have dissipated (feet)

C = chezy coefficient

g = gravitational acceleration (32.2 feet/second^2)

Y = depth of flow (to be conservative, use maximum depth of flow, exclusive of scour, within the bend) (feet)

Equation 5.27 should only be used as a guide in determining the distance downstream of a curve that secondary currents will continue to be effective in producing bend scour. As a conservative estimate of the longitudinal extent of bend scour, both through and downstream of the curve, the engineer would be advised to consider bend scour commencing at the upstream point of curvature (P.C.) and extending a distance X (computed with Equation 5.27) downstream of the point of tangency (P.T.) Engineering judgement should be used in electing to deviate from this generalized recommendation.

Example

Proposed channel improvements on a river system include the installation of soil-cement on the channel banks to prevent bank erosion. The river reach where these improvements are to be installed includes a channel bend which has the following hydraulic data:

$Y = 9.39'$	$S_e = 0.0013$ ft/ft
$V = 12.62$ fps	$\alpha = 24°$
$Y_h = 9.18'$	$n = 0.025$

In order to prevent undermining of the soil cement bank protection, it is desired to extend the soil-cement a certain distance below the natural channel bed. This toe-down depth will include allowances for long-term degradation, general scour, sand wave troughs and bend scour. The maximum bend scour component of the toe-down depth is computed as follows:

$$\Delta Z_{bs} = \frac{0.0685\ YV^{0.8}}{Y_h^{0.4} S_e^{0.3}} \left[2.1 \left(\frac{\sin^2 \frac{\alpha}{2}}{\cos \alpha}\right)^{0.2} -1\right]$$

$$\Delta Z_{bs} = \frac{0.0685\ (9.39)\ (12.62)^{0.8}}{(9.18)^{0.4}(0.0013)^{0.3}} \left[2.1 \left(\frac{\sin^2 12°}{\cos 24°}\right)^{0.2} -1\right]$$

$\Delta Z_{bs} =$ 2.09 feet (use 2.1 feet)

This dimension (2.1 ft) will be added to any other computed vertical bed adjustments (general scour, sand wave troughs, etc.) for the curved portion of the channel. The distance downstream of the curve to which the bend scour component will be applied, is computed using Equation 5.27.

$$x = 2.3 \left(\frac{C}{\sqrt{g}}\right) Y$$

where $C = \frac{1.486}{n} R^{1/6}$

For the design flow, the hydraulic radius, R, was determined to be 9.03 feet. Accordingly, C is computed as follows:

$$C = \frac{1.486}{0.025} (9.03)^{1/6}$$

$$C = 85.77$$

Substituting into Equation 5.27:

$$X = 2.3 \left(\frac{85.77}{\sqrt{32.2}}\right) (9.18)$$

$$X = 319 \text{ feet}$$

Therefore, the bend scour component (2.1 feet) will be applied to the soil-cement toe-down depth through the entire curve and for 319 feet downstream of the point of tangency of the curve.

5.3.13 Evaluation of Low-Flow Channel Incisements

Discussion - When large width-depth ratios exist, consideration should be given to the development of low-flow channels. For example, a channel formed predominantly by a 5-year to 10-year flood will develop width and depth characteristics to carry this relatively large discharge in a hydraulically efficient manner; however, for smaller floods these channel dimensions may result in a flow pattern approaching sheet flow conditions. Rather than carrying the flow in this manner, the channel will develop a low-flow channel that provides more efficient conveyance of the low-flow discharges. The development of a low-flow channel will create entirely different hydraulic conditions than those occurring in the original channel geometry, and may create bank instability from incisement. Therefore, it is important for the engineer/designer to anticipate the potential for low-flow channel incisement.

Application - There are no rigorous methodologies for the prediction of low-flow channel incisement. A field inspection of the study area is probably the best method to determine the potential for low flow channel incisement. If the existing channel has developed a low-flow channel, then it is appropriate to use the observed incisement depth for design purposes. If the existing channel does not have low-flow incisement, but proposed channelization or other changes result in conditions favorable for low-flow channel development, then as a rule of thumb a reasonable incisement depth (ΔZ_i) is one to two feet. The incisement depth should be added to any other vertical channel adjustment that is used to determine the burial depth of piers, pipelines, bank stabilization, etc.

5.3.14 Evaluation of Gravel Mining Impacts

Discussion - Common gravel mining practices in arid areas include instream mining, flood-plain mining and terrace mining. Instream and flood-plain mining activities have potential impacts on the river response and require adequate hydraulic, erosion and sedimentation analyses to develop an acceptable mining plan. For example, sand and gravel mining may affect the sediment movement and supply in a channel system. Such operations can be beneficial or detrimental, depending on watershed and river characteristics and on the mining and management practices followed.

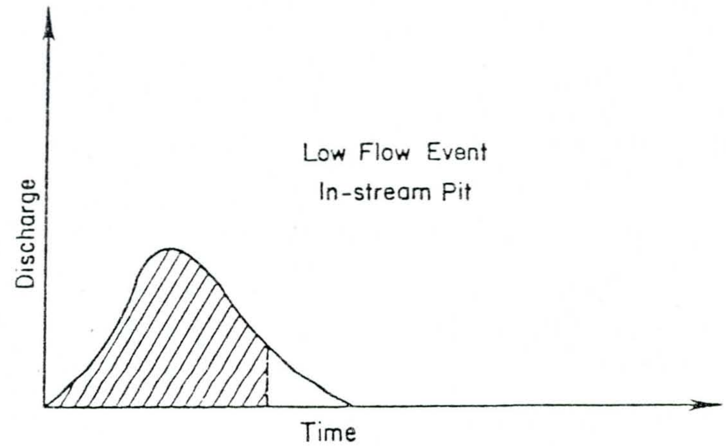
Excessive sand and gravel removal from a river channel (removal greater than supply in any given reach) can endanger the stability of the river system and bridges by inducing general scour and headcutting. For example, bridges over the Salt, Gila and Agua Fria Rivers have been endangered during floods due to significant bed erosion and/or lateral migration of channels. Sand and gravel mining in the river channel has been identified as a contributor to documented bridge instability and/or failure. Analysis of the effects of sand and gravel mining on the stability of a river system and bridges is important, and protection of the bridges may be required where the sand and gravel mining is of significant magnitude.

On the flood plain adjacent to the river channel many of the same processes are at work; however, impacts are generally restricted to overbank flooding conditions. Water and sediment transport rates over the flood plain are generally reduced by the influence on resistance to flow of such flood plain features as vegetation and structures. Just as headcutting above instream gravel pits can endanger upstream bridges, erosion of flood plain gravel pits could encroach on adjacent properties or threaten nearby structures. Of equal concern when flood flows spill over into a gravel pit is the potential erosion of a dike or buffer zone designed to separate the pit from the active river channel. A headcut and erosion through such a buffer zone could alter local river channel characteristics and transport rates, and impact both upstream and downstream reaches. If the channel reach adjacent to a flood plain gravel pit is geomorphically active, e.g., migrating laterally, the same result might occur if protective measures or an adequate buffer zone are not provided during site development.

Application - The extent of damage to the system that can result from a headcut induced by sand and gravel mining is a function of volume and depth of the gravel pit, location of the pit, bed-material size, flood discharge, and sediment inflow rates and volume. The presence of an instream 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 can tip the balance of sediment transport and induce degradation just upstream of the pit and aggradation in the pit. When storm runoff impinges on the gravel pit the energy slope, flow velocity and sediment transport capacity increase at the upstream boundary of the gravel pit and then attenuate in the gravel pit. In response to the changes of sediment transport capacities at the pit boundary, the channel initiates bank sloughing and/or downcutting upstream of the pit. Furthermore, since the velocity of flow through the pit is negligible compared with both the flood-plain and main channel velocities, the pit will act as a sediment trap. Due to this lowered velocity, water leaving the pit does not have the capacity to carry sand- and gravel-sized material. This relatively sediment-free water will flow back into the main channel downstream, and thus the possibility of general scour due to the introduction of clearer water into the main channel must be considered.

The length of time during which conditions are favorable for bank erosion and headcutting depends on the volume of the pit and on the inflow hydrograph. For a low-flow event an instream pit will not fill or reach equilibrium as soon as it will during a high-flow event. During a high-flow event the rising limb of the hydrograph rapidly fills the pit with water and drowns out the effect of a steeper energy slope. This concept is illustrated in Figure 5.26 for representative low- and high-flow hydrographs. The crosshatching indicates the relative times required to fill a gravel pit to the level where channel hydraulics control the flow conditions.

For a gravel pit in the overbank, low flows are generally not of concern. Flood flows will not be of concern until overbank flows occur. While overbank flows are filling a flood plain gravel pit, the same potential exists for headcutting and erosion as with an instream pit; however, once the flood plain pit is filled, it will constitute only a pool or slack-water area on the flood



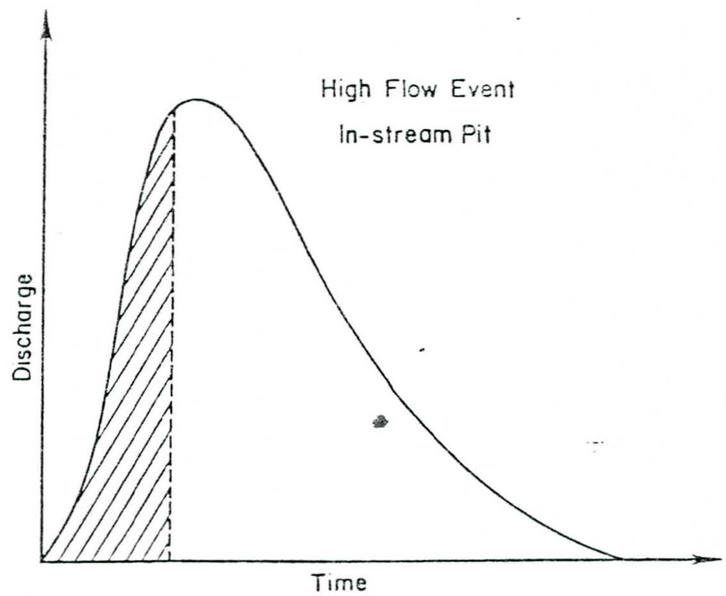


Figure 5.26. Relative time for filling a gravel pit and reaching equilibrium for a low and high flow event.

plain. The central segment of the hydrograph, then, is critical to the stability of a flood plain pit. This concept is illustrated in Figure 5.27, in which the crosshatched area represents the time between overbank spill into the pit and final filling of the pit.

The scour and deposition problems associated with sand and gravel mining are very complicated. Simplifying assumptions are needed to obtain a practical and economical solution. The dominant physical processes include water runoff, sediment transport, sediment routing by size fraction, degradation, aggradation, and breaking and forming of the armor layer. These processes are unsteady and complicated in nature. Furthermore, each situation is unique and requires independent analysis. No standard procedure can be adopted which is universally applicable to all gravel mining evaluations. However, some typical steps that might be required to analyze a headcut profile upstream of a gravel pit would include:

1. Selection of a design hydrograph. Several hydrographs may be evaluated to determine the sensitivity of the gravel pit to different size and shapes of hydrographs.

2. Determine gradation of bed material.

3. Compute hydraulic parameters (velocity and depth) for a range of slope values and the anticipated headcut geometry.

4. Determine unit discharge sediment transport relationship representative of the conditions identified in Steps 2 and 3.

5. Dimension pit geometry for beginning of flood conditions.

6. Select upstream sediment supply cross section and develop transport equation.

7. Route discretized hydrograph through sediment supply section and gravel pit. Adjust bed profile upstream and downstream of the pit entrance at the end of each time step to balance the volume of material eroded from the upstream edge of the pit with the volume of material deposited within the pit.

Example - An instream sand and gravel mining operation just downstream of the Oracle Highway Bridge over Rillito Creek in Tucson, Arizona, was analyzed. The reach length studied was approximately two miles (river mile 4.00 to 6.1), with the bridge located at river mile 5.05. The gravel pit extended from mile 4.65 to 5.03. Assumed dimensions of the pit for analysis purposes were 10 feet deep by 400 feet wide by approximately 2,000 feet long. Upstream of

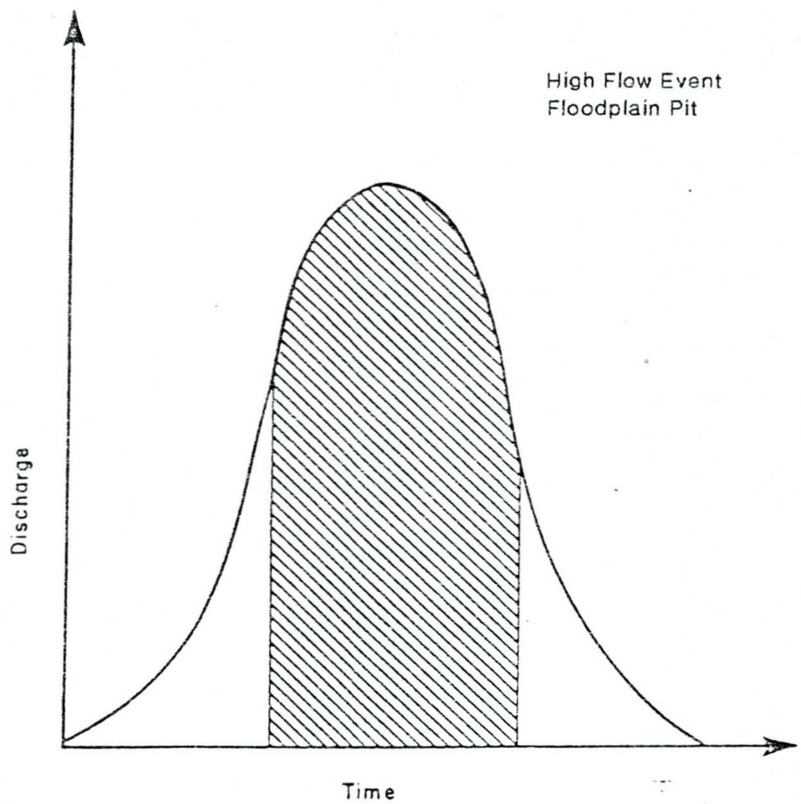


Figure 5.27. Critical time for erosion of a floodplain gravel pit.

the bridge the channel is 350 feet wide. Five cross sections were used within the pit to define the geometric conditions.

The hydrograph used for testing was the two-year flood event with a peak discharge of 7,000 cfs. The 18-hour duration was divided into six time steps of three hours each. The changes occurring in the geometry of the upstream edge of the pit were defined at each of these time increments.

The inital condition was a dry river bed and an empty gravel pit (i.e., no water). Prior to filling the pit with water and sediment, a normal depth approximation was used, rather than the HEC-2 analysis, to determine the hydraulic conditions and sediment transport rate needed for the headcut profile calculations. After the pit filled with water, HEC-2 analysis was used to define the hydraulic conditions. The inflow occurring during the first time step (three hours) initiates the headcut by eroding the corner off the upstream edge of the pit and depositing sediment in the bottom of the pit at the upstream end (Figure 5.28). The slope of the headcut and deposited material is 0.050; however, a discontinuity of 2.40 feet exists. At time 5.20 hours the discontinuity between the headcut and deposition slope disappears and a continuous slope of 0.050 exists. The changes occurring throughout the hydrograph are illustrated in Figure 5.28. The pivot point actually shifted upstream 18 feet, although the resolution on the figure does not illustrate this. The calculated scour occurring at the bridge as a result of the headcut was 4.7 feet at the end of the storm, which is consistent with actual soundings that indicated approximately five feet of general scour for this event.

5.3.15 Cumulative Channel Adjustment

Discussion - The potential vertical adjustment of the channel bed in any given reach is determined from consideration of all the possible incremental adjustments. For example, it is possible that a given reach will be simultaneously degradational while local scour and contractual scour are occurring at the bridge crossings. In this situation the three erosion components would have to be accounted for to establish the ultimate bed elevation.

Application - The cumulative channel adjustment at any given location is the summation of six possible components:

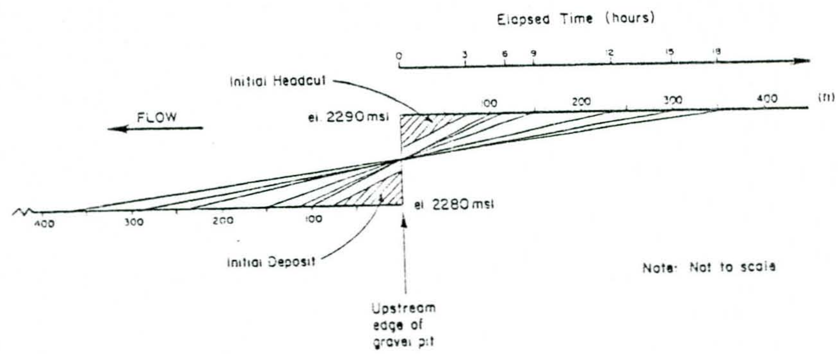


Figure 5.28. Definition sketch of the temporal changes at the upstream edge of a gravel pit.

$$\Delta Z_{tot} = \Delta Z_{deg} + \Delta Z_{ls} + \Delta Z_{gs} + \Delta Z_{bs} + \Delta Z_i + \frac{1}{2} h_a \qquad (5.28)$$

where ΔZ_{tot} is the total vertical adjustment in bed elevation, ΔZ_{deg} is the change from long-term degradation (Section 5.3.9), ΔZ_{ls} is the local scour depth (Section 5.3.10), ΔZ_{gs} is any relevant general scour depth (e.g. Section 5.3.11 or 5.3.14), ΔZ_{bs} is the bend scour depth (Section 5.3.12), ΔZ_i is the low-flow incisement depth (Section 5.3.13), and h_a is the antidune wave height (Section 4.6.2). As a conservative practice, any long-term aggradation amount that might mitigate some elevation decrease is normally not considered.

Due to the complex interaction that will occur among these six phenomena, it is perhaps impossible to accurately predict the total cumulative bed adjustment that might occur at a given location. The hydraulic parameters (velocity, depth, top width, etc.) that are used to compute the dimension of each phenomenon will constantly change as this interaction proceeds; however, the parameters that are used in the calculations are normally based on rigid-bed conditions which give no consideration to channel geometry changes that may be initiated as a result of the simultaneous occurrence of all or part of the six phenomena. Accordingly, the application of a factor of safety to the total computed vertical adjustment (ΔZ_{tot}) is very judgmental, i.e., no firm value can be recommended. In deciding to apply a factor of safety to the computed result, the engineer should consider the magnitude of damage that might accompany a design failure, the probability or risk that such an event might occur, the construction cost associated with applying a safety factor, and the reliability of the data that were used in the channel adjustment calculations. Depending upon the answer to such questions, typical safety factors will probably range from 1.0 to 1.5.

Example - In the example of Section 5.3.9, a potential degradation of 9 feet was calculated for Reach 3 of the Pinal Creek channel. For purposes of illustration, assume a bridge crossing in this reach produces local scour at the bridge piers of 12 feet and 0.5 feet of general scour through the contraction. The channel is straight through this reach of Pinal Creek, therefore bend scour is not applicable. Inadequate data exist to compute low-flow channel incisement, and therefore a value of one foot is assumed. The potential antidune height is calculated as 3.9 feet. Therefore, the total possible scour at the bridge piers, considering a safety factor of 1.0, is

$$\Delta Z_{tot} = 9 + 12 + 0.5 + 0 + 1.0 + \frac{1}{2}(3.9)$$

$$= 24.5 \text{ ft}$$

5.4 Level III Analysis

5.4.1 General

As discussed in Section 2.4.4, Level III analysis involves application of various physical-process mathematical models and provides the most accurate method of analysis. Physical-process models represent the system being modeled by dividing it into the relevant components, or physical processes. In comparison with regression-based models, where several controlling physical processes may be lumped into one parameter or equation, physical-process models uniquely consider the governing equations of each relevant physical process. For example, a physical-process model for water routing from a watershed would include equations describing interception losses, infiltration rates, overland flow routing, and channel flow routing. The sophistication of most physical-process models, particularly in terms of the number of physical processes considered and the iterations performed in solution of various equations, requires computer application for solution.

5.4.2 Application of Level III Analysis

The decision to conduct a Level III analysis is generally based on project objectives under the constraints of time and budget. For engineering analysis of fluvial systems, the most common Level III analysis applied is the evaluation of erosion/sedimentation using a moveable-bed model. Models developed for this purpose include HEC-6 (U.S. Army COE), HEC-2SR (Simons, Li & Associates, Inc.) and others. With a moveable-bed model the channel geometry is updated during a given flood simulation to reflect the erosion/sedimentation that has occurred. In contrast, the sediment continuity procedure (discussed in Section 5.3.6) is a simplification of this analysis where the channel boundary is not updated. Generally, results of sediment continuity tend to overpredict, providing conservative erosion/sedimentation volumes. Therefore, the decision to conduct a Level III analysis might be motivated by the desire or need for more accurate, refined results.

This need for more accurate results must be balanced by the time and money available. As the analysis becomes more complicated, accounting for more factors, the level of effort necessary becomes proportionally larger. In the analysis of fluvial systems no computer model can be treated as a black box. Proper application of the model relies upon an understanding of how the model operates and upon careful evaluation and interpretation of results. As with any model, the computer is simply a tool to expedite tedious or multiple calculations, and conclusions will still rely on engineering judgment and interpretation.

This concern illustrates the value of the three-level analysis approach, where the results of Levels I and II provide insight and guidance to the Level III analysis. The Level III analysis is never a substitute for Levels I and II; rather, the results of all three levels complement each other and minimize the risk of erroneous conclusions. To initiate analysis of a complicated problem with a Level III approach prior to Levels I and II could not only provide incorrect solutions, but result in wasted time and effort.

As discussed in Section 5.3.6, for many practical engineering problems the sediment continuity analysis of Level II is adequate, without incurring the time or expense of a Level III analysis. In deciding if the Level II analysis is adequate, each case will need to be evaluated independently, weighing the objectives of the project against the available time and budget.

VI. CHANNEL DESIGN CRITERIA

6.1 General

Information presented in Chapters I to V provides the basic tools to conduct a comprehensive engineering analysis of fluvial systems. Not all the techniques and methodologies presented will be applicable or necessary in every situation encountered. Project objectives and scope will determine the type of analysis and level of sophistication necessary. Through proper selection and application of the methodologies presented, the engineer or designer can complete a logical sequence of analyses that will provide a comprehensive understanding of the fluvial system and its response mechanisms.

Such knowledge of the fluvial system is useful in and of itself in order to explain various historical events and/or to predict possible future conditions. Furthermore, and of equal importance, such knowledge will establish the design criteria for channelization, bridge design, bank revetment and other structures located in the channel or flood plain. Information useful to both major and minor engineering design has been presented in Chapters I to V. Major design is one where if failure occurred, loss of life is possible and/or loss of property could be significant. For a minor design, lives are not in jeopardy and the potential loss of property is relatively insignificant. Major drainage design involves application of the more rigorous analysis procedures that provide reasonable quantitative results. Conversely, minor drainage design can often be completed using simplified concepts, rules of thumb, minimum criteria, etc. The following sections briefly discuss some of the specific applications of information in Chapters I to V to both major and minor engineering design work.

6.2 Bank/Levee Height

The total bank/levee height required will be the bank/levee height necessary to contain the design flood plus any freeboard. The minimum guidelines discussed in Section 4.6.5 (2.0 ft in rectangular, 2.5 ft in concrete trapezoidal, riprap or soil cement channels, and 3.0 for earthen levees) provide a means of checking the results of Equations 4.28a and 4.28b. These guidelines are often adequate for direct application to minor structure design. For major or minor designs with Froude numbers near one, an additional factor of safety may be appropriate due to the potential for standing waves and other flow instabilities, if such phenomena cannot be directly quantified.

6.3 Bank/Levee Toedown

Toedown is the distance bank revetment must be buried to prevent undermining as the bed elevation fluctuates. Equation 5.28 of Section 5.3.15 provides the total cumulative channel adjustment possible from six components (degradation, local scour, general scour, bend scour, low flow incisement and bed forms). Local conditions will establish which of these components must be accounted for.

Selection of a safety factor is dependent upon acceptable risk, construction costs, available data and sophistication of analysis, i.e. Level I, II or III. As stated in Section 5.3.15, safety factors will probably range from 1.0 to 1.5. Due to the nature of sediment transport calculations and the importance and expense of bank revetment, engineering judgment should always be applied.

6.4 Lateral Migration

One important application of lateral migration analysis (Section 5.3.9) in design work is for establishing a buffer zone for erosion and flooding in which development would not be considered prudent. In this context, the operational definition of the term "prudent" is related to the concepts of hydrologic uncertainty, that is, the acceptable degree of risk established by the return period (recurrence interval) of a hydrologic event. The National Flood Insurance Program establishes as a precedent that when considering hydrologic events in urban areas it is generally not considered an exercise of sound judgment to accept a degree of risk any greater than that associated with the 100-year event. With reference to the calculated risk diagram (~~Figure 6.1~~), using the 100-year event as a basis for the definition of "prudent" implies that there is a 90 percent certainty that the event will not occur in a 10-year period and about 78 percent certainty that it will not occur in 25 years. Conversely, this means acceptance of a calculated risk of 10 percent in a 10-year period and 22 percent in a 25-year period if boundaries of the buffer zone are based on the erosion and flooding potential of a 100-year flood. Asking a property owner to accept a greater risk than this would not appear to be prudent.

While damages due to flooding are generally associated with a single, short-term event, the impacts of erosion (lateral migration) can also be cumulative over the long term. Consequently, one must assess the erosion poten-

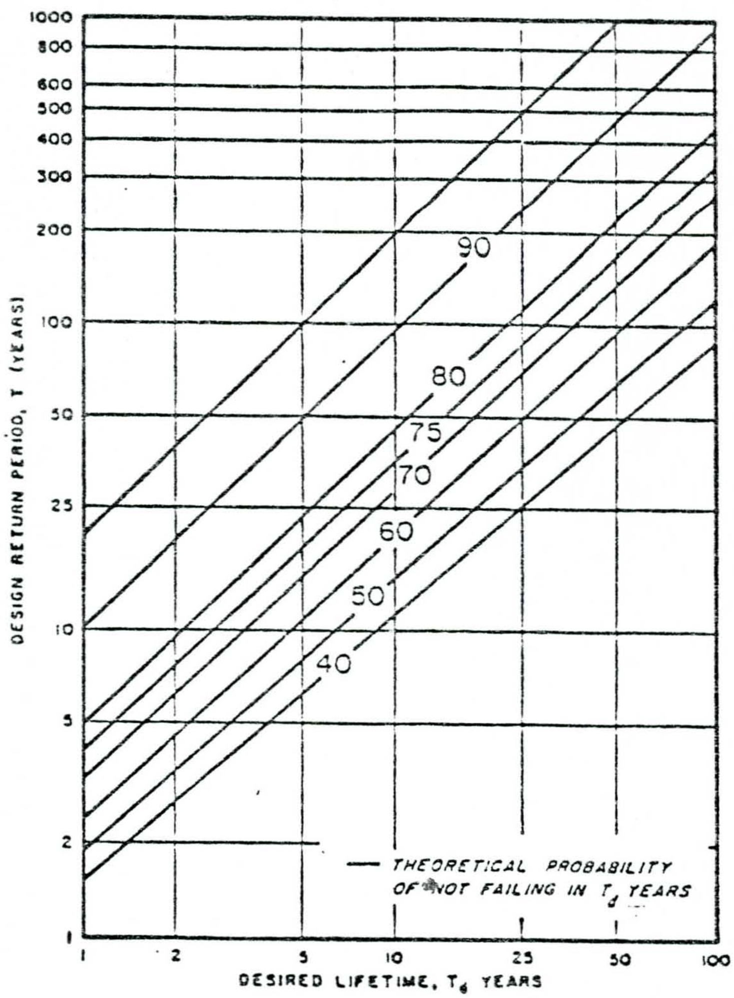


Figure 6.1. Calculated risk diagram.

tial not only of a single event, such as a 100-year flood, but also the cumulative impact of a series of smaller flows. One approach to evaluating long-term erosion impacts is to develop a "representative" annual storm and then to extrapolate in time the effect of this storm. This concept is similar to the practice in hydrology of adopting the two-year flood as being representative of the annual event; however, for purposes of long-term erosion analysis the representative annual event can be more accurately defined by a probability weighting of the erosion resulting from several single storms (see Section 3.4). With this approach the long-term analysis of erosion potential accounts for the probability of occurrence of various flood events during any one year.

After establishing the representative annual storm for evaluating long-term erosion (lateral migration) potential, the duration in years defining the "long term" must be determined. For example, based on both the limitations of the probability weighting approach and the single-event probability of occurrence of a 100-year flood in a 25-year period (22 percent), a reasonable definition of the "long term" for an urban area might be 25 years. Thus the boundaries of an erosion and flooding buffer zone could represent the envelope established by the reach-by-reach calculation of the erosion and flooding potential of either the 100-year flood (short term) or the cumulative erosion impact of a series of smaller events over a 25-year period (long term), whichever is greater.

The buffer zone is then plotted by consideration of the controlling factor (100-year flooding, 100-year erosion, or long-term erosion) for each cross section used in the analysis. The boundary of the buffer zone between cross sections can be drawn as a smooth curve or as a series of tangent lines that can be easily referenced to existing survey data and readily compared with existing survey plats. The buffer zone so defined goes one step further than the conventional 100-year flood plain boundary by considering potential changes in channel configuration from lateral migration. The selection of this definition of the buffer zone is supported by the legal and policy precedents of the National Flood Insurance Program, the short- and long-term degree of risk associated with the 100-year return period event, and the accuracy of the methodology used for estimating long-term erosion impacts [i.e. by limiting the extrapolation to reasonable length in time (e.g., 25 years)]. For detailed discussion of the methodology and its application the reader is referred to Lagasse, et al. (1984).

6.5 Grade-Control Structures

Grade-control structures are effective channel stabilization measures that may be used singly or as an integral part of a stabilization plan involving bed and bank revetment, etc. The primary function of a grade-control structure is to decrease the gradient of a channel to either create a condition of equilibrium (sediment inflow equal to sediment outflow), or to reduce the protection required from other stabilization measures. For example, a grade-control structure can be used to decrease the channel slope so that smaller riprap can be used for stabilization. If sufficient coarse material exists in the natural alluvium, it may be possible to use grade-control structures to assist in developing an armor layer and avoid the need for all or part of the bed or bank revetment.

Grade-control structures can range in complexity from simple rock riprap or soil-cement drop structures to large concrete structures with baffled aprons and stilling basins. For many applications in the Southwest a series of smaller soil-cement drop structures may be more effective and economical than a single concrete structure of larger dimensions.

The design of grade-control structures to create equilibrium conditions is based on the equilibrium slope (Section 5.3.7). The design of grade-control structures to be used in combination with riprap is based on the incipient motion slope, as defined by the Shields relation (Section 5.3.4). After establishing the required design slope, the number and spacing of the structures must be determined. The vertical height that must be controlled for a given reach to achieve the required slope can be evaluated from

$$\Delta H = (S_0 - S)\ \Delta X \tag{6.1}$$

where ΔH is the total height requiring structural control within the reach, S_0 is the original channel slope, S is the estimated design slope, and ΔX is the length of channel to be controlled. The number of drop structures required depends on the maximum allowable height of each structure (H_{max}), which is a function of the type of structure utilized. Rules of thumb for conservative design are three feet for riprap drop structures and five feet for soil cement. The number of structures required (N) to control the total vertical height within a reach is

$$N = \frac{\Delta H}{H_{max}} \tag{6.2}$$

and the spacing L of the drop structures is

$$L = \frac{\Delta X}{N} \tag{6.3}$$

6.6 Common Bank Protection Methods

Numerous types of bank protection are available, including vegetation, rock riprap (dumped, hand-placed, wire-enclosed and grouted), soil cement and concrete, mattresses (concrete, brick, willow and asphalt), jacks and jetties, dikes (rock-filled, earth-filled and timber), automobile bodies, and many others. Many publications on channel stabilization have been prepared by various government agencies and others detailing the design and application of different techniques. It is not intended that an exhaustive coverage of the various channel stabilization measures be made in this section, but rather to briefly review those methods that are most appropriate to, and have proven successful in, the Southwest. In particular the use of dumped rock riprap, wire-enclosed riprap and soil cement will be considered.

Rock riprap is usually the most economical material for bank protection when available in sufficient size and quantify within a reasonable haul distance. Rock riprap protection is flexible and local damage is easily repaired. Construction must be accomplished in a prescribed manner, but is not complicated. Although riprap must be placed to the proper level in the bed, there are no foundation problems. Appearance of rock riprap is natural and after a period of time vegetation will grow between the rocks. Wave runup on rock slopes is usually less than on other types of slopes. Finally, when the usefulness of the protection is finished, the rock is salvageable.

Important factors to be considered in designing rock riprap protection are:

1. Durability of the rock.
2. Density of the rock.
3. Velocity (both magnitude and direction) of the flow in the vicinity of the rock.
4. Slope of the bed or bank line being protected.

5. Angle of repose for the rock.

6. Size of the rock.

7. Shape and angularity of the rock.

A good discussion of many of these factors is provided by the Corps of Engineers (1970). In this COE publication the size of rock required is based on the tractive force approach, generally considered to be a more physically based and more accurate method than those based on the permissible velocity approach.

A tractive force approach that provides the entire channel design (geometry and riprap size), given the design discharge and slope, is detailed by Anderson, et al. (1970). A more involved tractive force approach that generally provides a more precise riprap size through detailed consideration of lift and drag forces is the safety factor approach, presented in Simons and Senturk (1977). Another tractive force approach that considers in detail the lift and drag forces is the probability methodology presented by Li and Simons (1979). This methodology defines the failure probability of riprap and provides a less subjective estimate of riprap stability than that provided by the safety factor approach. For example, Li and Simons demonstrate that for an assumed set of flow and geometry conditions (conditions that are in the range of many practical design situations) a riprap safety factor of 1.0 has a probability of failure of 0.5. Similarly, a safety factor of 1.5 has a probability of failure of about 0.1 and not until a safety factor of about 1.9 is the failure probability equal to zero.

In contrast to the relative complexity of factor of safety and failure probability designs, is a permissible velocity approach that has found acceptance due to its ease of application. The method is detailed by the Denver Urban Drainage and Flood Control District (1982) and is useful as a check on riprap designs developed from more complex procedures or as the primary design method, particularly for minor structure design. The method is limited to Froude numbers less than 0.8 and due to its simplicity is anticipated to provide conservative design, a consideration that may be of importance if larger rock sizes are not readily available or if budget constraints exist.

After evaluating the required median riprap size, the riprap gradation and filter requirements must be established. Riprap gradation should follow a smooth particle size distribution with a ratio of the maximum size and the median size of about two and the ratio of the median size and the 20 percent size also about two. With a distributed size range, the interstices formed by larger stones are filled with smaller sizes in an interlocking fashion, preventing formation of open pockets. Riprap consisting of angular stones is more suitable than that consisting of rounded stones. Control of the gradation of the riprap is almost always made by visual inspection.

Filters underneath the riprap are recommended to protect the fine embankment or riverbank material from washing out through the riprap. Two types of filters are commonly used: gravel filters and plastic filter cloths. Detailed filter design is provided by the COE (1970), Anderson, et al. (1970), Simons, Li & Associates, Inc. (1981), and others.

When adequately sized riprap is not available, rocks of cobble sizes may be placed in wire mesh baskets and used for a variety of channel stabilization problems. The baskets are constructed into various geometric shapes depending on the application. For channel lining applications mattresses are commonly used, which as the name implies are relatively broad and flat (typically less than 12 inches thick). Rectangular baskets (gabions) of more symmetrical proportion are often used as building blocks for check dams, drop structures, bank protection, etc. Modern gabions and mattresses are made of a thick steel wire mesh, woven with a triple twist at the intersections. Heavy wire is sometimes added or woven into the mesh before or after filling to increase stability and durability. The wire mesh can be galvanized or coated with PVC if used under highly corrosive conditions.

The strength and flexibility of the steel wire mesh allows gabions and mattresses to change shape without failure if undermined. They are also permeable, which minimizes hydraulic lift forces, allows vegetation to grow and provides some trapping efficiency. It should be noted than when gabions or mattresses are used in streams transporting cobbles or rocks, the wire baskets can be damaged or broken, reducing or destroying the protection near the bed.

Gabions and mattresses are supplied to the job site as folded mesh and tied in pairs. They are unfolded, placed in position like brick, tied together and filled with durable rock. The mesh containers can also be filled first and placed by hand or by crane to areas difficult to access (e.g. under water).

After an extensive literature review, Simons, et al. (1983) concluded that there was little information on the design of mattresses for channel lining, consequently, from model and prototype studies, design guidelines were formulated. The major findings of the model and prototype studies where (1) that hydraulic conditions in a mattress channel are the same as those in a gravel-cobble channel, (2) the roughness of the mattresses is mainly caused by the filling rocks, with an insignificant effect from the wire mesh, and (3) flow velocity and shear stress causing incipient motion of the filling rock within the mattress compartment are approximately twice as large as the same size of unbound rock. For steep slope channels of high velocity, rocks within mattress compartments were found to propagate downstream, causing a ripple deformation of the mattress surface. Additionally, because of relatively large velocities at the mattress-to-bed interface, filter requirements and design are critical to successful steep slope application. Following the design procedure suggested by Simons, et al. provides a mattress thickness that is 1.5 to 3.0 times less than the required thickness of dumped riprap. Consequently, significant economies of cost are often possible with mattress linings since less rock is required, the required size is smaller and excavation requirements are less.

In areas where any type of riprap is scarce, use of in-place soil combined with cement provides a practical alternative. The resulting mixture, soil cement, has been successfully used as bank protection in many areas of the Southwest. A stairstep construction is typically used, with each lift about 12 inches thick before compaction and about 6 to 8 inches after compaction. The lifts are usually about 8 feet wide to easily accommodate construction equipment. Unlike other types of bank revetment, where milder side slopes are desirable, soil cement in a stairstep construction can be used on steeper slopes (i.e. typically one to one), which reduces channel excavation costs. For many applications, soil cement is also more aesthetically pleasing than other types of revetment.

For use in soil cement, soils should be easily pulverized and contain at least 5 percent, but no more than 35 percent, silt and clay (material passing the No. 200 sieve). Finer textured soils usually are difficult to pulverize and require more cement, as do 100 percent granular soils, which have no material passing the No. 200 sieve. In construction, special care should be exercised to prevent raw soil seams between successive layers of soil cement. If uncompleted embankments are left at the end of the day, a sheepsfoot roller should be used on the last layer to provide an interlock for the next layer.

The completed soil cement installation must be protected from drying out for a seven-day hydration period. Procedures for constructing soil cement slope protection by the stairstep method can be found in "Suggested Specifications for Soil-Cement Slope Protection for Embankments (Central-Plant Mixing Method," Portland Cement Association Publication IS052W).

When velocities exceed six to eight feet per second and the flow carries sufficient bed load to be abrasive, special precautions are advisable for soil cement design. The aggregates in this case should contain at least 30 percent gravel particles retained on a No. 4 (4.75 mm) sieve.

VII. COMPREHENSIVE DESIGN EXAMPLE

7.1 Project Description

Sportsman's Haven is a small, resort-oriented community located along Pinto Creek in Gila County, Arizona. Due to its location within the 100-year flood plain, the community is subjected to flood damage when flows on Pinto Creek exceed the 10-year event of about 16,500 cfs. Attempts have been made locally to provide some degree of flood protection by constructing a levee embankment around the community. Due to the cohesionless nature of the embankment material, which was obtained from the bed of Pinto Creek, the levee is vulnerable to the erosive forces of the floodwaters. Additionally, the levee was not constructed to a sufficient height to prevent overtopping by the 100-year flood (46,785 cfs).

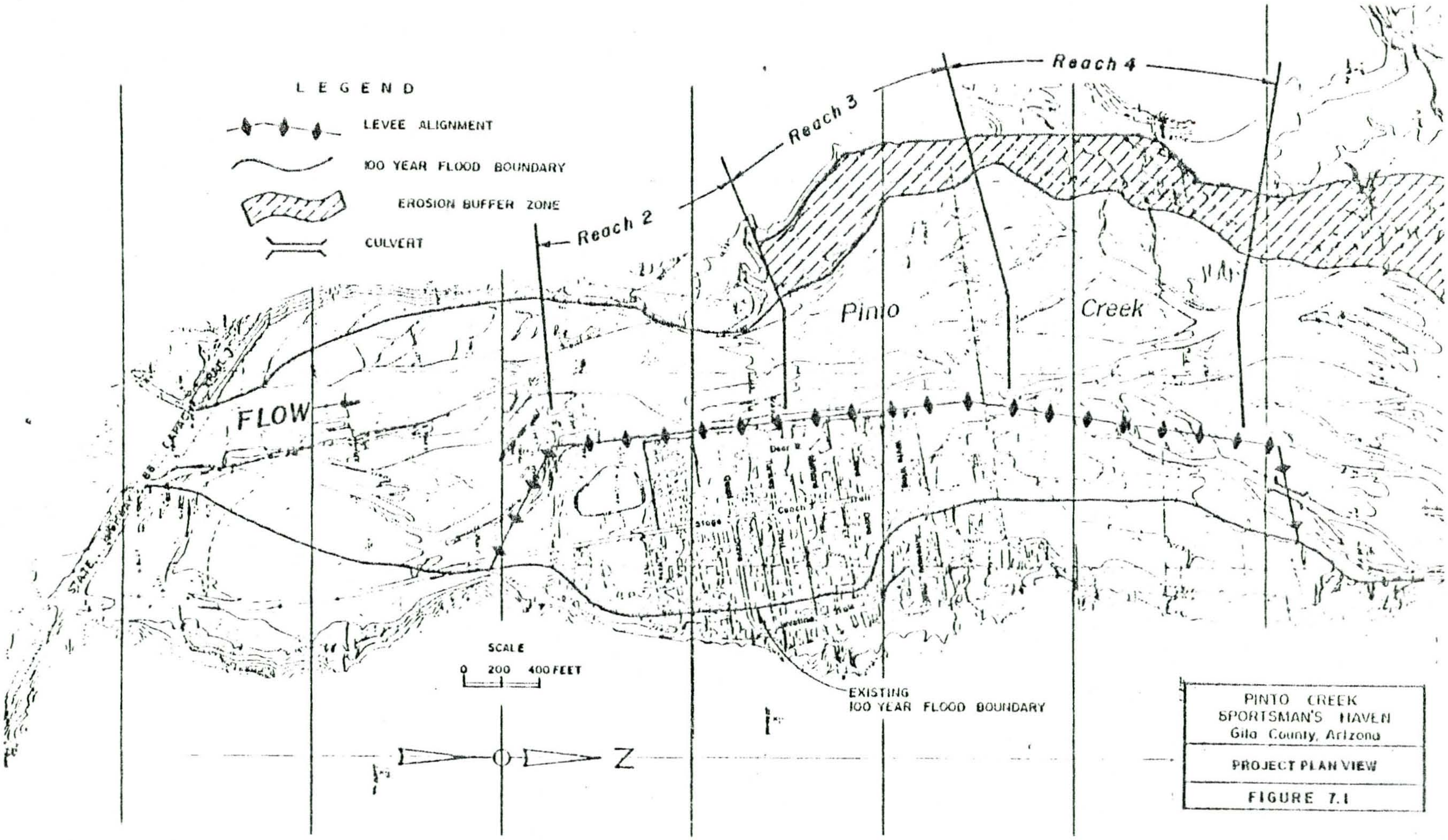
Under the authority of the Flood Control Planning Program, the Arizona Department of Water Resources undertook a reconnaissance level evaluation of this problem and prepared a preliminary levee system design which would provide the community with protection from the 100-year flood. This levee design project was selected to illustrate the application of several of the analytical tools presented in this manual. The following pages present a Level I and Level II analysis of the Pinto Creek levee system. Figure 7.1 presents a plan view of the study area and the proposed levee alignment.

7.2 Level I

General (qualitative) characteristics of Pinto Creek can be determined from a review of historical watershed data and an application of some empirical relationships involving slope, discharge, sinuosity ratio, etc. The amount of information to be derived from this type of analysis will vary from project to project since the amount of available data will vary from site to site.

The Level I analysis for the Pinto Creek Project will address the following items:

1. Sinuosity
2. Geomorphic relationships
    a. Lane
    b. Leopold and Wolman
3. Historical aerial photographs


LEGEND
LEVEE ALIGNMENT
100 YEAR FLOOD BOUNDARY
EROSION BUFFER ZONE
CULVERT
Reach 2
Reach 3
Reach 4
Pinto
Creek
FLOW
SCALE
0 200 400 FEET
EXISTING
100 YEAR FLOOD BOUNDARY
N
PINTO CREEK
SPORTSMAN'S HAVEN
Gila County, Arizona
PROJECT PLAN VIEW
FIGURE 7.1

4. Historical bed profile comparison

5. Visual geological investigation

A detailed discussion on each item follows.

1. Sinuosity - Leopold, Wolman and Miller adopted the sinuosity ratio, which is defined as the thalweg length divided by the valley length, as a criterion which could be used to classify river patterns. Through the observation of several natural river systems, they concluded that systems with a sinuosity ratio equal to or greater than 1.5 would be classified as meandering while those less than 1.5 would be braided or straight.
   The sinuosity of the study reach is computed as follows:

   $$\text{sinousity ratio} = \frac{\text{thalweg length}}{\text{down valley distance}}$$

   $$\text{sinuousity ratio} = \frac{8925}{8175} = 1.1$$

   This low value indicates that this reach of Pinto Creek is straight or braided. Additional analyses will next be conducted to confirm this fact. General conclusions will then be drawn regarding channel pattern classification.

2. Geomorphic Relationships - Figure 5.5 will be used to examine the relationship between slope, discharge and channel pattern. This analysis will be based on the premise that the dominant discharge will be most influential in determining the channel pattern. For the Pinto Creek Project, the average bed slope through the entire study reach is 0.0089 and the dominant discharge is 16,514 cfs. Applying these values to Figure 5.5 shows that the channel plots well into the braided region, using both Leopold and Wolman's criteria as well as Lane's. When using Figure 5.5 the engineer should remember that these relationships were derived from data on perennial channels, rather than ephemeral washes which are more common in Arizona. Accordingly, their strict application in the southwest United States should be with caution and knowledge of their derivation.

3. Historical Aerial Photographs - Three sets of historical photos were located for this reach of Pinto Creek. These photos were taken in 1947, 1967, and 1981. Examination of these pictures indicates a braided channel pattern has existed during the last 38 years (see Figure 7.2). Overall, the braided channel segments appear narrower in the 1947 and 1967 photos than in the 1981 photo. This may be in response to construction of the Pinto Creek bridge on Highway 88 in 1972. The approach embankments to this bridge partially obstruct the floodplain and cause the flow to be concentrated in a smaller width than existed prior to bridge construction. The widening of the main channel for about two miles downstream of the bridge may be the result of this local concentration of high velocity of flow which has cleared out a wider, cleaner channel section. This localized

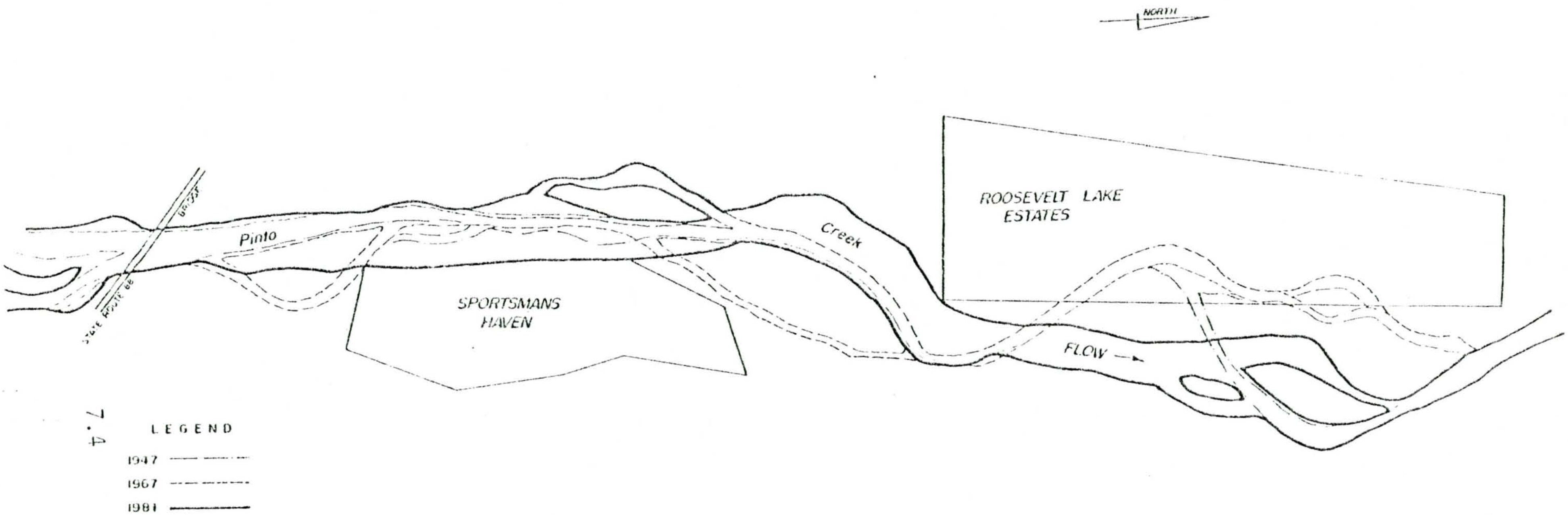


Figure 7.2 Time sequence analysis of historical aerial photographs.

response diminishes beyond the two-mile downstream limit where the channel assumes a more consistent pattern in all three photo sets.

4. Historical Bed Profile Comparison - USGS quadrangle maps were prepared for this area in both 1949 and 1964. Unfortunately, the 1949 map is a 15-minute quad while the 1964 edition is a 7.5-minute quad. This non-uniformity of scale presented a problem since this reach of Pinto Creek is on the edge of the maps and meanders back and forth between them, making it impossible to measure the entire study reach on either map. The third bed profile was taken from a 1981 topographic map (1" = 200, 2' C.I.) prepared especially for this project.

   Taking topographic measurements from these maps, superimposed bed profiles were plotted at a scale of 1" = 1,000' horizontal, 1" = 20' vertical (see Figure 7.3). Through the study reach, this plot shows a fairly consistent drop (about 3.0') in bed elevation from 1949 to 1964 and a varying amount of aggradation (about 0' to 1') from 1964 to 1981.

   There is no evidence of any significant manmade disturbance to the watershed that would readily explain the drop in bed elevation between 1949 and 1964, nor the aggradation that has occurred since 1964. Since Pinto Creek discharges into Roosevelt Lake, fluctuating lake levels could influence the bed profile to a certain degree but is doubtful that such influences would propagate this far (three miles) upstream from the lake. Further investigation into this possibility could be pursured by reviewing historical reservoir level data from the Salt River Project and correlating this information with historical hydrologic/hydraulic data for the watershed to see if significant flooding may have occurred during periods of low lake levels.

   Due to the scale discrepancies on the two quad maps and possible survey datum inconsistencies between the USGS maps and the 1981 base map, this bed profile comparison may not be totally accurate and, therefore, should be interpreted with caution. For these reasons, the historic bed profile evaluation for Pinto Creek will not be considered a reliable tool for the Level I analysis.

5. Visual Geological Investigation - A field visit was made to the project site in order to identify any geologic formations that might control either horizontal or vertical channel movement. This visit also provided an opportunity for a ground level inspection of the channel geometry, channel pattern, and bed material composition. No natural or artifical bed controls were located within the study reach which could be used for pivot points in an equilibrium slope analysis.

   The west bank through Reach 2 was found to consist of rock which will restrict lateral channel migration at this location. This is verified through a cross check with the historic photos which shows no westward bank movement through Reach 2 since 1947.

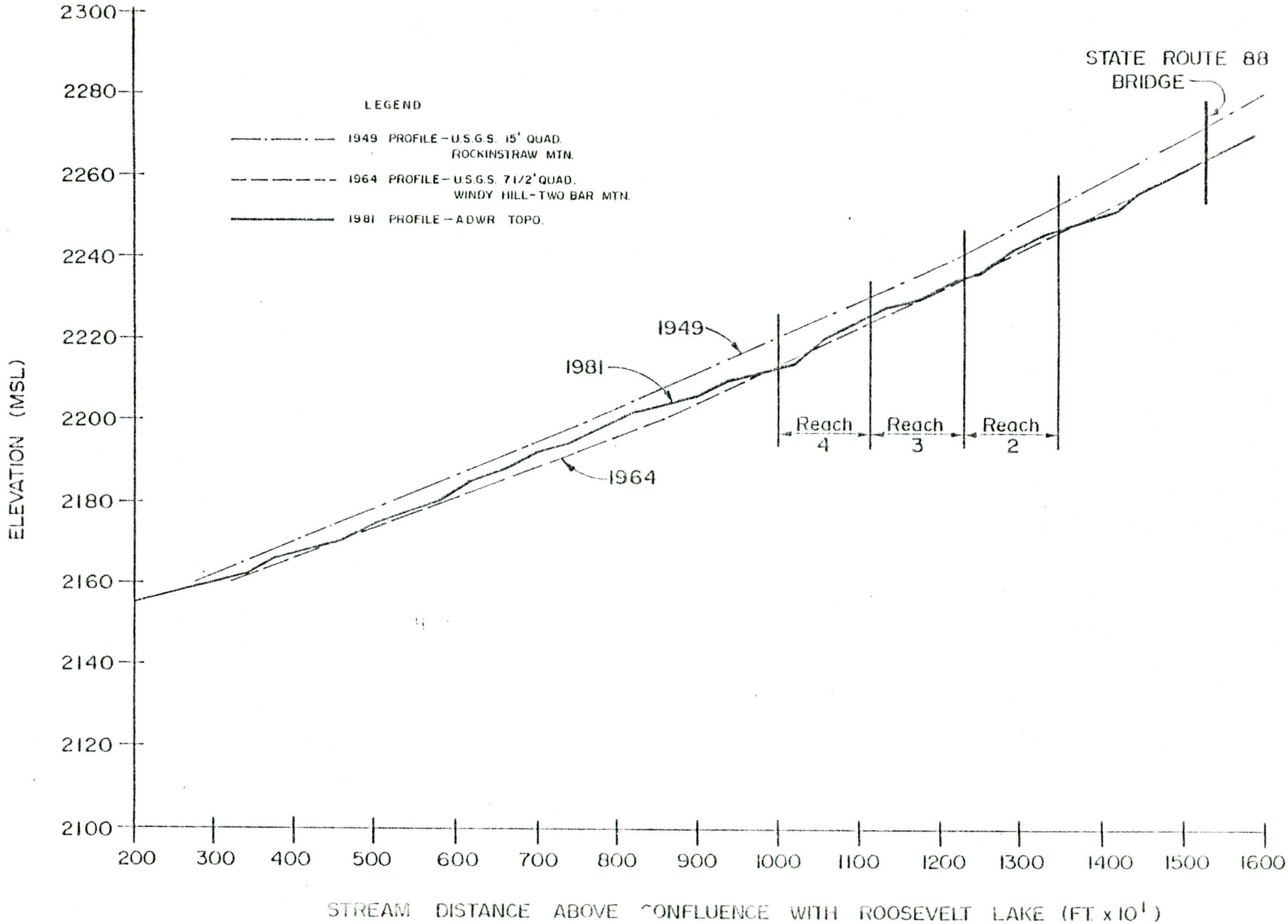


Figure 7.3 Comparison of historical bed profiles.

> Visual inspection of the channel through all four reaches revealed a wide, braided pattern with poorly defined banks that were 2' to 3' in height. The channel bed was composed primarily of sand and sandy gravel with the intermittent presence of small cobbles. Island and bar formations were evident throughout the study reach. Except for the rock formation noted in Reach 2, all other portions of the channel banks consisted of erodible alluvium.

All aspects of this Level I analysis confirm the existence of a braided channel pattern. Braided channels are generally wide, have unstable, poorly defined banks and consist of two or more main channels that cross one another giving the riverbed a braided appearance at low flow. These channels have sinuosity ratios less than 1.5 and exhibit steeper slopes than meandering channels.

Braiding is believed to result primarily from random deposition of materials (sediment) transported during high flows in quantities or sizes too great for continued transport during low flows. Accordingly, as the stream discharge is reduced, larger sediment particles begin to drop to the bed as the stream "sorts" or leaves behind those sizes of the load which it is unable to transport. The accumulation of these particles on the channel bed initiates the formation of a bar which serves to trap even more sediment particles. Although the depth of flow over the growing bar is gradually decreased, velocity over the bar tends to remain undiminished or even to increase so that some particles moving along the bar are deposited beyond the downstream end where a significant decrease in velocity is associated with the marked increase in depth of flow. Thus, the bar grows by successive addition of sediment particles at its downstream end and some additional growth along its sides.

The growth of the bar will eventually reach a size that will significantly alter the channel capacity, at which time the channel will seek a new equilibrium condition by eroding and widening its banks. Additional bars will then be propagated through the same process described above until the channel obtains its characteristic braided pattern.

Because deposition is essential to the formation of the braided pattern, it is obvious that sediment transport is essential to braiding. Also, the channel banks must be sufficiently erodible so that they, rather than the newly formed bar, give way as the channel cross section is increased to provide the required flow capacity. Therefore, sediment transport and erodible banks provide the essential conditions of braiding. However, rapidly fluctuating changes in stage contribute to the instability of the sediment transport regime as well as to erosion of the banks, so this factor should also be considered as a contributory cause of braiding. Studies by Leopold, Wolman and Miller also indicate that heterogeneity of the bed material may create irregularities in the movement of sediment, and thus, may also contribute to braiding.

7.3 Level II

The Level II analysis will provide the technical refinements necessary to establish dimensions and specifications for the actual levee design. The end product of this analysis will be:

1. levee crest profile
2. determine requirements for bank stabilization
3. maximum estimated depth of bed movement adjacent to all portions of the levee
4. estimated distance of lateral channel migration opposite the leveed reach of the stream

Figure 7.4 presents a block diagram showing the major components of the Level II analysis. A technical discussion and analysis of each of these components follows.

7.3.1 Levee Embankment Height

The Corps of Engineers HEC-2 program was used to establish the water surface profile for the 100-year design flood of 46,785 cfs. This program was initially run in the subcritical mode using channel "n" values shown in Table 4.2, "For Depth and Flood Control". Since it was anticipated that the large flow associated with the 100-year event would produce flow velocities sufficiently high to create antidunes, a channel "n" value of 0.030 was selected from Table 4.2. This "n" value, coupled with the subcritical assumption for HEC-2, should produce a design water-surface profile that reflects "worst case" type conditions. The reader should remember, however, that the "n" values selected for overbank areas should be based on the best estimate of actual roughness in these areas rather than Table 4.2. Overbank areas typi-

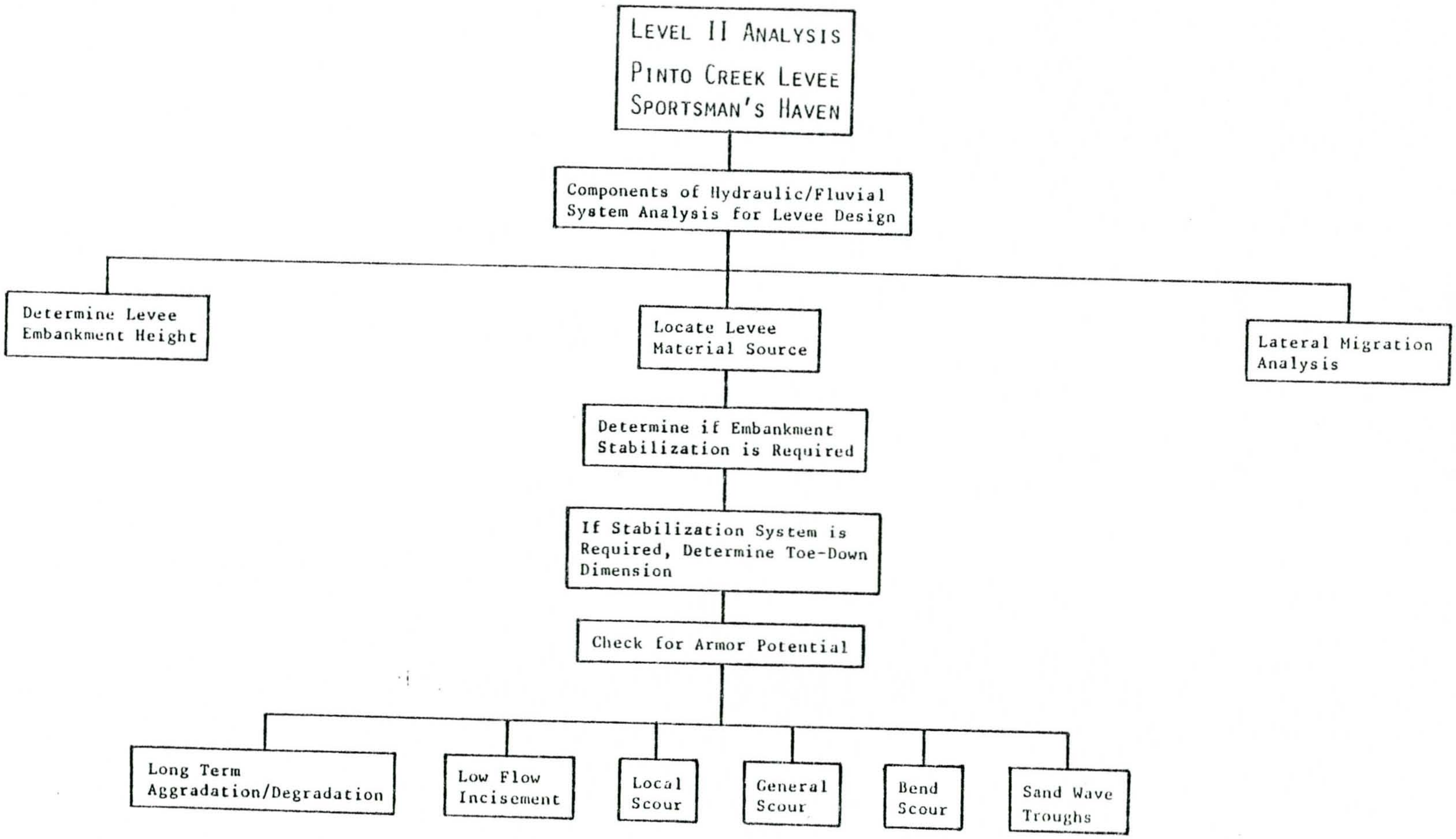


Figure 7.4 Block diagram of level II analysis.

cally exhibit dense vegetation or other obstructions to flow and thus have different hydraulic characteristics than the main channel. Overbank "n" values for the Pinto Creek analysis ranged from 0.045 to 0.085.

A review of the subcritical HEC-2 run using the "n" values listed above, indicated that critical depth was assumed at several cross sections. This leads one to suspect that the stream will probably be flowing at critical, or even supercritical at the peak of the design flood, which in turn lends credibility to the assumption of antidune formations in selecting a channel "n" value for the levee height analysis.

Once the design water surface profile is established under the criteria outlined above, the levee crest elevation is simply equal to the water surface elevation plus a freeboard dimension.

The freeboard dimension for this project is computed through application of only Equation 4.28b, since a soil-cement lining will be placed to the top of the levee embankment.

Freeboard for Earth Levee and Soil-Cement Lining

$$F.B._{TOT\ EMB/B.L.} = 1/2\ h_a + \Delta y_{se} + \Delta y_s + \Delta y_d + \Delta y_{agg}$$

Due to the absence of channel bends along the levee, Δy_{se} and Δy_s are both zero. Referring to Section 7.3.2.2.6, the reader will note that h_a varies from reach to reach.

Of the two remaining terms, Δy_d is zero due to the absence of a bridge pier along the levee, while Δy_{agg} will have to be assumed since the lack of pivot points prevented a quantitative assessment of long-term aggradation (see Section 7.3.2.2.2). A value of two feet will be assigned to Δy_{agg}. The freeboard is now calculated as follows:

1. Reach 2: $F.B._{TOT\ EMB/B.L.} = 1/2\ (3.5) + 2.0 = 3.75$ feet
2. Reach 3: $F.B._{TOT\ EMB/B.L.} = 1/2\ (4.3) + 2.0 = 4.15$ feet
3. Reach 4: $F.B._{TOT\ EMB/B.L.} = 1/2\ (2.1) + 2.0 = 3.05$ feet

The freeboard dimension for both the levee and soil-cement lining is measured from the water-surface profile generated by the <u>subcritical</u> HEC-2 analysis, since that condition will give the maximum expected flood elevation for the design event.

The freeboard dimension for the upstream segment of levee that extends through the right overbank to high ground will be based on both velocity head and FEMA requirements. The freeboard for this segment of levee will initially be estimated as twice the velocity head for the overbank at cross section 1409. Accordingly, the freeboard is calculated to be:

$$F.B_{TOT\ EMB/B.L.} = 2\left(\frac{v^2}{2g}\right) = 2\left(\frac{3.97^2}{64.4}\right) = 0.49 \text{ ft.}$$

Due to the low value for velocity head and the potential for wave run-up in this area, the overbank levee (including bank lining) will be assigned a freeboard dimension equal to minimum FEMA standards of 3.0 feet.

7.3.2 Levee Embankment Stabilization

7.3.2.1 Erosion of Embankment Material

The results of the subcritical HEC-2 analysis used to establish the levee height revealed channel velocities of 10 to 14 fps could be expected adjacent to the levee. If supercritical conditions were to occur as anticipated, the velocities would be even higher. Since the levee embankment should be designed to withstand the worst conditions expected during the design event, a supercritical HEC-2 run was made to establish an upper limit for a velocity profile through the project reach. Again, referring to Table 4.2, a channel "n" value of 0.025 was selected for the supercritical analysis used in the bank stability investigation. The end result of the lower "n" value and supercritical HEC-2 run was an average increase of about 1 fps in velocity at each cross section.

In this particular project, the supercritical analysis was somewhat academic in that the 10 to 14 fps velocities associated with the subcritical run already indicate that some form of bank protection will be needed to prevent erosion. This velocity range is well above that recommended for earth embankments in such publications as Hydraulic Design of Flood Control Channels, Corps of Engineers, 1970 and Open Channel Hydraulics, Chow, 1959. Cases may be encountered, however, where a subcritical water-surface profile may yield mean velocities that are low enough to be considered non-erosive. In these cases, the possibility of supercritical flow should be considered as a worst case condition for embankment erosion if there is a reasonable chance of it occurring. The ability of the levee or channel bank to withstand erosion should then be based on the supercritical velocities. The erodibility of an earth embankment can be determined by using methods such as the "allowable velocity" approach presented in Design of Open Channels, Technical Release No. 25, October, 1977, U.S.D.A., SCS. The reader is referred to TR-25 for detailed examples of this procedure.

7.3.2.2 Toe-Down Requirements For Stabilization System

Since the water-surface profile analysis indicated erosive velocities would exist during the design event, provisions must be made to protect the levee embankment. Soil cement was selected as one of the most economical and durable alternatives.

One of the most important aspects in designing the soil cement system was to determine the depth below the channel invert that the soil cement must be extended in order to prevent undercutting by vertical adjustments to the channel bed. Phenomena that must be considered in this analysis include:

1. armor potential
2. long-term degradation
3. low flow incisement
4. local scour
5. general scour
6. bend scour
7. sand wave troughs

The analysis of each of these phenomena (excluding bend scour) is discussed in detail in the following sub-sections. Bend scour was omitted due to the absence of bends along the levee.

7.3.2.2.1 Armor Potential

The first step in analyzing the vertical adjustment of a channel bed should focus on the potential for armoring to occur during the design event. If armoring were to occur, it may act as a control for the majority of the channel bed and prevent further downward movement except at areas of localized disturbance such as bridge piers or along the nose of a spur dike. If it can be guaranteed that armoring will uniformally occur across the channel during the design flow, the toe-down depth for a bank stabilization system may be reduced from that which may be required for a non-armored channel. If this condition (armoring) were to occur, the embankment stabilization system should be keyed into the armor layer by extending the toe-down 2' to 3' below the top of the predicted armor layer elevation.

For purposes of this example, the armor calculations will be based on the hydraulic characteristics of Reach 2. Similiar procedures would be applied to Reaches 3 and 4 to see if channel armoring is probable at those locations.

1. The hydraulic parameters used in the armor analysis of Reach 2 are listed as follows:

Q_{100} = 46,785 cfs (supercritical HEC-2 run)

Channel topwidth = 559 feet

Channel area = 2,859 feet^2

Energy slope = 0.0087 feet/feet

2. Using Shield's relationship (Sec. 5.3.4), compute the incipient motion particle size for the design event (100 year flood).

$$Dc = \frac{\tau}{0.047\ (\gamma_s - \gamma)}$$

Dc = sediment particle size (ft) at incipient motion

τ = shear stress on channel bottom (lb/ft^2)

γ_s = specific weight of sediment (assume 165 lb/ft^3)

γ = specific weight of water (62.4 lb/ft^3)

Shear stress will be computed using procedures in Section 4.3. Check width/depth ratio to see if γds approaches γRS:

$$\text{hydraulic depth} = \frac{A}{T} = \frac{2859}{559} = 5.11 \text{ feet}$$

$$\text{width/depth ratio} = \frac{b}{d} = \frac{559}{5.11} = 109\ (b{=}T)$$

Since $\frac{b}{d} > 10$, γds can be assumed equivalent to γRS and Figure 4.4 can be used to compute the maximum shear stress or tractive force on the channel bottom.

$$\text{From Figure 4.4b: } \frac{\tau_{max}}{\gamma ds} = 1.0$$

$$\tau_{max} = \gamma ds = (62.4)\ (5.11)\ (0.0087)$$

$$\tau_{max} = 2.77\ lb/ft^2$$

Substituting in Eq. 5.5: $$Dc = \frac{2.77}{0.047\ (165 - 62.4)}$$

$$Dc = 0.57\ ft\ or\ 175mm$$

3. Referring to Figure 7.5, which is an average gradation curve for the bed material in Pinto Creek, it can be seen there are no particle sizes in the bed as large as 175 mm. Therefore, it can be concluded that all bed particles will be moving during the peak of the 100-year event and that armoring will not occur.

7.3.2.2.2 Long-Term Aggradation/Degradation

A review of the Level I historical aerial photographs of Pinto Creek indicates the channel alignment through the study reach has not been stable. The observed lateral movement of the river has probably been accompanied by slope changes in the bed profile. An equilibrium slope analysis will be performed to estimate the long term response of the channel bed adjacent to the proposed levee.

For purposes of this example, an assumed stable sediment supply section was located approximately one mile upstream of the proposed levee. In locating a supply section, the engineer should look for a stable alignment on historical photos, a stable elevation on historical bed profiles, and field evidence of the river's impact on vegetation in the channel (exposed root systems, buried tree trunks, etc.).

It should be noted that the Highway 88 bridge lies between the selected equilibrium sediment supply section and the proposed levee alignment. If possible, it is preferable to not have any man-made obstructions within that reach of the channel between the equilibrium supply section and the reach of channel for which the hydraulic conditions that could restrict the amount of sediment being supplied to the study reach from the upstream supply section. For this design example, the bridge was assumed to have no impact in controlling the sediment supply to the leveed reach of Pinto Creek. This

COMPOSITE CURVE OF REPRESENTATIVE BED MATERIAL THRU STUDY REACH

PROJECT and STATE	SAMPLE LOCATION
Pinto Creek @ Sportsman's Haven, Arizona	various

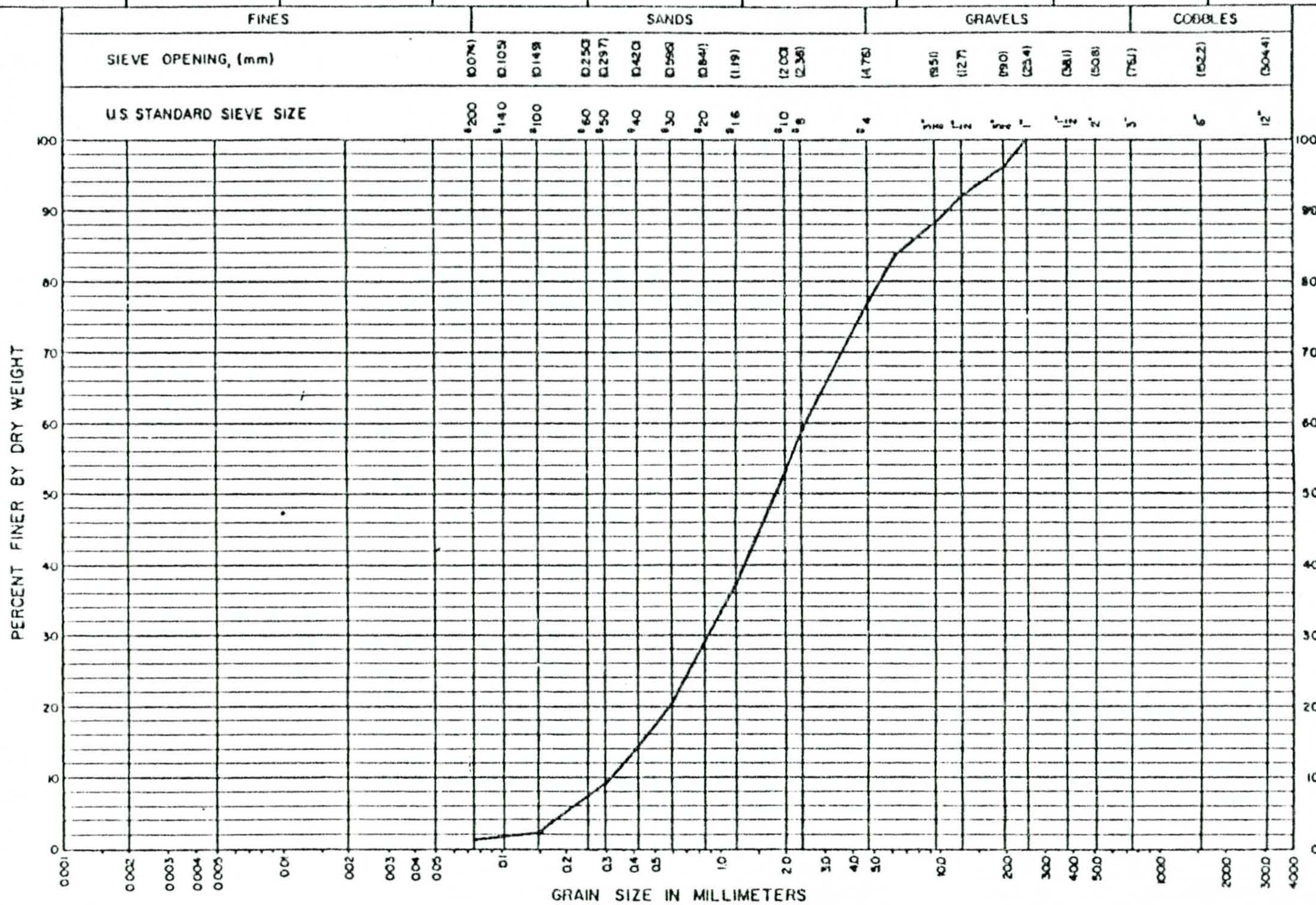
FIELD SAMPLE NO.	DEPTH	GEOLOGIC ORIGIN
	0.5-2.0'	River run- Channel bed material

TYPE OF SAMPLE	TESTED AT	APPROVED BY	DATE
Bulk			9/10/84

SYMBOL	DESCRIPTION
SW-SP	Poorly graded sands, gravelly sands

GRAIN SIZE DISTRIBUTION

D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	D_{MAX}	C_u	C_c
0.325	0.445	0.93	1.91	2.68	7.86	25.4	8.25	0.99



SPECIFIC GRAVITY (G_s)		ATTERBERG LIMITS						SOLUBLE SALTS	SHRINKAGE LIMIT	UNDISTURBED CONDITION	
		NATURAL MOISTURE		AIR DRY		OVEN DRY				MOISTURE	DRY UNIT WEIGHT
(-) #4	(+) #4	LL	PI	LL	PI	LL	PI	%	%	%	g/cc pcf

REMARKS:

Figure 7.5

assumption was based on the fact that the bridge opening was greater than the effective channel width at the upstream supply section and the fact that the channel bed slopes and n values at both locations were about equal. Under these conditions, the bridge opening should be able to pass the incoming sediment supply from the 10-year event without causing any reduction in transport capacity.

Pinto Creek was divided into three reaches along the proposed levee alignment. The reach boundaries were selected to provide segments of similiar hydraulic characteristics. An equilibrium slope will be computed for each reach. A set of detailed calculations will only be shown for Reach 2, the same procedures would be applied to the other two reaches.

1. Dominant discharge = 16,514 cfs. This value was selected on the results of a HEC-2 analysis for Reach 2 which showed the bankfull discharge was about 16,620 cfs. Since the 10-year event had a discharge of 16,514 cfs, it was selected as the dominant discharge for use in the equilibrium slope analysis.

2. Compute sediment supply.

   a. Equation 5.8b will be used to compute the transport capacity for the upstream supply section. A gradation coefficient and D_{50} particle size must be determined for use in this equation.

   b. A sieve analysis of bed material at the upstream supply section provided the following information:

      D_{50} = 1.19mm

      $D_{15.9}$ = 0.37mm

      $D_{84.1}$ = 4.67mm

$$\text{Gradation coefficient} = G = \frac{1}{2}\left(\frac{D_{84.1}}{D_{50}} + \frac{D_{50}}{D_{15.9}}\right)$$

$$G = \frac{1}{2}\left(\frac{4.67}{1.19} + \frac{1.19}{0.37}\right)$$

$$G = 3.57$$

   c. A cross section plot of the upstream supply section is shown in Figure 7.6. Although this section was judged to have a constant "n" value of 0.025, it is recommended the section be analyzed as having a main channel and an overbank. Even though the "n" value is constant, the hydraulic calculations for velocity and depth will differ if the entire section is considered to be "channel" versus analyzing the section as a channel with "overbank". The subdivision of the section into a channel and overbank should yield more accurate results since the velocity and hydraulic depth computations for each subdivision will be based on the calculated conveyance within each subdivision. This procedure eliminates the "weighted" velocity and hydraulic depth that would result from basing such calculations on the total conveyance for the entire cross section. In a 'benched' cross section, such as shown in Figure 7.6, this procedure allows computation of seperate sediment transport rates for the channel and overbank. When using a power relation such as Equation 5.8b, which is dependent upon velocity and hydraulic depth, it is prudent to consider this approach to insure that the velocity and depth parameters are truly representative of that portion of the cross section to which they are being applied.

Through a series of converging iterations with Manning's Equation, the following hydraulic parameters were determined for the supply section:

$Q = 16{,}514$ cfs
$S = 0.0097$
$n = 0.025$ (channel and overbank)

Depth = 6.58 feet

$Q_{CH} = 14{,}497$ cfs	$Q_{OB} = 2{,}017$ cfs
$V_{CH} = 19.50$ fps	$V_{OB} = 9.71$ fps
$A_{CH} = 743.3$ ft.^{2}	$A_{OB} = 207.6$ ft.^{2}

d. Substitute data from b. and c. into Eq. 5.8b:

$$q_s = 0.0064 \; \frac{n^{1.77} \, V^{4.32} \, G^{0.45}}{Y_h^{\;0.30} \, D_{50}^{\;0.61}}$$

<u>For the "channel" section:</u>

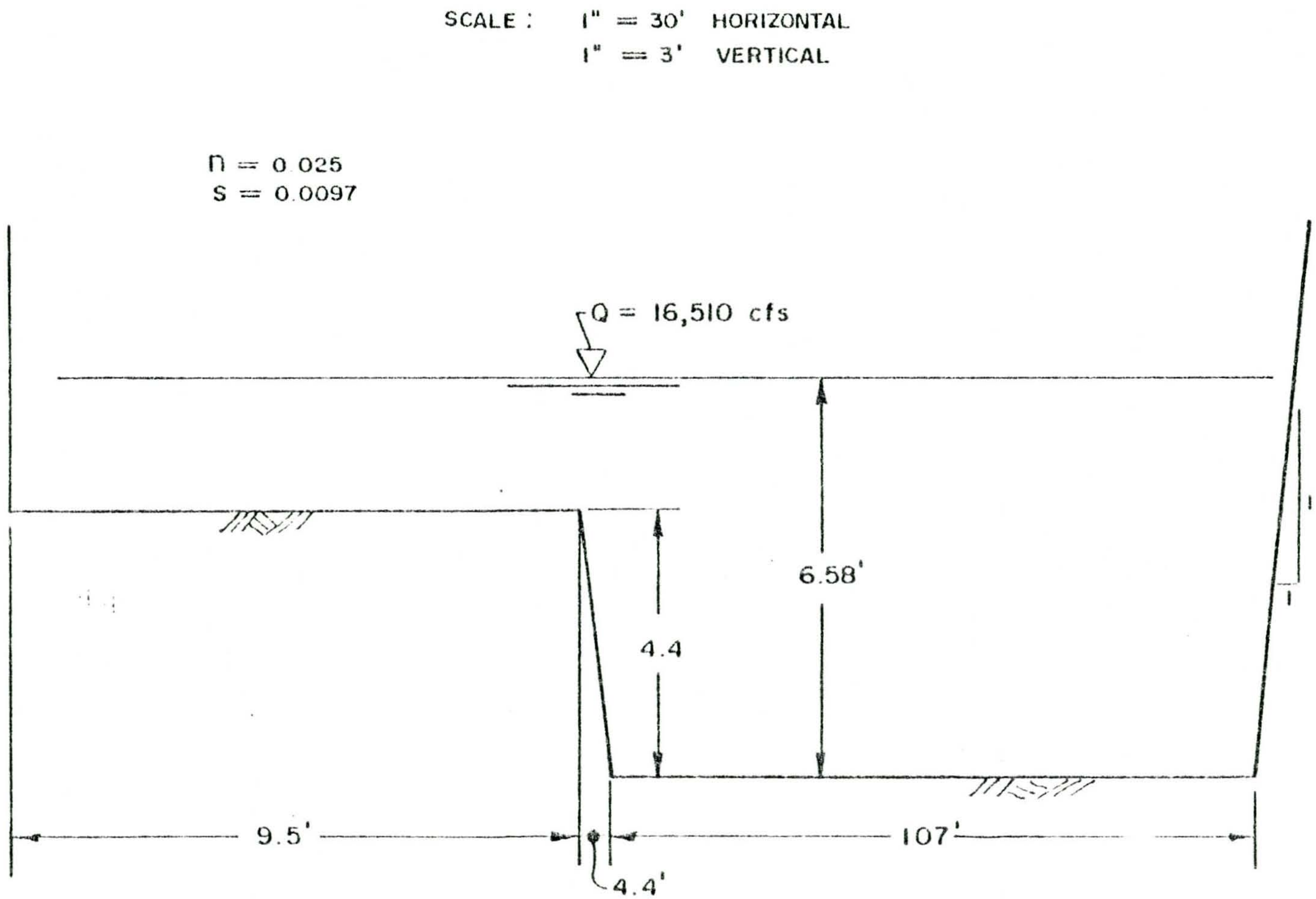
average flow width = 117 feet

therefore; $Y_h = \frac{A}{W} = \frac{743.3}{117}$

$$Y_h = 6.35 \text{ ft}$$

$$q_{s_{CH}} = 0.0064 \left[\frac{(0.025)^{1.77}(19.50)^{4.32}(3.57)^{0.45}}{(6.35)^{0.30}\,(1.19)^{0.61}} \right]$$

Figure 7.6. CROSS SECTION GEOMETRY FOR UPSTREAM SEDIMENT SUPPLY SECTION USED IN EQUILIBRIUM SLOPE ANALYSIS



$$q_{s_{CH}} = 3.20 \text{ cfs/ft}$$

For a width of 117 ft; $Q_{s_{CH}} = q_{s_{CH}} \times W_{CH}$

$$Q_{s_{CH}} = (3.20)\ (117)$$

$$Q_{s_{CH}} = 374.5 \text{ cfs}$$

<u>For the overbank section</u>:

average flow width = 97 feet

therefore; $Y_h = \frac{207.6}{97}$

$$Y_h = 2.14 \text{ ft}$$

$$q_{s_{OB}} = 0.0064 \left[\frac{(0.025)^{1.77}\ (9.71)^{4.32}\ (3.57)^{0.45}}{(2.14)^{0.30}\ (1.19)^{0.61}} \right]$$

$$q_{s_{OB}} = 0.218 \text{ cfs/ft}$$

For a width of 97 feet, $Q_{s_{OB}} = (0.218)\ (97)$

$$Q_{s_{OB}} = 21.2 \text{ cfs}$$

Total sediment transport = $Q_{s_{TOT}} = Q_{s_{CH}} + Q_{s_{OB}}$

$$Q_{s_{TOT}} = 374.5 + 21.2$$

$$Q_{s_{TOT}} = 395.7 \text{ cfs}$$

This value (395.7 cfs) will be used as the sediment supply for all downstream reaches for which the equilibrium slope analysis is performed.

3. Compute equilibrium slope. An iteration procedure is now employed to compute the sediment transport capacity for a cross section typical of Reach 2. Manning's Equation is used to compute V and Y_h, while Equation 5.8b is used to compute transport capacity. The bed slope value is adjusted between iterations until the transport rate equals the supply rate (395.7 cfs). All calculations are based on the

following data:

$Q = 16{,}514$ cfs
$n = 0.025$
$D_{50} = 1.91$mm
$G = 3.74$
$W = 552$ feet
q_s computed using Eq. 5.8b

A series of iterations resulting in an equilibrium slope of 0.0187 is presented in Table 7.1.

The same procedures outlined in Steps 1 through 3 are also applied to the equilibrium slope analysis for Reaches 3 and 4. A summary of the predicted equilibrium slopes for all three reaches adjacent to the proposed levee is shown below:

Reach	Existing Slope	Equilibrium Slope
2	0.0103	0.0187
3	0.0082	0.0195
4	0.0083	0.0310

The existing slope of Reach 2 is 0.0103 ft/ft. A review of the calculations in Table 7.1 indicates the existing sediment-transport rate for Reach 2 is considerably less than the incoming supply (395.7 cfs). The existing transport rates for Reaches 3 and 4, 112.9 cfs and 60.8 cfs, respectively, are also considerably less than the estimated sediment supply of 395.7 cfs. Due to this relatively large difference between the transport capacities of Reaches 2, 3, and 4 compared to the upstream supply reach, the engineer might suspect that the chosen equilibrium supply section is not really in equilibrium. Unless there have been significant man-made changes in the river system during recent years (refer to Level I historical data), it is unlikely that such large differences would exist between sediment transport rates for river cross sections that are within a mile of each other.

In consideration of the results obtained from the previous equilibrium slope calculations, the engineer should re-evaluate his selection of an equilibrium slope cross section to insure that it has truly been a long term, stable cross section. Additional field inspections might reveal the existance of a better supply section for the equilibrium slope analysis.

Table 7.1. Equilibrium Slope Calculations For Reach 2.

Slope ft/ft	V (fps)	Y_h (ft)	q_s (cfs/ft)	W (ft)	Q_s (cfs)
0.0103	11.38	2.63	0.3115	552	171.9
0.0170	13.26	2.24	0.63	552	352.3
0.0190	13.69	2.18	0.732	552	404.2
0.0187	13.63	2.19	0.717	552	396.0

Since 396.0 cfs is approximately equal to the supply rate of 395.7 cfs, the calculations are terminated at this point and 0.0187 is accepted as the equilibrium slope.

For purposes of completing this example, it will be assumed that the original equilibrium slope calculations are valid. Accordingly, one could expect sediment deposits will begin to occur in the upstream portion of Reach 2 in an effort to steepen the bed slope to 0.0187 ft/ft for the remaining downstream portion of this reach.

Unfortunately, inspections of this area failed to reveal any natural or manmade controls which could be used as a pivot point for the computed equilibrium slopes. Under these circumstances, the equilibrium slope analysis can only be applied in a qualitative sense, i.e., Reaches 2, 3 and 4 should aggrade over the long term. Response to such aggradation might be lateral migration, channel braiding, channel widening, or a combination of these phenomena.

7.3.2.2.3 Low Flow Incisement

Field inspections of the study reach provided an opportunity to check the stream for low flow incisement. A low flow channel on the order of 2' to 2.5' was observed during this visit.

The proposed levee improvements do not include any modifications to the channel bed which would eliminate the existing low flow channel. In the absence of channel improvements, the invert of the existing low flow channel will be used as a base elevation from which all scour, degradation, etc. dimensions will be measured. This decision is justified on the probability that the existing low flow channel will migrate across the stream bed and ultimately be in contact with any point along the levee toe.

Had channelization been part of the proposed plan, it would have been prudent to add 2' to the toe-down depth for the soil cement since a new low flow channel would probably re-form through the channelized reach of the stream. This low flow depth can usually be based on the dimensions of low flow channels observed prior to construction of channel improvements. Any break in grade with the natural channel invert at the upstream and downstream end of the channelized reach should also be considered.

7.3.2.2.4 Local Scour

Potential for local scour occurs at the upstream end of the proposed levee where the alignment turns easterly to tie into high ground and becomes nearly perpendicular to the direction of flow. This will obstruct the right overbank flow and divert it westerly where it will merge into the main channel at the upstream corner of the levee. As the overbank flow passes the levee corner and re-enters the main channel, the velocity will increase and generate a scour pocket around the levee toe. The approximate dimensions of this scour hole must be considered in determining the soil cement toe down depth at this location.

Since the soil cement embankment will have sloping sides, either Equation 5.17b or 5.18 will be used, the final selection being determined by the ratio $\frac{a}{y}$. The analysis proceeds as follows:

1. Cross section 1409 (see Figure 7.7) will be used to determine the hydraulic data needed for this analysis. This cross section is chosen because it represents average right overbank flow conditions prior to being intercepted by the levee.

2. From the supercritical HEC-2 run for the 100-year event, the following data is obtained:

XSEC	VROB (fps)	AROB (ft^2)	$Width_{ROB}$ (ft)	Y_h (ft)
1409	3.97	1232	560	2.20

3. Compute $\frac{a}{y}$. The embankment length, a, will be measured by taking the projected length of the right overbank levee perpendicular to flow (see Figure 7.7). In this particular case, "a" is equal to the width of the right overbank at cross section 1409. "Y" will be computed as the hydraulic depth of the right overbank at cross section 1409. Using the above definitions:

   a = 560 feet

   Y_h = 2.20 feet

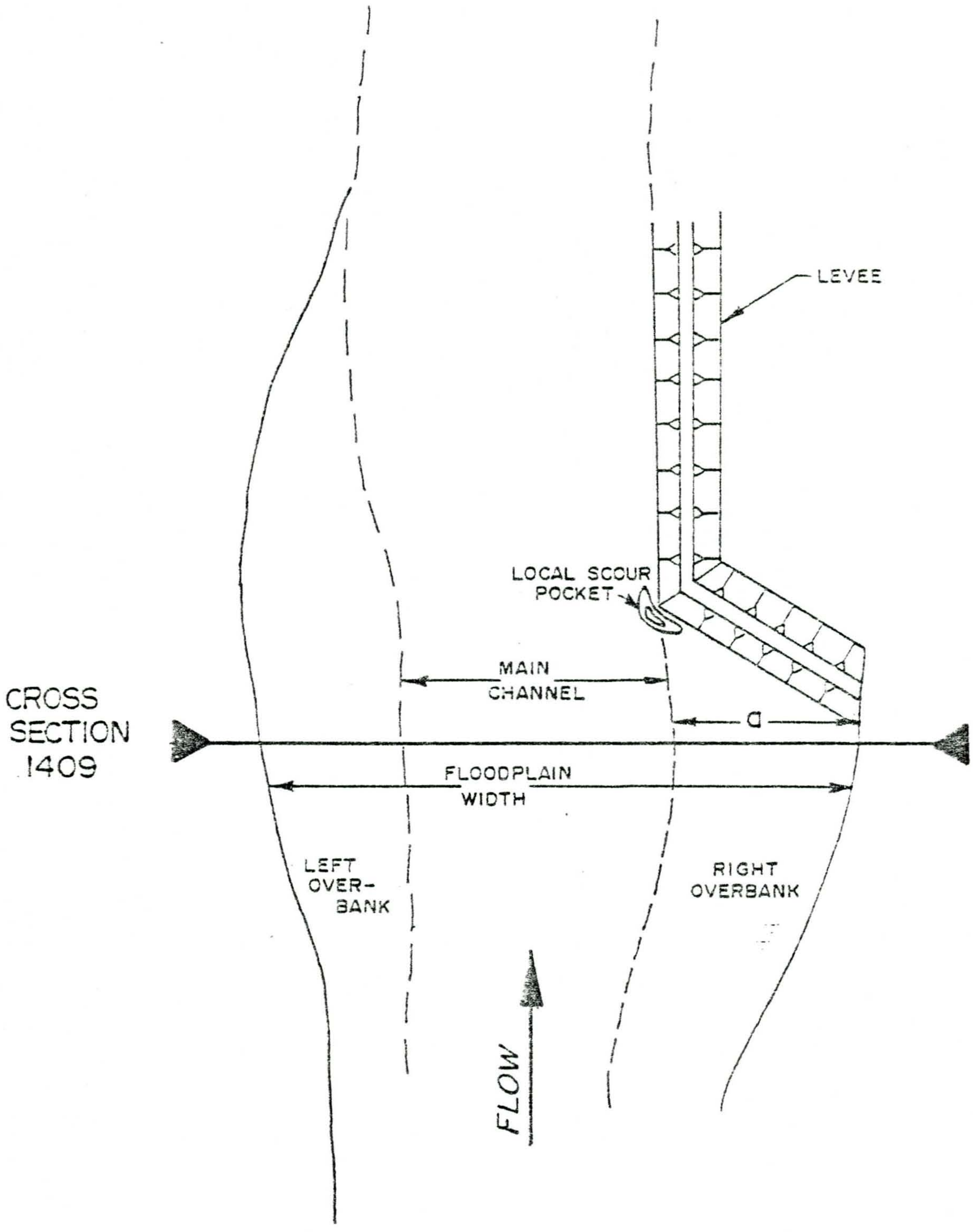


Figure 7.7 Plan view of levee alignment analyzed in local scour evaluation.

therefore; $\frac{a}{y} = \frac{560}{2.20} = 254$

Since $\frac{a}{y}$ is greater than 25, Equation 5.18 will be used.

4. Compute embankment scour.

$$\frac{\Delta Z_{1s}}{Y} = 4\ Fr^{0.33}$$

$$\text{where } Fr = \frac{V}{\sqrt{g\ Y_h}} = \frac{3.97}{\sqrt{(32.2)\ (2.20)}}$$

$$Fr = 0.47$$

$$\text{therefore; } \Delta Z_{1s} = 4\ Fr^{0.33} Y$$

$$\Delta Z_{1s} = (4)\ (0.47)^{0.33} (2.20)$$

$$\Delta Z_{1s} = 6.86 \text{ ft}$$

This vertical scour depth will be assumed to extend 50 feet on either side of the levee corner. Assuming the bed material at this location has an angle of repose of 30° (typical for sands and gravels), the sides of the scour hole will be assumed to slope along the levee toe at an angle of 15°. This procedure provides a safety factor of 2.0 as discussed in Section 5.3.10.

The remainder of the right overbank levee segment should also incorporate some toe down for the soil cement to prevent possible erosion that may occur as the overbank flow impinges on the levee and is diverted westerly. On the basis of engineering judgement, the remainder of this segment of the overbank levee will be toed down a distance slightly greater than the hydraulic depth (2.20 feet) at cross section 1409. A toe-down depth of 3.0 feet was chosen.

7.3.2.2.5 General Scour

Since the river cross section geometry is not constant through the three reaches along the proposed levee alignment, the sediment transport characteristics will vary from reach to reach. These variable transport characteristics will influence the amount of sediment being delivered from reach to reach for a given flood event. Any differences between sediment supply to a reach and sediment transport capacity within that reach will cause either scour or deposition during the flow event being modeled. These short-term bed changes can be evaluated using the principle of sediment continuity. Any lowering of the bed that occurs as a result of this phenomenum is considered a type of general scour and needs to be considered in the design of the levee toe-down dimension.

General scour of this type is most accurately analyzed at Level III using a moveable bed computer model such as HEC-2SR. However, a Level II approximation can be achieved using rigid-bed hydraulic and sediment-transport calculations to estimate the imbalance between sediment-transport capacity and sediment supply between adjacent reaches. The net imbalance within a reach can be converted to a volume which in turn is converted to a channel bed depth adjustment.

Since the 100-year flood is the design standard for this levee project, this event was used in the following analysis. The data requirements and calculation sequence follows.

1. Discretize the 100-year flood hydrograph at 1 hour intervals (see Figure 7.8).

2. Develop sediment transport rating curves for each of the four reaches. The hydraulic data required for these calculations can be taken from the HEC-2 runs for the 2-, 5-, 10-, 25-, 50-, and 100-year peak discharge values. Since only one rating curve is developed for each reach, average hydraulic parameters which are characteristic of each reach must be used. For purposes of this example, the velocity, area, and topwidths from the HEC-2 analyses were averaged for all the cross sections in each reach. These average values were segregated by channel and overbank partitions in order that sediment transport calculations could be performed within each of these partitions. The total transport rate for each reach (for a given discharge) is the sum of the transport rate for the channel plus the transport rate for the overbank(s).

   The end product of this step is a curve representing a plot of Q_s vs. Q for the range of water discharge values being evaluated. Figure 7.9 illustrates the rating curve developed for Reach 2.

3. Route design hydrograph through the study reach. The purpose of this step is to determine the amount of sediment transported through each reach during the passage of a given hydrograph. This is accomplished in a tabular format as illustrated in Table 7.2. The sediment transport for each step of the hydrograph is read from the sediment transport rating curve for each reach. The transport rates for each time interval are then summed to get a total transport rate for each

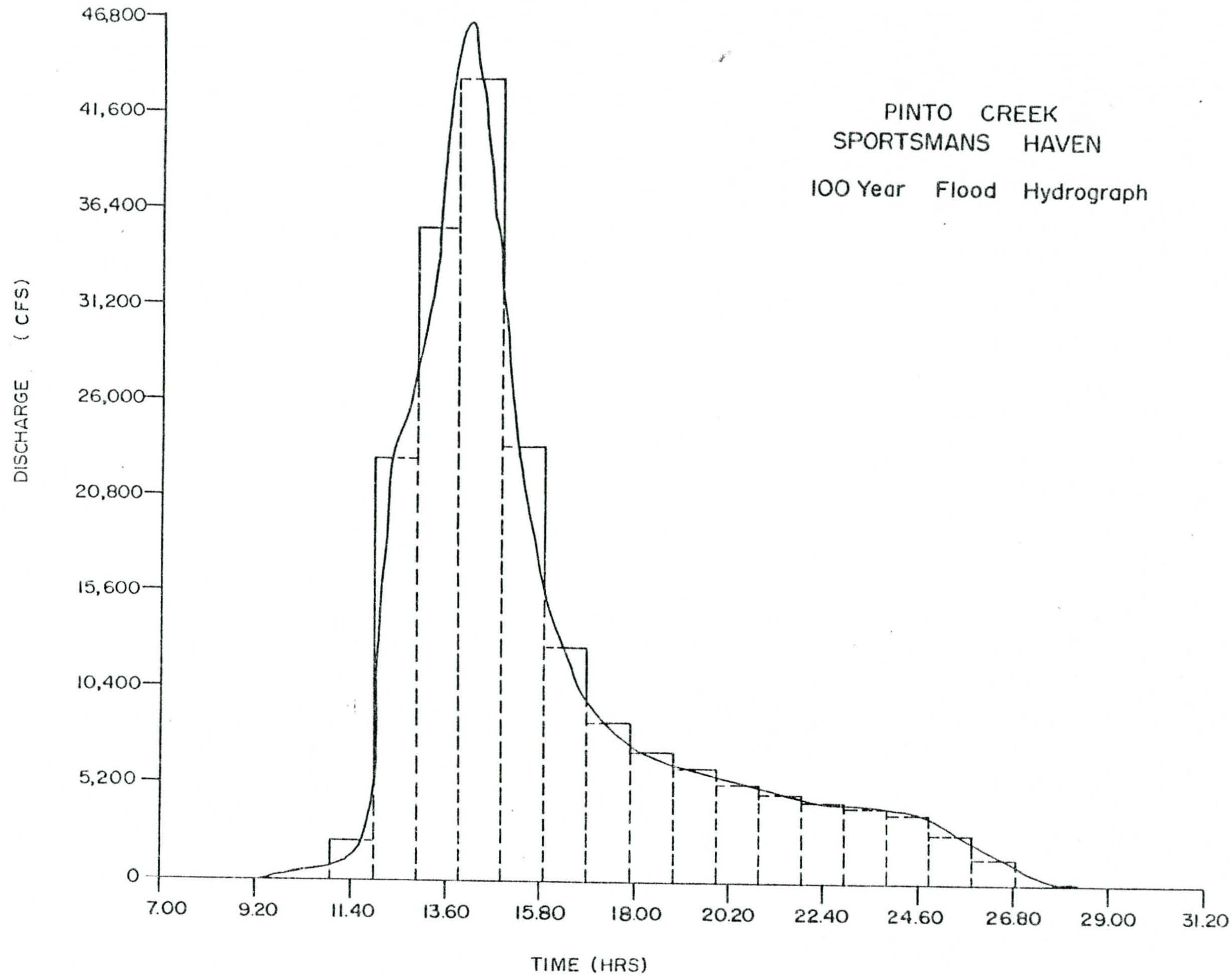


Figure 7.8 Discretized 100-year flood hydrograph.

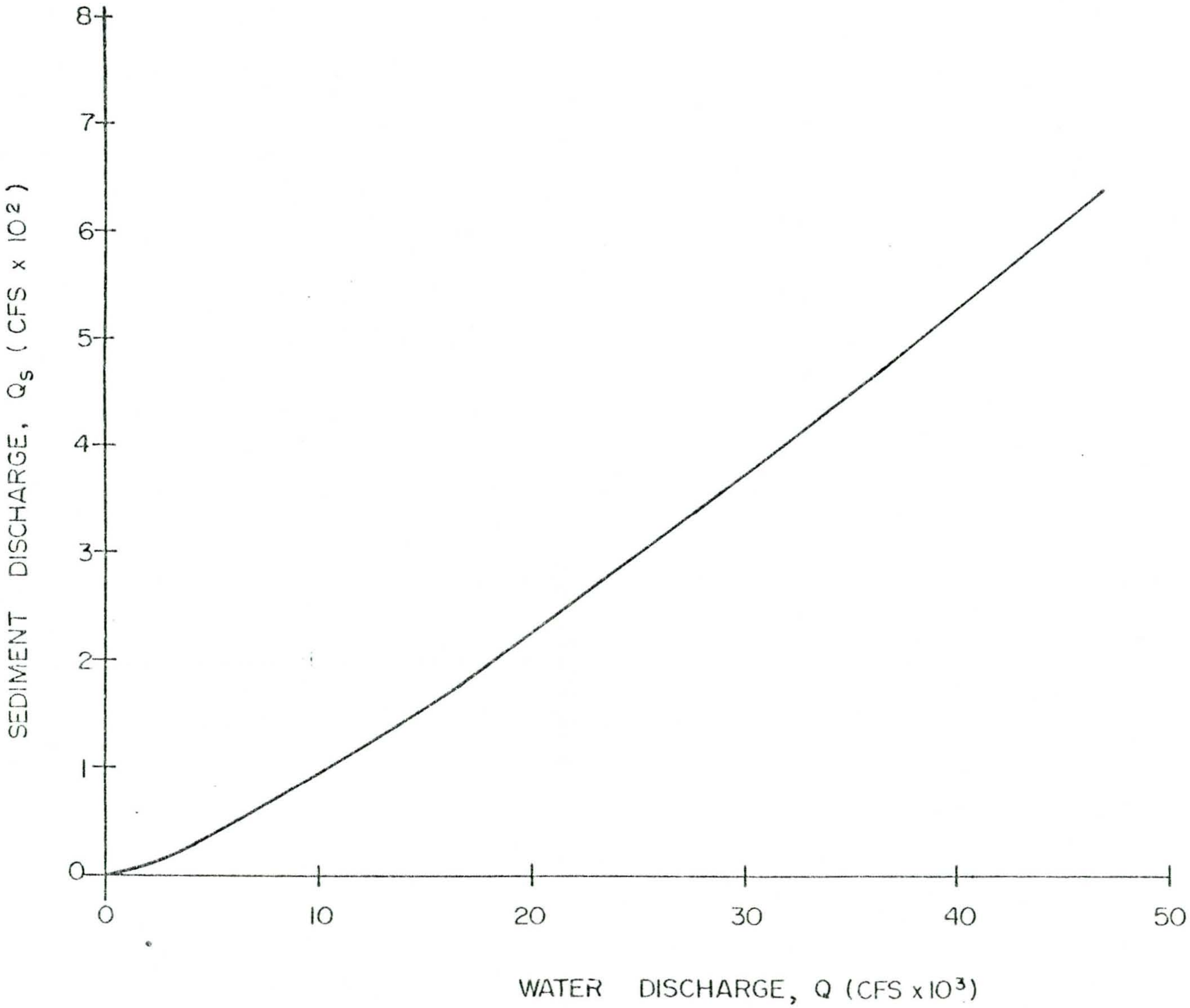


Figure 7.9 Sediment transpo[illegible] rating curve - Reach 2, Pinto Creek at Sportsman's Haven.

Table 7.2. General Scour Analysis Using Sediment Continuity, 100-year Event.

Time (Hours)	Q (cfs)	Q_s (cfs) Reach 1	Reach 2	Reach 3	Reach 4
10.96 - 11.96	2,080	12	15	12	14
11.96 - 12.96	22,932	215	271	338	225
12.96 - 13.96	35,412	372	456	694	172
13.96 - 14.96	43,576	484	585	954	462
14.96 - 15.96	23,504	222	280	355	232
15.96 - 16.96	12,688	99	126	114	121
16.96 - 17.96	8,580	61	78	71	80
17.96 - 18.96	6,968	48	61	54	62
18.96 - 19.96	5,200	33	41	36	42
19.96 - 20.96	5,356	34	43	37	43
20.96 - 21.96	4,784	29	37	31	37
21.96 - 22.96	4,316	26	31	26	31
22.96 - 23.96	4,056	23	29	24	28
23.96 - 24.96	3,640	21	25	21	25
24.96 - 25.96	2,600	15	18	15	18
25.96 - 26.96	1,300	7	9	7	9
	TOTAL:	1,702	2,104	2,790	1,804
	DIFFERENCE:	-402	-686	986	

reach for the duration of the hydrograph. The difference in transport rates between adjacent reaches represents the sediment imbalance that must be satisfied through scour or deposition.

4. The transport rate imbalance between reaches must be converted to sediment volumes before channel bed adjustments can be computed. The volume conversion and sediment distribution through each reach is most easily accomplished in a tabular format. The following information is needed for this step:

   a. Difference in sediment transport rates (ΔQ_s) between adjacent reaches.

   b. Time interval for the discretized hydrograph (Δt).

   c. Channel and overbank reach lengths.

   d. Channel and overbank widths.

   e. Channel and overbank conveyance values.

   f. Sediment porosity.

   The procedure consists of converting the ΔQ_s values to volumes by multiplying ΔQ_s (cfs) by the discretization interval, Δt (hr) and a seconds to hours conversion factor (3,600). This calculation yields an unbulked sediment volume in cubic feet. This volume is then distributed through the channel and overbanks in proportion to the conveyance ratios for each of these partitions. The proportioned volume for each partition is then uniformally distributed by dividing the volume by the product of the partition length and width. The resultant answer will represent vertical bed movement in terms of unbulked sediment. To correct for sediment bulking, this answer must be divided by (1-n), where n is sediment porosity.

The general scour calculations, using the sediment continuity principle and rigid-bed hydraulics, are summarized in Table 7.3. This analysis predicts approximately 3 to 5 feet of general scour for Reaches 2 and 3 but nearly 6 feet of aggradation for Reach 4. This illustrates the dynamic changes that can occur in a riverbed during a major flood. The engineer must remember that these are net changes that would be expected at the <u>end</u> of the hydrograph. Transport imbalances may occur <u>during</u> the hydrograph that produce temporary scour that is more severe than the net change observed at the end of the flood. At Level II, this additional scour potential is accounted for by applying a factor of safety to the sum of all scour components.

Table 7.3. Pinto Creek 100-year Sediment Continuity Analysis.

(1) Reach	(2) ΔQ_s (cfs)	(3) Δt (hr)	(4) Partition	(5) Length (feet)	(6) Width (feet)	(7) $K_{PARTITION}$	(8) K_{TOTAL}	(9) $\frac{K_P}{K_T}$	(10) Vertical Movement (feet) n = 0.4 Unbulked	Bulked
			LOB	1,190	54	12,759		0.025	-0.56	-0.93
2	-402	1.0	CH	1,190	547	505,858	518,613	0.975	-2.17	-3.62
			ROB	N/A	0	0		0	0	0
			LOB	1,170	427	105,289		0.256	-1.27	-2.12
3	-686	1.0	CH	1,190	484	306,275	411,569	0.744	-3.19	-5.32
			ROB	N/A	0	0		0	0	0
			LOB	1,320	672	206,149		0.437	1.75	2.92
4	986	1.0	CH	1,190	487	266,009	472,158	0.563	3.45	5.75
			ROB	N/A	0	0		0	0	0

$$\text{Vertical Movement} = \frac{\Delta\ Q_s\ \text{ft}^3}{\text{sec.}} \times \frac{\Delta t\ \text{hr.}}{1} \times \frac{3600\ \text{sec.}}{\text{hour}} \times \frac{1}{(\text{length}) \times (\text{width})\ \text{ft}^2}\ \frac{K_p}{K_T}$$

or/ col. 10 = col. 2 x col. 3 x 3600 ÷ [(col. 5) x (col. 6)] x col. 9

Δt = time interval for discretized hydrograph

K = conveyance

7.3.2.2.6 Sand Wave Troughs

The results of the HEC-2 hydraulic analysis for this project indicate that supercritical flow will probably occur in Pinto Creek during the 100-year flood. This condition will lead to the formation of antidunes in a sand bed channel. Although the supercritical flow needed for the formation of antidunes dunes would normally be expected to occur near the center of the channel cross section, the formation of sand or gravel bars may create a meandering filament of high velocity flow. If this filament were to be diverted across the channel and begin flowing next to the levee, antidunes could form along the levee too. Under these conditions, the troughs created by these bed forms could undercut the soil-cement embankment and cause a levee failure. To protect against this type of failure, the soil cement should be extended a sufficient distance below the channel bed to prevent undercutting by antidunes.

The estimated depth of antidune troughs is computed by using Equation 4.25:

$$h_a = 0.027\ V^2$$

The computed value of h_a represents the distance from the crest to the trough of an antidune (see Figure 4.7). Accordingly, h_a must be divided by two to get the trough depth below the original bed elevation. When using this equation, the engineer must remember that, in reality, h_a can never exceed the actual depth of flow (Y_0). Therefore, the trough depth calculations must be compared to the actual depths of flow expected in a channel. If the computed value of h_a exceeds Y_0, h_a should be assumed equal to Y_0.

The proper value to use for V in Equation 4.25 is the maximum velocity expected within the channel cross section, rather than a weighted channel average of the velocity. This may entail subdividing a channel section into vertical strips and computing conveyance values for each, which can then be combined with known energy slopes from a HEC-2 analysis to derive a velocity for each strip.

If there is a wide variation in maximum velocity from one cross section to the next, the engineer should consider seperate antidune calculations for different reaches of the river. The squared velocity term in Equation 4.25 makes this calculation very sensitive to changes in this parameter.

For this project, average maximum velocities (based on the maximum velocity at each cross section within a reach) were determined for Reaches 2, 3, and 4 and trough depth calculations were performed for each reach. These calculations, which employed Equation 4.25, are summarized in Table 7.4.

7.3.3 Lateral Migration

Changes to the boundaries of river systems occur in the horizontal direction as well as the vertical. Quite often, horizontal movement is induced or aggravated by the construction of man-made improvements within a floodplain. In the case of the Pinto Creek project, the installation of an armored levee along the east bank of the river may accelerate erosion along the west bank. The soil cement embankment will eliminate a potential sediment source along the east bank that may have historically been required to help satisfy deficits between upstream supply and transport capacity within a given reach.

Analysis of lateral migration potential is perhaps most accurately determined through a Level I review of historical aerial photographs of the river system. The proposed installation of an armored levee, however, introduces a variable that is not reflected in the channel movement observed in historical photos.

As discussed in Section 5.3.9, quantification of lateral migration in a disturbed river system can be pursued through the application of the sediment continuity concept. If a sediment deficit is found to exist within a given reach as the result of routing the design hydrograph through the reach, a worst case condition can be established by assuming the sediment deficit is satsified by using one bank of the reach as the sole, local sediment supply. The sediment deficit could either be uniformly distributed along the entire bank line or be concentrated in a location where a bend might form or is already in existence. The following analysis for Pinto Creek will illustrate both cases.

1. Use sediment continuity and assume all sediment deficits will be satisfied by erosion of material from the west bank. The analysis will use data from the general scour analysis (Section 7.3.2.2.5) for the 100-year flood.

Table 7.4. Calculation of Antidune Trough Depths.

Reach	Average Maximum Velocity (fps)	h_a (feet)[1]	Trough Depth (feet) ($1/2 \times h_a$)
2	16.18	7.07	3.5
3	17.92	8.67	4.3
4	12.56	4.26	2.1

[1]$h_a = 0.027\ V^2$ (Equation 4.25).

2. Reach 2 - This reach will be armored along the east bank, as a result of the proposed soil-cement levee, and presently consists of natural rock along the west bank. As a result, no lateral channel movement is expected to occur in this reach.

3. Reach 3 - From the sediment continuity analysis, this reach has a total sediment supply deficit of 686 cfs (see Table 7.2). This value is converted to a volume as follows:

$$\text{Vol.} = \Delta Q_s \times \Delta t \times 3{,}600 \text{ sec./hr.}$$

$$\text{Vol.} = (686)\ (1 \text{ hour})\ (3{,}600)$$

$$\text{Vol.} = 2{,}469{,}600 \text{ feet}^3$$

Correct for bulking (assuming a porosity of n = 0.4),

$$\text{Vol.} = \frac{2{,}469{,}600}{0.6} = 4{,}116{,}000 \text{ feet}^3$$

The average west bank height (H) through Reach 3 is six feet, while the bank length is 1,170 feet. Assuming uniform erosion, the lateral bank movement is computed as follows:

$$\Delta W = \frac{\text{Volume Sed. Def}}{\text{HWB} \times \text{LWB}}$$

$$\Delta W = \frac{4{,}116{,}000}{(6)\ (1{,}170)}$$

$$\Delta W = 586 \text{ feet}$$

An alternative to the uniform erosion approach is to assume the erosion will occur as a semi-circular bend. The volume of erosion is computed as:

$$\text{Volume}_{\text{Bank}} = \frac{1}{2} \pi r^2 H$$

Assuming the volume of bank erosion equals the volume of sediment deficit (i.e., bed and bank materials are similar); or

$$\text{Volume}_{\text{Bank}} = \text{Volume}_{\text{Sed. Def}}$$

we can solve for the radius of the semi-circle as follows:

$$r = \left(\frac{\text{Vol.}_{\text{Sed. Ded.}}}{1/2\ \pi\ H}\right)^{1/2}$$

$$r = \left[\frac{4,116,000}{(.5)(\pi)(H)}\right]^{1/2}$$

$$r = 661 \text{ feet}$$

The semi-circular erosion pattern gives a worst-case condition for this reach. Selection of the dimensions for an erosion buffer zone along the west bank is a matter of experience and engineering judgement, and should not be based solely on the results of a quantitative analysis such as that presented above. Cases may arise where it would be more economical to construct some type of structural measure to prevent erosion, rather than purchasing the right-of-way for an erosion buffer zone.

For this project, an erosion buffer zone was selected as the preferred alternative for the west bank. The west bank is undeveloped property and can be purchased at a low cost. Based on the results of the quantitative lateral migration analysis, a review of historical photos dating back to 1947, and the topography along the west bank, a variable width buffer zone was recommended for the west bank through Reach 3.

Due to the reconnaissance level nature of the Department of Water Resource study, subsurface geological data was not available for the stream bank opposite the proposed levee. However, a rock formation is visible along the west bank through the majority of Reach 2. The topography along the west bank suggests that this rock formation continues under the surface and probably constitutes the steep ridgeline along the west bank of Reach 3. On this assumption, the buffer zone for Reach 3 will be taken as that area from the west edge of the 100-year floodplain to the base of the steep ridge (see Figure 7.1). This width will vary from 200' to 300', and is within the limits derived from the quantitative analysis. Obviously, the geological assumptions used in this analysis would have to be verified prior to a final delineation and acquisition of the buffer zone.

4. Reach 4 - The sediment continuity analysis indicates this reach will receive more sediment than it is capable of transporting. As a result, bank erosion due to insufficient sediment supply should not occur. However, as the bed aggrades, the channel geometry could change and cause the main channel to shift westerly and possibly attack the western bank with high velocity flow. This is a very dynamic process which is difficult to quantify. Estimating the amount of lateral erosion in aggrading reaches is a matter of engineering judgement. Again, as for Reach 3, topographic features and historical photos were used in establishing a realistic buffer zone through Reach 4. This zone is an extension of the one through Reach 3 and essentially follows the base of another steep ridgeline which is suspected to be overlying rock along the northern half of the reach (see Figure 7.1 for buffer zone limits).

Although Figure 7.1 does not extend far enough downstream to show this feature, Pinto Creek makes about a 45° bend to the east after leaving Reach 4. In the absence of any visible topographic or geologic erosion resistant features in this downstream area, there is a good chance that some accelerated erosion could occur on the outside of this bend. This erosion process may be further accelerated as a result of the straightened alignment of the river from the Highway 88 bridge through Reach 4. The proposed levee tends to concentrate the flood water in a straight line that is directed into this bend. Prior to construction of the proposed levee, the water spread out through Sportsman's Haven and did not launch such a concentrated attack at the entrance to the bend.

As part of the final design phase of this project, it would be recommended that a detailed analysis be made of this problem in order that mitigation measures might be taken if the damage potential was found to be severe and directly related to construction of the upstream levee system.

The quantitative assessment of lateral migration presented in this section is based on a single flood event. Realizing that lateral migration is a continual process over a long period of time, some safety factor, say 2.0, could be applied to the quantitative calculations to establish a long-term limit. It must be emphasized, however, that quantitative calculations should only be used with considerable engineering judgement and an appreciation of historical events and physical constraints such as topography and geology.

7.4 Summary and Conclusions

Based on the preceding analyses, we are now prepared to establish the critical design dimensions for the proposed levee system.

1. Levee Crest Profile
   The crest of the levee will parallel the subcritical water-surface profile for the 100-year flood. As shown below, the top of the levee embankment and the soil-cement bank protection will be elevated an equal distance above this water-surface profile. These freeboard dimensions are minimum values and may be increased slightly during design to eliminate numerous grade breaks during actual levee construction.

Reach	Freeboard Distance Above Design Water Surface Profile (feet)	
	Levee Crest	Soil-Cement Lining
Upstream Right Overbank	3.0	3.0
2	3.8	3.8
3	4.2	4.2
4	3.1	3.1

2. Requirements For Bank Stabilization
The need for bank stabilization to prevent erosion of the earth levee was discussed in Section 7.3.2.1. For this project, a soil-cement blanket is proposed along the stream-side face of the levee to prevent erosion.

3. Toe-Down Requirements For Soil-Cement Embankment
The soil cement must be extended far enough below existing ground elevation so as to prevent undermining by the multiple scouring processes that occur on both a short and long-term basis. This toe-down dimension is determined as the sum of all the vertical channel adjustments that were analyzed in Section 7.3.2.2. A summary of the recommended toe-down depths for specific reaches of the levee is presented in Table 7.5.

4. Lateral Channel Migration
Armoring of the proposed east bank levee through an application of soil cement may accelerate erosion along the opposite bank of the stream. As a precautionary measure, an erosion buffer zone is recommended along the west bank of Pinto Creek. The buffer zone dimensions are summarized as follows:

Reach	Buffer Zone (feet)
2	0 (natural rock)
3	200 - 300
4	130 - 350

For this project, the buffer zone will be measured from the west edge of the 100-year flood plain. Unique circumstances on other projects might dictate that such buffer zones be measured from different reference points.

Table 7.5. Summary of Soil-Cement Toe-Down Dimensions.

Reach	Long-Term Degradation (feet)	Low Flow Incisement (feet)	Local Scour (feet)	General Scour (feet)	Antidune Troughs (feet)	Safety Factor	Total[4] Calculated Toe-Down (feet)
Upstream Right Overbank	0[1]	N/A[2]	0	2.2[3]	0	1.3	2.9
Upstream Levee Corner	0[1]	N/A[2]	6.9	3.6	3.5	1.3	18.2
2	0[1]	N/A[2]	0	3.6	3.5	1.3	9.2
3	0[1]	N/A[2]	0	5.3	4.3	1.3	12.5
4	0[1]	N/A[2]	0	0[1]	2.1	1.3	2.7

[1]The equilibrium slope or general scour analysis predict aggradation or deposition, respectively, at these locations. As a conservative approach, aggradation or desposition is not algebraicly added into the toe-down depth, a zero bed adjustment is assumed for these cases.

[2]The invert of the existing low flow channel will be used as a base elevation from which all other bed profile adjustments will be measured.

[3]On the basis of engineering judgement, the hydraulic depth in the right overbank was selected as being representative of this type of scour.

[4]These are minimum values and may be increased slightly during design to eliminate numerous grade breaks during construction.

This concludes the design example. The intent of this section was to illustrate the application of analytical concepts presented in this manual to an actual project study. As can be seen from the above analysis, "real world" problems do not always conform to the ideal conditions often used to describe the theory of a technical procedure. For instance, in this problem we found no controls in the channel bed that could be used as pivot points for an equilibrium slope analysis. A problem was also suspected in the selection of a sediment supply section for the equilibrium slope analysis. The lateral migration analysis demonstrated the need for considerable engineering judgement in selecting an erosion buffer zone.

Very seldom will projects involving fluvial systems be encountered that lend themselves to an ideal or textbook solution. All the quantitative procedures outlined in this manual should only be used as guidelines. As emphasized throughout this design manual, the final solution to a specific problem <u>must</u> be based on engineering judgement and experience.

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APPENDIX A

PACIFIC SOUTHWEST INTER-AGENCY COMMITTEE (PSIAC) METHOD FOR PREDICTING WATERSHED SOIL LOSS

Note: The information presented in APPENDIX A is from the following source: "Pacific Southwest Inter-Agency Committee, Report of the Water Management Subcommittee on Factors Affecting Sediment Yield in the Pacific Southwest Area and Selection and Evaluation of Measures for Reduction of Erosion and Sediment Yield," October, 1968.

Introduction

The material that follows is suggested for use in the evaluation of sediment yield in the Pacific Southwest. It is intended as an aid to the estimation of sediment yield for the variety of conditions encountered in this area.

The classifications and companion guide material are intended for broad planning purposes only, rather than for specific projects where more intensive investigations of sediment yield would be required. For these purposes it is recommended that map delineations be for areas no smaller than 10 square miles.

It is suggested that actual measurements of sediment yield be used to the fullest extent possible. This descriptive material and the related numerical evaluation system would best serve its purpose as a means of delineating boundaries between sediment yield areas and in extrapolation of existing data to areas where none is available.

This may involve a plotting of known sediment yield data on work maps. Prepared materials such as geologic and soil maps, topographic, climatic, vegetative type and other references would be used as aids in delineation of boundaries separating yield classifications. A study of the general relationships between known sediment yield rates and the watershed conditions that produce them would be of substantial benefit in projecting data to areas without information.

Sediment Yield Classification

It is recommended that sediment yields in the Pacific Southwest area be divided into five classes of average annual yield in acre-feet per square mile. These are as follows:

Classification 1	> 3.0	acre-feet/square mile	
2	1.0 - 3.0	"	"
3	0.5 - 1.0	"	"
4	0.2 - 0.5	"	"
5	< 0.2	"	"

Nine factors are recommended for consideration in determining the sediment yield classification. These are geology, soils, climate, runoff, topography, ground cover, land use, upland erosion, and channel erosion and sediment transport.

Characteristics of each of the nine factors which give that factor high, moderate, or low sediment yield level are shown on Table A-1. The sediment yield characteristic of each factor is assigned a numerical value representing its relative significance in the yield rating. The yield rating is the sum of values for the appropriate characteristics for each of the nine factors. Conversion to yield classes should be as follows:

Rating	Class
> 100	1
75 - 100	2
50 - 75	3
25 - 50	4
0 - 25	5

Guidelines which accompany the table are an integral part of the procedure. They describe the characteristics of factors which influence sediment yield and these are summarized in the space provided on the table.

The factors are generally described, for purposes of avoiding complexity, as independently influencing the amount of sediment yield. The variable impact of any one factor is the result of influence by the others. To account for this variable influence in any one area would require much more intensive investigational procedures than are available for broad planning purposes.

To briefly indicate the interdependence of the factors discussed separately, ground cover is used as an example. If there is no vegetation, litter or rock fragments protecting the surface, the rock, soil, and topography express their uniqueness on erosion and sediment yield. If the surface is very well protected by cover, the characteristics of the other factors are obscured by this circumstance. In similar vein, an arid region has a high potential for erosion and sediment yield because of little or no ground cover, sensitive soils and rugged topography. Given very low intensity rainfall and rare intervals of runoff, the sediment yield could be quite low.

Each of the 9 factors shown on Table A-1 are paired influences with the exception of topography. That is, geology and soils are directly related as are climate and runoff, ground cover and land use, and upland and channel erosion. Ground cover and land use have a negative influence under average or better conditions. Their impact on sediment yield is therefore indicated as a negative influence when affording better protection than this average.

It is recommended that the observer follow a feedback process whereby he checks the sum of the values on the table from A through G with the sum of H and I. In most instances high values in the former should correspond to high values in the latter. If they do not, either special erosion conditions exist or the A through G factors should be re-evaluated.

Although only the high, moderate and low sediment yield levels are shown on the attached table, interpolation between these levels may be made.

Surface Geology

Over much of the southwest area, the effect of surface geology on erosion is readily apparent. The weaker and softer rocks are more easily eroded and generally yield more sediment than do the harder more resistant types. Sandstones and similar coarse-textured rocks that disintegrate to form permeable soils erode less than shales and related mudstones and siltstones under the same conditions of precipitation. On the other hand, because of the absence of cementing agents in some soils derived from sandstone, large storms may produce some of the highest sediment yields known.

The widely distributed marine shales, such as the Mancos and shale members of the Moenkopi Formation, constitute a group of highly erodible formations. The very large areal extent of the shales and their outwash deposits gives them a rank of special importance in relation to erosion. Few of the shale areas are free from erosion. Occasionally, because of slope or cover conditions, metamorphic rocks and highly fractured and deeply weathered granites and granodiorites produce high sediment yield. Limestone and volcanic outcrop areas are among the most stable found within the western lands. The principal reason for this appears to be the excellent infiltration characteristics, which allow most precipitation to percolate into the underlying rocks.

In some areas, all geologic formations are covered with alluvial or colluvial material which may have no relation to the underlying geology. In such areas the geologic factor would have no influence and should be assigned a value of 0 in the rating.

Soils

Soil formation in the Pacific Southwest generally has not had climatic conditions conducive to rapid development. Therefore, the soils are in an immature stage of development and consist essentially of physically weathered rock materials. The presence of sodium carbonate (black alkali) in a soil tends to cause the soil particles to disperse and renders such a soil susceptible to erosion.

There are essentially three inorganic properties--sand, silt, and clay--which may in any combination give soil its physical characteristics. Organic substances plus clay provide the binding material which tends to hold the soil separates together and form aggregates. Aggregate formation and stability of these aggregates are the resistant properties of soil against erosion. Unstable aggregates or single grain soil materials can be very erodible.

Climate and living organisms acting on parent material, as conditioned by relief or topography over a period of time, are the essential factors for soil development. Any one of these factors may overshadow or depress another in a given area and cause a difference in soil formation. For instance, climate determines what type of vegetation and animal population will be present in an area, and this will have a definite influence or determine the type of soil that evolves. As an example, soils developing under a forest canopy are much different from soils developing in a grassland community.

The raw, shaley type areas (marine shales) of the Pacific Southwest have very little, if any, solid development. Colluvial-alluvial fan type areas are usually present at the lower extremities of the steeper sloping shale areas. Infiltration and percolation are usually minimal on these areas due to the fine textured nature of the soil material. This material is easily dispersed and probably has a high shrink-swell capacity. Vegetation is generally sparse, and consists of a salt desert shrub type.

There are areas that contain soils with definite profile development, and also, stony soils that contain few fines, which constitutes an improved physical condition for infiltration and plant growth over the fine textured shaley areas. These areas usually occur at higher and more moist elevations where bare, hard crystalline rocks provide the soil parent material. Vegetation and other ground cover, under these circumstances, provide adequate protection against the erosive forces and thus low sediment yield results.

In arid and semi-arid areas, an accumulation of rock fragments (desert pavement) or calcareous material (caliche) is not uncommon. These layers can offer substantial resistance to erosion processes.

The two extreme conditions of sediment yield areas have been described. Intermediate situations would contain some features of the two extremes. One such situation might be an area of predominately good soil development that contains small areas of badlands. This combination would possibly result in an intermediate classification.

Climate and Runoff

Climatic factors are paramount in soil and vegetal development and determine the quantity and discharge rate of runoff. The same factors constitute the forces that cause erosion and the resultant sediment yield. Likewise, temperature, precipitation, and particularly the distribution of precipitation during the growing season, affect the quantity and quality of the ground cover as well as soil development. The quantity and intensity of precipitation determine the amount and discharge rates of runoff and resultant detachment of soil and the transport media for sediment yield. The intensity of prevailing and seasonal winds affects precipitation pattern, snow accumulation and evaporation rate.

Snow appears to have a minor effect on upland slope erosion since raindrop impact is absent and runoff associated with snow melt is generally in resistant mountain systems.

Frontal storms in which periods of moderate to high intensity precipitation occur can produce the highest sediment yields within the Southwest. In humid and subhumid areas the impact of frontal storms on sediment may be greatest on upland slopes and unstable geologic areas where slides and other downhill soil movement can readily occur.

Convective thunderstorm activity in the Southwest has its greatest influence on eroison and sedimentation in Arizona and New Mexico and portions of the adjoining states. High rainfall intensities on low density cover or easily dispersed soils produces high sediment yields. The average annual sediment yield is usually kept within moderate bounds by infrequent occurrence of thunderstorms in any one locality.

High runoff of rare frequency may cause an impact on average annual sediment yield for a long period of time in a watershed that is sensitive to erosion, or it may have little effect in an insensitive watershed. For example, sediment that has been collecting in the bottom of a canyon and on side slopes for many years of low and moderate flows may be swept out during the rare event, creating a large change in the indicated sediment yield rate for the period of record.

In some areas the action of freezing and thawing becomes important in the erosion process. Impermeable ice usually forms in areas of fine textured soils where a supply of moisture is available before the advent of cold weather. Under these conditions the ice often persists throughout the winter and is still present when the spring thaw occurs. In some instances water tends to run over the surface of the ice and not detach soil particles, but it is possible for the ice in a surface layer to thaw during a warm period and create a very erodible situation. Spring rains with ice at shallow depth may wash away the loose material on the surface.

In some areas of the Pacific Southwest, particularly those underlain by marine shale, freezing and thawing alters the texture of soil near the surface, and thus changes the infiltration characteristics. .These areas generally do not receive enough snow or have cold enough temperatures to build a snow pack for spring melt. Later in the year soil in a loosened condition is able to absorb a large part of the early rainfall. As rains occur during the summer, the soil becomes compacted on the surface, thus allowing more water to run off and affording a greater chance for erosion.

Topography

Watershed slopes, relief, floodplain development, drainage patterns, orientation and size are basic items to consider in connection with topography. However, their influence is closely associated with geology, soils, and cover.

Generally, steep slopes result in rapid runoff. The rimrock and badlands, common in portions of the Pacific Southwest, consist of steep slopes of soft shales usually maintained by the presence of overlying cap rock. As the soft material is eroded, the cap rock is undercut and falls, exposing more soft shales to be carried away in a continuing process. However, high sediment yields from these areas are often modified by the temporary deposition of sediment on the intermediate floodplains.

The high mountain ranges, although having steep slopes, produce varying quantities of sediment depending upon the type of parent materials, soil development, and cover which directly affect the erosion processes.

Southerly exposed slopes generally erode more rapidly than do the northerly exposed slopes due to greater fluctuation of air and soil temperatures, more frequent freezing and thawing cycles, and usually less ground cover.

The size of the watershed may or may not materially affect the sediment yield per unit area. Generally, the sediment yield is inversely related to the watershed size because the larger areas usually have less overall slope, smaller proportions of upland sediment sources, and more opportunity for the deposition of upstream derived sediments on floodplains and fans. In addition, large watersheds are less affected by small convective type storms. However, under other conditions, the sediment yield may not decrease as the watershed size increases. There is little change in mountainous areas of relatively uniform terrain. There may be an increase of sediment yield as the watershed size increases if downstream watersheds or channels are more susceptible to erosion than upstream areas.

Ground Cover

Ground cover is described as anything on or above the surface of the ground which alters the effect of precipitation on the soil surface and profile. Included in this factor are vegetation, litter, and rock fragments. A good ground cover dissipates the energy of rainfall before it strikes the soil surface, delivers water to the soil at a relatively uniform rate, impedes the flow of water, and promotes infiltration by the action of roots within the soil. Conversely, the absence of ground cover, whether through natural growth habits or the effect of overgrazing or fire, leave the land surface open to the worst effects of storms.

In certain areas, small rocks or rock fragments may be so numerous on the surface of the ground that they afford excellent protection for any underlying fine material. These rocks absorb the energy of falling rain and are resistant enough to prevent cutting by flowing water.

The Pacific Southwest is made up of land with all classes of ground cover. The high mountain areas generally have the most vegetation, while many areas in the desert regions have practically none. The abundance of vegetation is related in a large degree to precipitation. If vegetative ground cover is destroyed in areas where precipitation is high, abnormally high erosion rates may be experienced.

Differences in vegetative type have a variable effect on erosion and sediment yield, even though percentages of total ground cover may be the same. For instance, in areas of pinyon-juniper forest having the same percentage of ground cover as an area of grass, the absence of understory in some of the pinyon-juniper stands would allow a higher erosion rate than in the area of grass.

Land Use

The use of land has a widely variable impact on sediment yield, depending largely on the susceptibility of the soil and rock to erosion, the amount of stress exerted by climatic factors and the type and intensity of use. Factors other than the latter have been discussed in appropriate places in this guide.

In almost all instances, use either removes or reduces the amount of natural vegetative cover which reflects the varied relationships within the environment. Activities which remove all vegetation for parts of each year for several years, or permanently, are cultivation, urban development, and road construction. Grazing, logging, mining, and fires artifically induce permanent or temporary reduction in cover density.

High erosion hazard sites, because of the geology, soils, climate, etc., are also of high hazard from the standpoint of type and intensity of use. For example, any use which reduces cover density on a steep slope with erodible soils and severe climatic conditions will strongly affect sediment yield. The extent of this effect will depend on the area and intensity of use relative to the availability of sediment from other causes. Construction of road or urban development with numerous cut and fill slopes through a large area of widespread sheet or gully erosion will probably not cause a change in sediment yield classification. Similar contruction and continued disturbance in an area of good vegetative response to a favorable climate can raise yield by one or more classifications.

Use of the land has its greatest potential impact on sediment yield where a delicate balance exists under natural conditions. Alluvial valleys of fine, easily dispersed soils from shales and sandstones are highly vulnerable to erosion where intensive grazing and trailing by livestock have occurred. Valley trenching has developed in many of these valleys and provides a large part of the sediment in high yield classes from these areas.

A decline in vegetative density is not the only effect of livestock on erosion and sediment yield. Studies at Badger Wash, Colorado, which is underlain by Mancos shale, have indicated that sediment yield from ungrazed watersheds is appreciably less than from those that are grazed. This difference is attributed to the absence of soil trampling in the ungrazed areas, since the density of vegetation has not noticeably changed since exclusion began.

Areas in the arid and semi-arid portions of the Southwest that are surfaced by desert pavement are much less sensitive to grazing and other use, since the pavement affords a substitute for vegetative cover.

In certain instances the loss or deterioration of vegetative cover may have little noticeable on-site impact but may increase off-site erosion by acceleration of runoff. This could be particularly evident below urbanized areas where accelerated runoff from pavement and rooftops has increased the stress on downstream channels. Widespread destruction of cover by poor logging practices or by brush and timber fires frequently increases channel erosion as well as that on the directly affected watershed slopes. On the other hand, cover disturbances under favorable conditions, such as a cool, moist climate, frequently result in a healing of erosion sources within a few years.

Upland Slope Erosion

This erosion form occurs on sloping watershed lands beyond the confines of valleys. Sheet erosion, which involves the removal of a thin layer of soil over an extensive area, is usually not visible to the eye. This erosion form is evidenced by the formation of rills. Experience indicates that soil loss from rill erosion can be seen if it amounts to about 5 tons or more per acre. This is equivalent in volume per square mile to aproximately 2 acre-feet.

Wind erosion from upland slopes and the deposition of the eroded material in stream channels may be a significant factor. The material so deposited in channels is readily moved by subsequent runoff.

Downslope soil movement due to creep can be an important factor in sediment yield on steep slopes underlain by unstable geologic formations.

Significant gully erosion as a sediment contributor is evidenced by the presence of numerous raw cuts along the hill slopes. Deep soils on moderately steep to steep slopes usually provide an environment for gully development.

Processes of slope erosion must be considered in the light of factors which contribute to its development. These have been discussed in previous sections.

Channel Erosion and Sediment Transport

If a stream is ephemeral, runoff that traverses the dry alluvial bed may be drastically reduced by transmission losses (absorption by channel alluvium). This decrease in the volume of flow results in a decreased potential to move sediment. Sediment may be deposited in the streambed from one or a series of relatively small flows only to be picked up and moved on in a subsequent larger flow. Sediment concentrations, determined from field measurements at consecutive stations, have generally been shown to increase many fold for instances of no tributary inflow. Thus, although water yield per unit area will decrease with increasing drainage area, the sediment yield per unit area may remain nearly constant or may even increase with increasing drainage area.

In instances of convective precipitation in a watershed with perennial flow, the role of transmission losses is not as significant as in watersheds with ephemeral flow, but other channel factors, such as the shape of the channel, may be important.

For frontal storm runoff, the flow durations are generally much longer than for convective storms, and runoff is often generated from the entire basin. In such instances, sediment removed from the land surfaces is generally carried out of the area by the runoff. Stream channel degradation and/or aggradation must be considered in such cases, as well as bank scour. Because many of the stream beds in the Pacific Southwest are composed of fine-grained alluvium in well defined channels, the potential for sediment transport is limited only by the amount and duration of runoff. Large volumes of sediment may thus be moved by these frontal storms because of the longer flow durations.

The combination of frontal storms of long duration with high intensity and limited areal-extent convective activity will generally be in the highest class for sediment movement in the channels. Storms of this type generally produce both the high peak flows and the long durations necessary for maximum sediment transport.

Sediment yield may be substantially affected by the degree of channel development in a watershed. This development can be described by the channel cross sections, as well as by geomorphic parameters such as drainage density, channel gradients and width-depth ratio. The effect of these geomorphic parameters is difficult to evaluate, primarily because of the scarcity of sediment transport data in the Pacific Southwest.

If the cross section of a stream is such as to keep the flow within defined banks, then the sediment from an upstream point is generally transported to a downstream point without significant losses. Confinement of the flow within alluvial banks can result in a high erosional capability of a flood flow, especially the flows with long return periods. In most channels with wide floodplains, deposition on the floodplain during floods is often significant, and the transport is thus less than that for a within bank flow. The effect of this transport capability can be explained in terms of tractive force which signifies the hydraulic stress exerted by the flow on the bed of the stream. This average bed-shear stress is obtained as the product of the specific weight of the fluid, hydraulic radius, and energy gradient slope. Thus, greater depth results in a greater bed shear and a greater potential for moving sediment. By the same reasoning, steep slopes (the energy slope and bed slope are assumed to be equivalent) also result in high bed-shear stress.

The boundary between sediment yield classifications in much of the Pacific Southwest may be at the mountain front, with the highest yield designation on the alluvial plain if there is extensive channel erosion. In contrast, many mountain streams emerge from canyon reaches and then spread over fans or valley flats. Here water depths can decrease from many feet to only a few inches in short distances with a resultant loss of the capacity to transport sediment. Sediment yield of the highest classification can thus drop to the lowest in such a transition from a confined channel to one that has no definition.

Channel bank and bed composition may greatly influence the sediment yield of a watershed. In many areas within the Pacific Southwest, the channels in valleys dissect unconsolidated material which may contribute significantly to the stream sediment load. Bank sloughing during periods of flow, as well as during dry periods, piping, and bank scour generally add greatly to the sediment load of the stream and often change upward the sediment yield classification of the watershed. Field examination for areas of head cutting, aggradation or degradation, and bank cutting are generally necessary prior to classification of the transport expectancy of a stream. Geology plays a significant role in such an evaluation. Geologic controls in channels can greatly affect the stream regimen by limiting degradation and headcuts. Thus, the transport capacity may be present, but the supply of sediment from this source is limited.

Man-made structures can also greatly affect the transport characteristics of the stream. For example, channel straightening can temporarily upset the channel equilibrium and cause an increase in channel gradient and an increase in the stream velocity and the shear stress. Thus, the sediment transport capacity of the stream may be temporarily increased. Structures such as debris dams, lined channels, drop spillways, and detention dams may drastically reduce the sediment transport.

AN EXPLANATION OF THE USE OF THE RATING CHART (TABLE A-1) FOR EVALUATING FACTORS AFFECTING SEDIMENT YIELD IN THE PACIFIC SOUTHWEST FOLLOWS

Table A.1. Factors Affecting Sediment Yield in the Pacific Southwest.

Sediment Yield Levels	A SURFACE GEOLOGY (10)*	B SOILS (10)	C CLIMATE (10)	D RUNOFF (10)	E TOPOGRAPHY (20)	F GROUND COVER (10)	G LAND USE (10)	H UPLAND EROSION (25)	I CHANNEL EROSION & SEDIMENT TRANSPORT (25)
High	a. Marine shales and related mudstones and siltstones.	a. Fine textured; easily dispersed; saline-alkaline; high shrink-swell characteristics b. Single grain silts and fine sands	a. Storms of several days' duration with short periods of intense rainfall. b. Frequent intense convective storms c. Freeze-thaw occurrence	a. High peak flows per unit area b. Large volume of flow per unit area	a. Steep upland slopes (in excess of 30%) High relief; little or no floodplain development	Ground cover does not exceed 20% a. Vegetation sparse; little or no litter b. No rock in surface soil	a. More than 50% cultivated b. Almost all of area intensively grazed c. All of area recently burned	a. More than 50% of the area characterized by rill and gully or landslide erosion	a. Eroding banks continuously or at frequent intervals with large depths and long flow duration b. Active headcuts and degradation in tributary channels
**									
Moderate	(5) a. Rocks of medium hardness b. Moderately weathered c. Moderately fractured	(5) a. Medium textured soil b. Occasional rock fragments c. Caliche layers	(5) a. Storms of moderate duration and intensity b. Infrequent convective storms	(5) a. Moderate peak flows b. Moderate volume of flow per unit area	(10) a. Moderate upland slopes (less than 20%) b. Moderate fan or floodplain development	(10) Cover not exceeding 40% a. Noticeable litter b. If trees present understory not well developed	(0) a. Less than 25% cultivated b. 50% or less recently logged c. Less than 50% intensively grazed d. Ordinary road and other construction	(10) a. About 25% of the area characterized by rill and gully or landslide erosion b. Wind erosion with deposition in stream channels	(10) a. Moderate flow depths, medium flow duration with occasionally eroding banks or bed
**									
Low	(0) a. Massive, hard formations	(0) a. High percentage of rock fragments b. Aggregated clays c. High in organic matter	(0) a. Humid climate with rainfall of low intensity b. Precipitation in form of snow c. Arid climate, low intensity storms d. Arid climate; rare convective storms	(0) a. Low peak flows per unit area b. Low volume of runoff per unit area c. Rare runoff events	(0) a. Gentle upland slopes (less than 5%) b. Extensive alluvial plains	(-10) a. Area completely protected by vegetation, rock fragments, litter. Little opportunity for rainfall to reach erodible material	(-10) a. No cultivation b. No recent logging c. Low intensity grazing	(0) a. No apparent signs of erosion	(0) a. Wide shallow channels with flat gradients, short flow duration b. Channels in massive rock, large boulders or well vegetated c. Artificially controlled channels

* THE NUMBERS IN SPECIFIC BOXES INDICATE VALUES TO BE ASSIGNED APPROPRIATE CHARACTERISTICS.
THE SMALL LETTERS, a, b, c, REFER TO INDEPENDENT CHARACTERISTICS TO WHICH FULL VALUE MAY BE ASSIGNED.

** IF EXPERIENCE SO INDICATES, INTERPOLATION BETWEEN THE 3 SEDIMENT YIELD LEVELS MAY BE MADE.

Use of the Rating Chart of Factors Affecting Sediment Yield in the Pacific Southwest

The following is a summary of the sediment yield classification presented for this methodology.

Classification	Rating	Sediment Yield AF/sq. mi.
1	> 100	3.0
2	75 - 100	1.0 - 3.0
3	50 - 75	0.5 - 1.0
4	25 - 50	0.2 - 0.5
5	0 - 25	< 0.2

In most instances, high values for the A through G factors should correspond to high values for the H and/or I factors.

An example of the use of the rating chart is as follows:

A watershed of 15 square miles in western Colorado has the following characteristics and sediment yield levels:

	Factors	Sediment Yield Levels	Rating
A	Surface geology	Marine Shales	10
B	Soils	Easily dispersed, high shrink-swell characteristics	10
C	Climate	Infrequent convective storms, freeze-thaw occurrence	7
D	Runoff	High peak flows; low volumes	5
E	Topography	Moderate slopes	10
F	Ground cover	Sparse, little or no litter	10
G	Land use	Intensively grazed	10
H	Upland erosion	More than 50% rill and gully erosion	25
I	Channel erosion	Occasionally eroding banks and bed but short flow duration	5
		TOTAL	92

This total rating of 92 would indicate that the sediment yield is in Classification 2. This compares with a sediment yield of 1.96 acre-feet per square mile as the average of a number of measurements in this area.

APPENDIX B

MODIFIED UNIVERSAL SOIL LOSS EQUATION FOR PREDICTING WATERSHED SOIL LOSS

The Modified Universal Soil Loss Equation (MUSLE) described by Williams (1975) is an empirically derived methodology for predicting watershed sediment yield on a per-storm basis. The MUSLE is

$$Y_s = R_w\ K\ LS\ C\ P \tag{B.1}$$

where Y_s is sediment yield in tons for the storm event, R_w is a storm runoff energy factor, K is the soil erodibility factor, LS is the topographic factor representing the combination of slope length and slope gradient, C is th ecover and management factor and P is the erosion control practice factor. Factors K, LS, C and P are as defined for the Universal Soil Loss Equation (USLE), as reviewed in later paragraphs (Smith and Wischmeier, 1975, Wischmeier, 1960, and Wischmeier and Smith, 1978, provide detailed descriptions of the USLE factors and their values).

The storm runoff energy factor R_w in Equation B.1 represents the modification of the USLE where R_w is given by

$$R_w = \alpha\ (Vq_p)^{\beta} \tag{B.2}$$

In Equation B.2, V is the storm event runoff volume in acre-feet, q_p is the storm event peak flow rate in cfs, and α and β are coefficients. Utilization of a storm runoff factor makes the MUSLE applicable to semiarid regions of the West where short-duration, high-intensity storms are cominant. For watersheds having measured sediment data, values for the coefficients α and β can be derived through calibration. Williams and Berndt (1972) determined values for α and β of 95 and 0.56, respectively, for experimental watersheds in Texas and Nebraska.

Soil erodibility factor K was found by Wischmeier et al. (1971) to be a function of percent of silt, percent of coarse sand, soil structure, permeability of soil, and percent of organic matter. The soil erodibility nomograph is shown in Figure B.1.

Wischmeier (1972) presented a method including graphical aids for determining the cover and management factor (cropping-management factor C). The

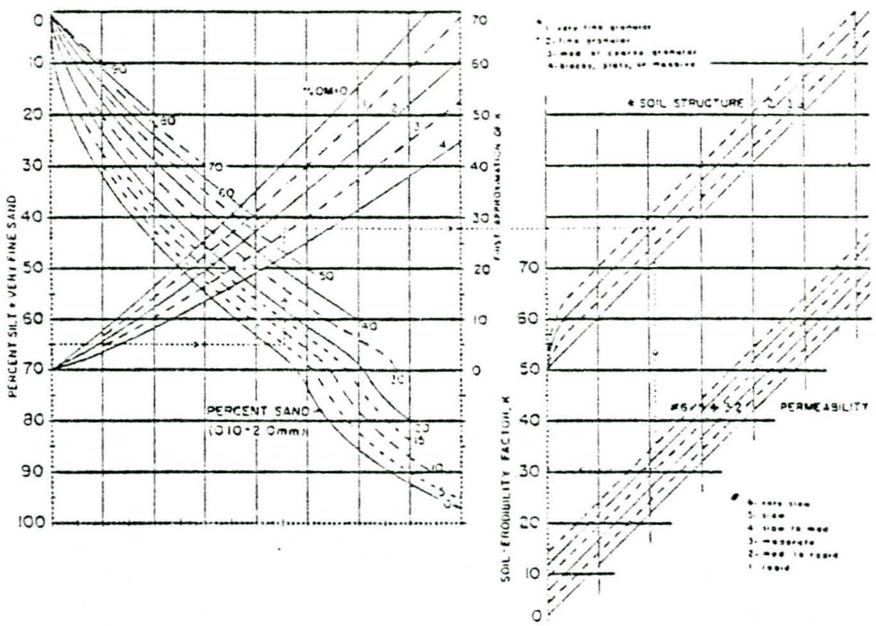

Figure B.1. Soil erodibility nomograph used to determine factor K for specific topsoils or subsoil horizons. Solutions are in tons/acre (from Wischmeier et al., 1971).

cropping-management factor can be divided into three distinct types of effects as follows: Type I -effects of canopy cover (C_I), Type II - effects of mulch or close-growing vegetation in direct contact with the soil surface (C_{II}), and Type III - tillage and residual effects of the land use (C_{III}). The cover and management factor is defined as the product of these factors:

$$C = C_I \, C_{II} \, C_{III} \tag{B.3}$$

Type I - Canopy Cover. Leaves and branches that do not directly contact the soil are effective only as canopy cover. A canopy can intercept falling raindrops, but waterdrops falling from the canopy may regain an appreciable velocity, although not the terminal velocities of free-falling raindrops. Therefore, canopy cover reduces rainfall erosivity by reducing impact energy at the soil surface. The amount of reduction depends on height and density of the canopy. Figure B.2 shows the canopy factor as a function of height and density of the canopy. Canopy factors for various percentages of cover at heights of 0.5, 1.0, 2.0 and 4.0 meters may be obtained directly from this graph.

Type II - Mulch and Close-Growing Vegetation. A mulch at the soil-atmosphere interface is much more effective than an equivalent percentage of canopy cover. Because intercepted raindrops have no remaining fall-height to the ground, their impact on the soil surface is eliminated. A mulch that makes a good contact with the ground also reduces runoff velocity, which greatly reduces the potential of runoff to detach and transport soil material.

Substantial rainfall simulator data are now available on erosion-reducing effectiveness of various amounts and types of mulches used on cropland and construction sites. Extrapolation of these data to other mulches and close covers such as those associated with range or woodland is facilitated by expressing them on the basis of percent surface cover rather than tons per acre. This conversion and a preliminary summarization of data are reflected in the relationship curve shown in Figure B.3.

Type III - Residual Effects of Land Use. This category includes residual effects of land use on soil structure, organic matter content and soil density, effects of tillage or lack of tillage on surface roughness and porosity, roots and subsurface stems, biological effects, and other factors. This factor can be evaluated from Figure B.4 by knowing the percent of root network in the topsoil relative to a good rotation meadow. This percent of root network

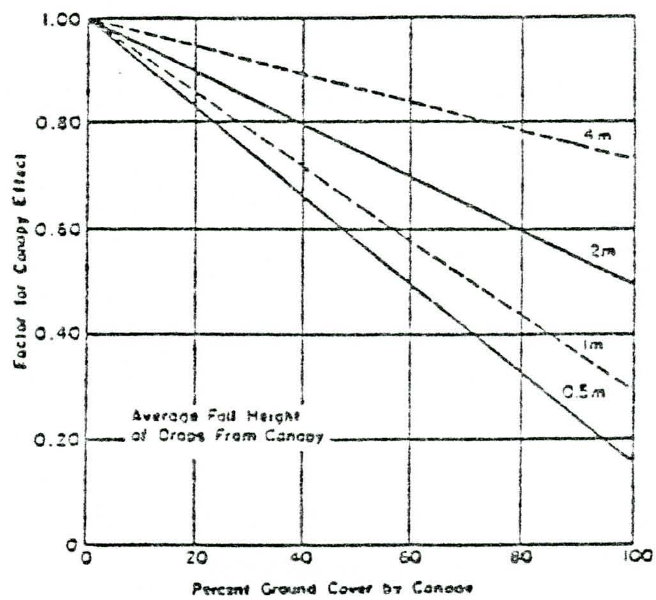

Figure B.2. Influence of vegetal canopy on effective-EI (after Wischmeier, 1972).

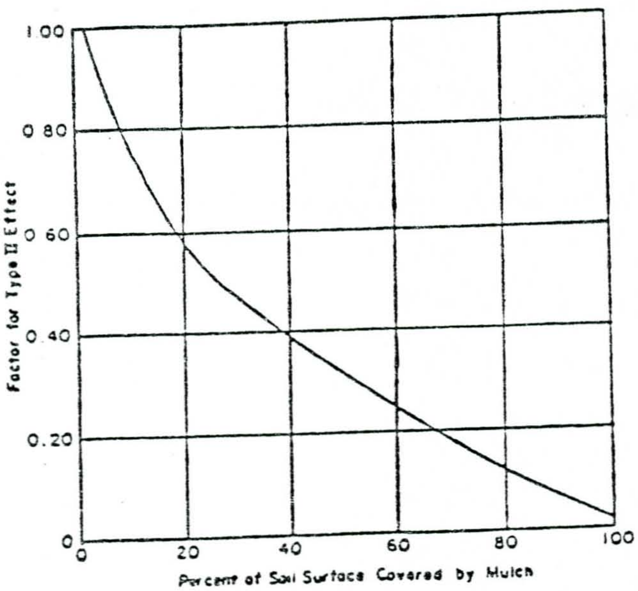

Figure B.3. Effect of plant residues or close-growing stems at the soil surface (after Wischmeier, 1972).

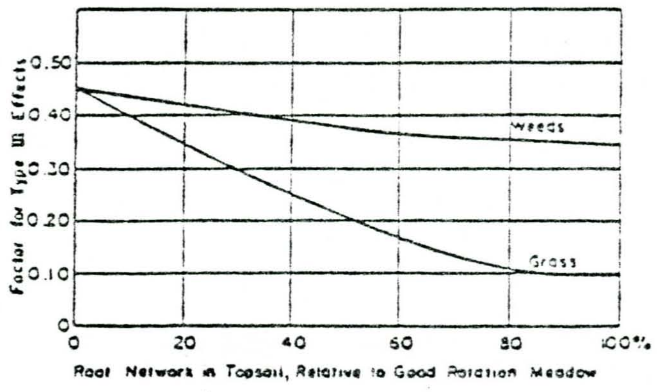

Figure B.4. Type III effects on undisturbed land areas (after Wischmeier, 1972).

is often a difficult value to estimate. The overall C factor can be evaluated by the product of three subfactors, i.e., Type I, II and III subfactors.

The topographic factor LS is defined as the ratio of soil loss from any slope and length to soil loss from a 72.6-foot plot length at a nine percent slope, with all other conditions the same. Slope length is defined as the distance from the point of overland flow origin to the point where either slope decreases to the extent that deposition begins or runoff water enters a well-defined channel (Smith and Wischmeier, 1957). Effect of slope length on soil loss is primarily a result of increased potential due to greater accumulation of runoff on the longer slopes. Based on data for slopes between three and 20 percent and with lengths up to 400 feet, Wischmeier and Smith (1965) proposed the topographic factor be computed as

$$LS = (\frac{\lambda}{72.6})^n \ (0.065 + 0.0454\ S + 0.0065\ S^2 \qquad \text{(B.4)}$$

where λ is slope length, S is percent slope, and n is an exponent depending upon slope. The exponent n is given by

$n = 0.3$ for slope ≤ 3 percent

$n = 0.4$ for slope $= 4$ percent

$n = 0.5$ for slope ≥ 5 percent

Erosion-control practice factor P accounts for the effect of conservation practices such as contouring, strip cropping, and terracing on erosion. It is defined as the ratio of soil loss using one of these practices to the loss using straight row farming up and down the slope. Terracing is generally the most effective conservation practice for decreasing soil erosion. This factor has no significance for wildland areas and can be set at 1.0.

When estimating sediment yield using the MUSLE, a useful computation is to express sediment yield in terms of an average concentration (ppm) based on the total water and sediment yields. This value can be compared with measured stream data in the area, if available. Annual sediment yield from the land surface can be estimated using the MUSLE in combination with the probability-weighting procedure described in Section 3.4. Application of the MUSLE to estimate watershed soil loss is illustrated in the following example.

Example:

Watershed area = 25.3 mi^2

Annual rainfall = 10.0 in.

Average runoff = 1.5 percent of rainfall = 0.15 in.

Watershed soil: 43 percent silt and very fine sand
40 percent sand
1 percent organic matter
Fine granular structure
Moderate permeability

Average watershed slope = 14 percent

Average slope length = 280 ft

Canopy cover density = 10 percent

Average fall height = 1.5 ft

Close-growing vegetation density = 15 percent

Root network in topsoil (weeds) = 20 percent

For purposes of illustration, assume $\alpha = 95$ and $\beta = 0.56$.

Step 1: Determine Factor Values

R_w Factor

See Table B.1.

K Factor

K factor nomograph (Figure B.1) $K = 0.26$

C Factor

From Figure B.2 for 10 percent canopy cover $C_I = 0.90$

From Figure B.3 for 15 percent ground cover $C_{II} = 0.67$

From Figure B.4 for 20 percent root network $C_{III} = 0.42$

$$C = C_I \, C_{II} \, C_{III} = 0.25$$

LS Factor

Applying Equation B.4,

$$LS = \left(\frac{280}{72.6}\right)^{0.5} [0.065 + 0.0454\ (14) + 0.0065\ (14^2)] = 3.9$$

P Factor

No supporting practices, therefore $P = 1.0$

Step 2: Apply Equation B.1

$$Y_s = R_w\ (0.26)\ (3.9)\ (0.25)\ (1.0)$$

$$= 0.25\ R_w$$

Results are given in Table B.2.

Table B.1. Factor R_w Calculation.

Return Period (years)	Runoff Volume (ac-ft)	Peak Runoff (cfs)	R_w
2	123	340	37,800
5	320	900	108,400
10	595	1,650	215,400
25	915	2,540	349,000
50	1,200	3,330	472,800
100	1,510	4,190	611,500

Table B.2. MUSLE Sediment Yield Estimate.

Return Period (years)	R_W	Washload (tons)
2	37,800	9,000
5	108,400	27,000
10	215,400	54,000
25	349,000	87,000
50	472,800	118,000
100	611,500	153,000