Hydraulic and Erosion Control Design for Flood Control Channels

FLOOD CONTROL DISTRICT of MARICOPA COUNTY

A TRAINING SEMINAR

hydrology & hydraulics

fluvial systems

flood control design

bank failure

sedimentation

alluvial fans

Presented
to the
Flood Control District of
Maricopa County
Dec. 3 - 5, 1996
by West Consultants,
Inc.

HYDRAULIC AND EROSION CONTROL DESIGN FOR FLOOD CONTROL CHANNELS

Presented to:

Flood Control District of Maricopa County
2801 West Durango
Phoenix, AZ

Presented by:

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WEST Consultants, Inc.

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Short Course Outline

Day 1

8:00 - 8:15	Introductions and Administrative
8:15 - 9:30	Lecture 1, Basics of Hydrology
9:30 - 10:00	Break
10:00 - 11:30	Lecture 2, Basics of Hydraulics
11:30 - 12:30	Lunch
12:30 - 2:00	Lecture 3, Basics of Fluvial Systems
2:00 - 2:30	Break
2:30 - 3:30	Lecture 3, Basics of Fluvial Systems, continued
3:30 - 3:45	Break
3:45 - 5:00	Lecture 4, Study Plan and Project Alternative Formulation

HYDRAULIC AND EROSION CONTROL DESIGN FOR FLOOD CONTROL CHANNELS

Day 2

8:00 - 9:30	Lecture 5,	Concepts and Considerations in Flood Control Channel Design
9:30 - 10:00	Break	
10:00 - 11:30	Lecture 6, Special Features of Flood Control Projects	
11:30 - 12:30	Lunch	
12:30 - 1:45	Lecture 7, Channel Modifications and Potential Response	
1:45 - 2:00	Break	
2:00 - 2:45	Lecture 8, Process of Bank Failure	
2:45 - 3:00	Break	
3:00 - 4:30	Lecture 9, Streambank Protection and Design Methods	
4:30 - 5:00	Lecture 10, Soil Cement	

HYDRAULIC AND EROSION CONTROL DESIGN FOR FLOOD CONTROL CHANNELS

Day 3

8:00 - 9:15	Lecture 11, Sediment Transport Concepts
9:15 - 9:45	Break
9:45 - 10:15	Lecture 12, Sedimentation or Debris Basins
10:15 - 11:30	Lecture 13, Stable Channel Concepts
11:30 - 12:30	Lunch
12:30 - 1:15	Lecture 14, Environmental Considerations in Hydraulic Design
1:15 - 2:00	Lecture 15, FEMA Regulation of Rivers, Streams and Washes
2:30 - 3:00	Break
3:00 - 3:45	Lecture 16, Alluvial Fans
3:45 - 4:30	Lecture 17, Field Reconnaissance
4:30 - 4:40	Lecture 18, Bibliography, Lecture 19, Glossary
4:40 - 5:00	Open Discussion

BASICS OF

HYDROLOGY

Lecture 1

<u>Class Goal</u> - Basic Understanding of Hydrological Processes

* Why is hydrology important?
* The Hydrologic Cycle - what it is, examination of its parts
* Streamflow concepts
* Frequency concepts

WHY IS HYDROLOGY IMPORTANT?

1. Long-term yield

2. Short-term effects

3. Usage of Water

THE HYDROLOGIC CYCLE

1. Schematic (Figure 1)

2. Where on earth is all the water?

Item	Percent of total water	Percent of fresh water
Oceans	96.5	-
Groundwater		
Fresh	0.76	30.1
Saline	0.93	-
Soil Moisture	0.0012	0.05
Polar Ice	1.7	68.6
Other Ice & Snow	0.025	1.0
Lakes		
Fresh	0.007	0.26
Saline	0.006	-
Marshes	0.008	0.03
Rivers	0.0002	0.006
Biological water	0.0001	0.003
Atmospheric water	0.001	0.04
Total water	100	-
Fresh water	2.5	100

(Data from UNESCO, 1978)

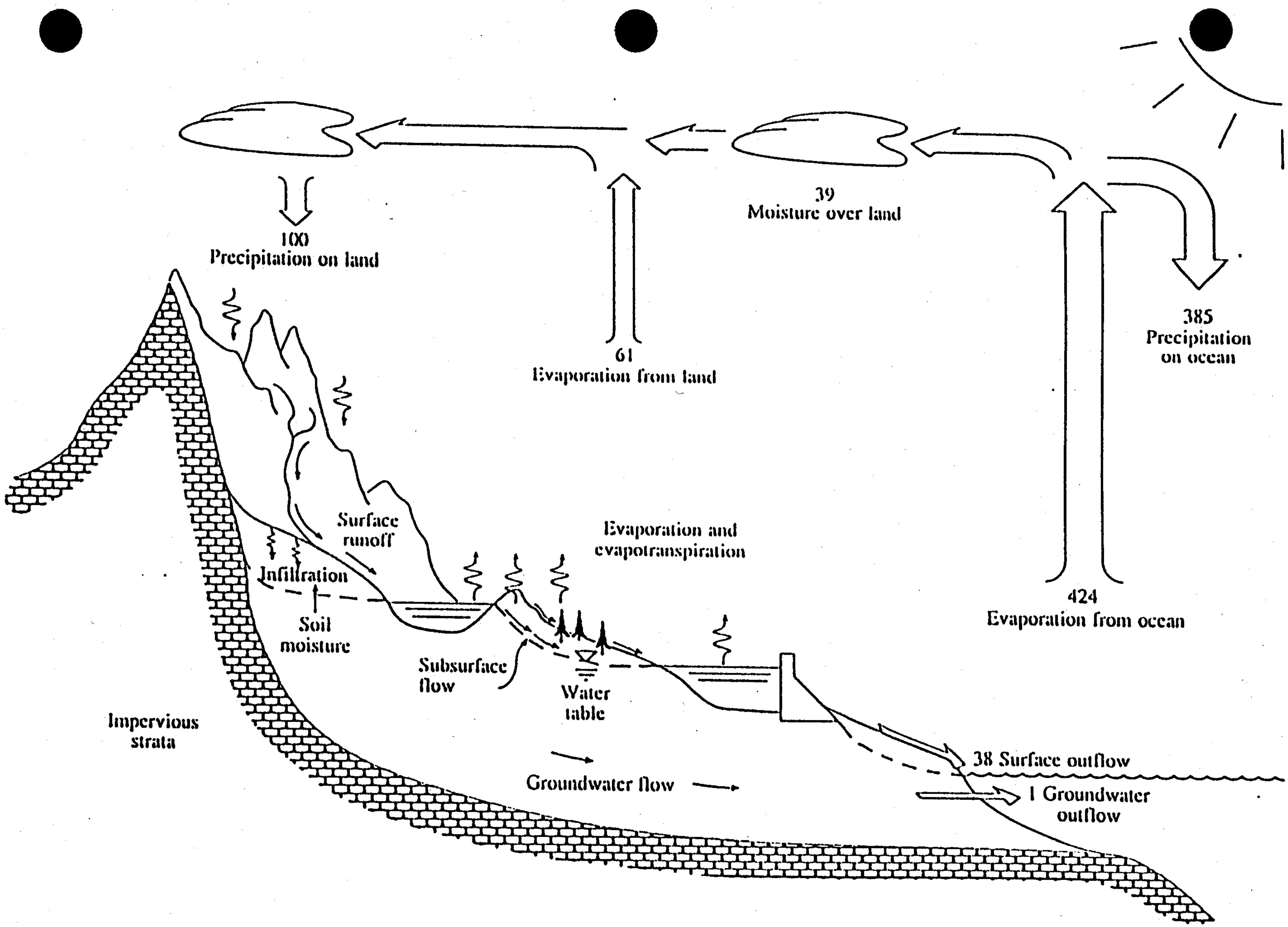


FIGURE 1.
Hydrologic cycle with global annual average water balance given in units relative to a value of 100 for the rate of precipitation on land.

3. Hydrologic Budget

O - I = dS/dt : Outflow - Inflow equals change in storage over time

4. Common Units of Measurement

Quantity	SI units	abbreviation	English units	Abbreviation
Flow	cubic meter per second	m^3/s	cubic feet per second	cfs
			gallons per minute	gpm
			millions of gallons per day	mgd
Volume	cubic meters	m3	acre-foot	acre-ft
	centimeters (over an area)	cm	inches (over an area)	in

1 acre = 43,560 ft^2
1 inch of runoff per square mile = 53.3 acre-ft = 2,323,200 ft^3
1 acre-ft = 1233 m^3 = 43,560 ft3
1 cfs = 0.02832 m^3/s
1 mgd = 694.4 gpm = 1.547 cfs = 2.629 m^3/min

5. Precipitation

Three main types:

- Convective (thunderstorms)
- Orographic (mountains)
- Cyclonic (movement of air masses)

Data Collection:

- ✱ Rain gage networks
- ✱ Radar estimates
- ✱ Use data from similar basins
- ✱ National Weather Service (NWS) published data:

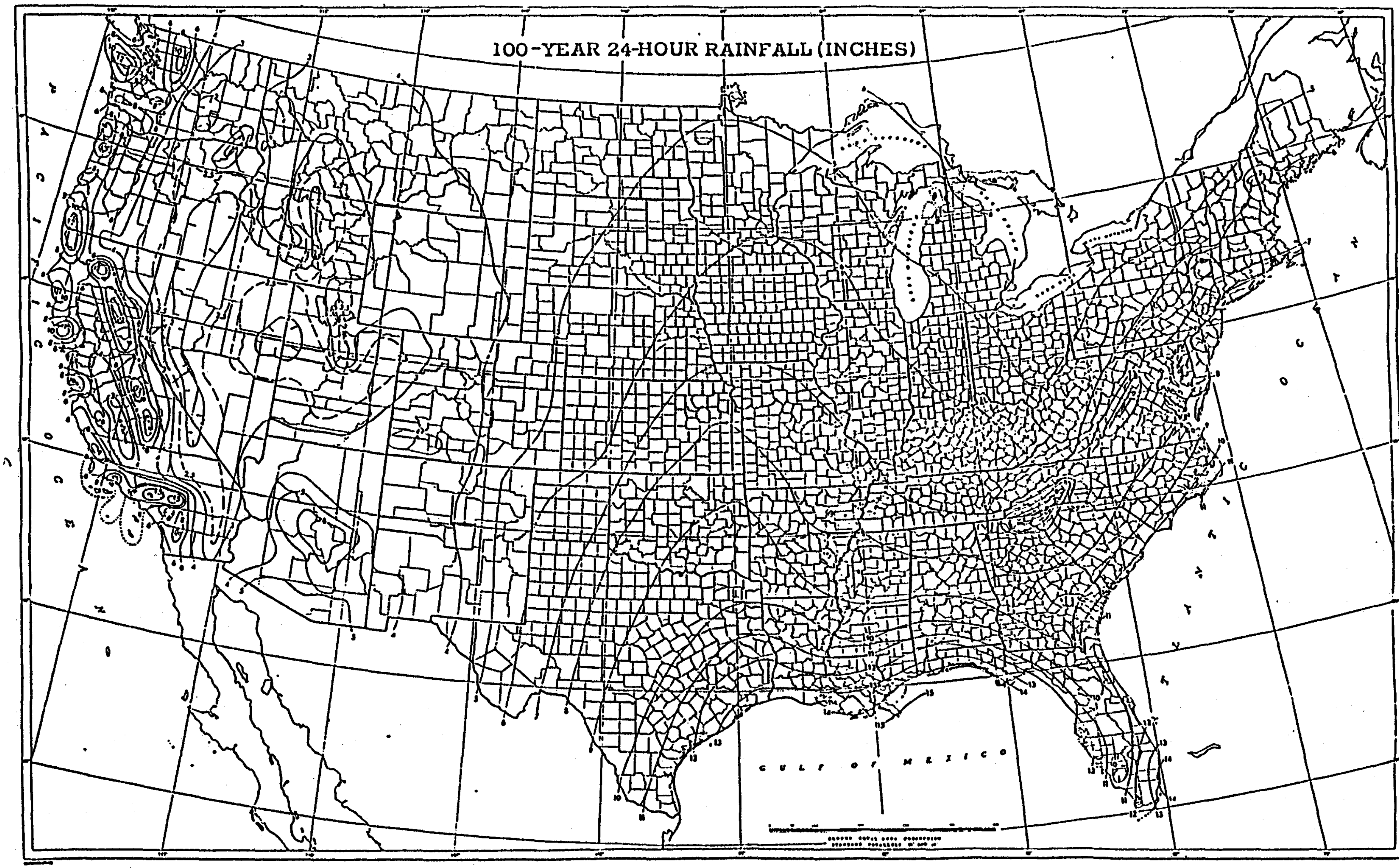
  ➤ TP-40: Provides maps for the 1-, 2-, 5-, 10-, 25-, 50-, and 100-year precipitation depths for durations from 30 minutes to 24 hours for the eastern United States (1961).

  ➤ TP-49: Provides maps for the 2- through 100-year precipitation depths for durations from 1 to 10 days for the eastern United States (1964).

  ➤ NOAA Technical Memorandum NWS Hydro-35: Provides revised precipitation estimates for durations of 5 to 60 for the eastern United States (1977, replaces some TP-40 data).

  ➤ NOAA Atlas 2: Provides precipitation estimates for the 2-, 5-, 10-, 25-, 50-, and 100-year recurrence intervals for durations of 6 and 24 hours for the western United States. Methods for estimating other durations also included (1977).

Probable Maximum Precipitation (PMP) : A synthetic storm having the theoretically greatest depth of precipitation for a given duration that is physically possible over a given size area at a particular geographic location at a certain time of the year.



From U.S. WEATHER BUREAU TECHNICAL PAPER NO. 40

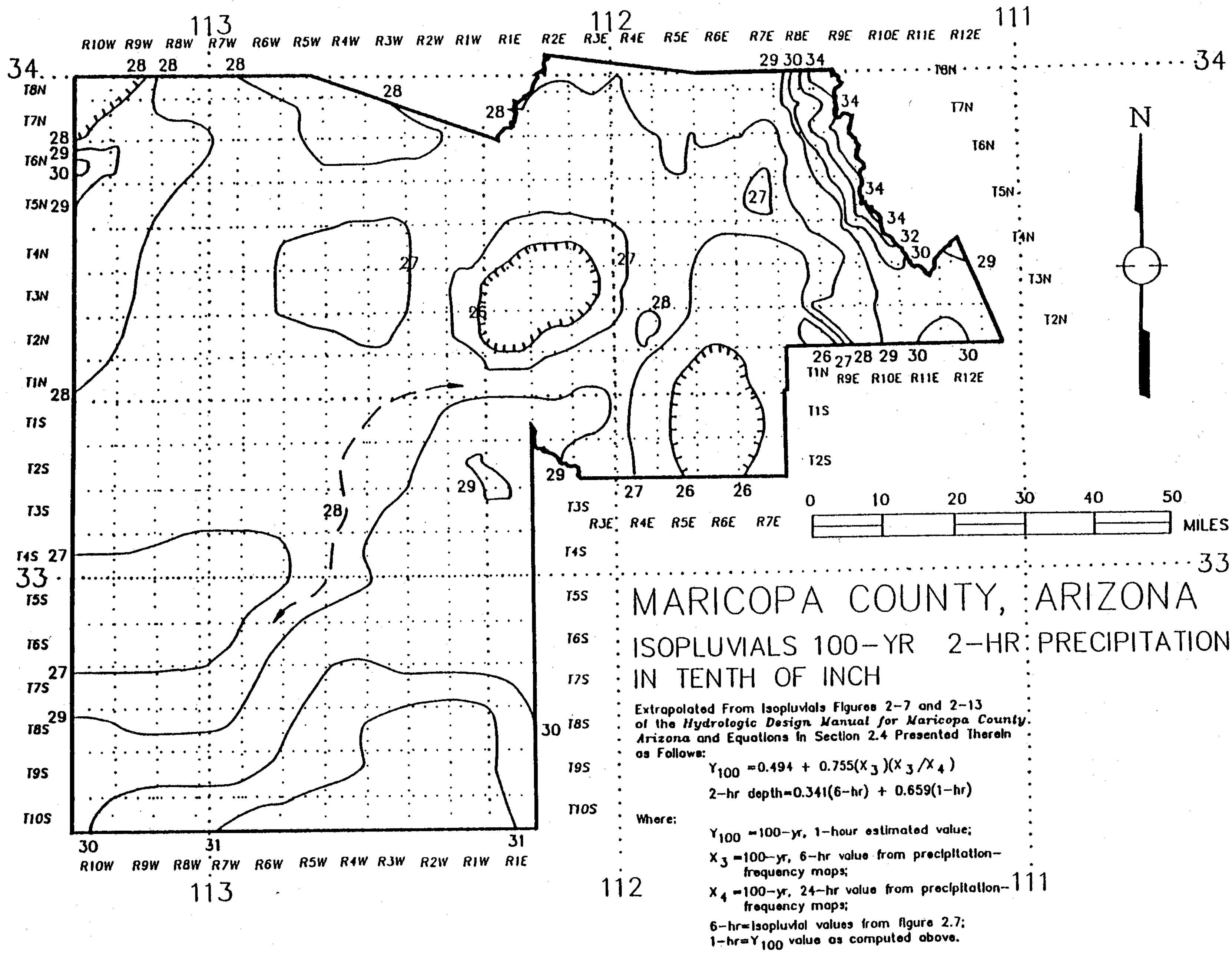


Figure 8.1
Isopluvial 100-Year, 2-Hour Precipitation

6. Abstractions

* Evaporation
* Infiltration
* Transpiration
* Interception

7. Basin characteristics affecting discharge

* Land use
* Soil types
* Vegetation

STREAMFLOW

1. Hydrographs

Definition - A hydrograph shows the flow rate as a function of time at a given location on a stream (Fig. 5.3.1).

* Rainfall - Abstractions = Excess Rainfall
* Excess Rainfall becomes direct runoff (Fig. 5.3.1)

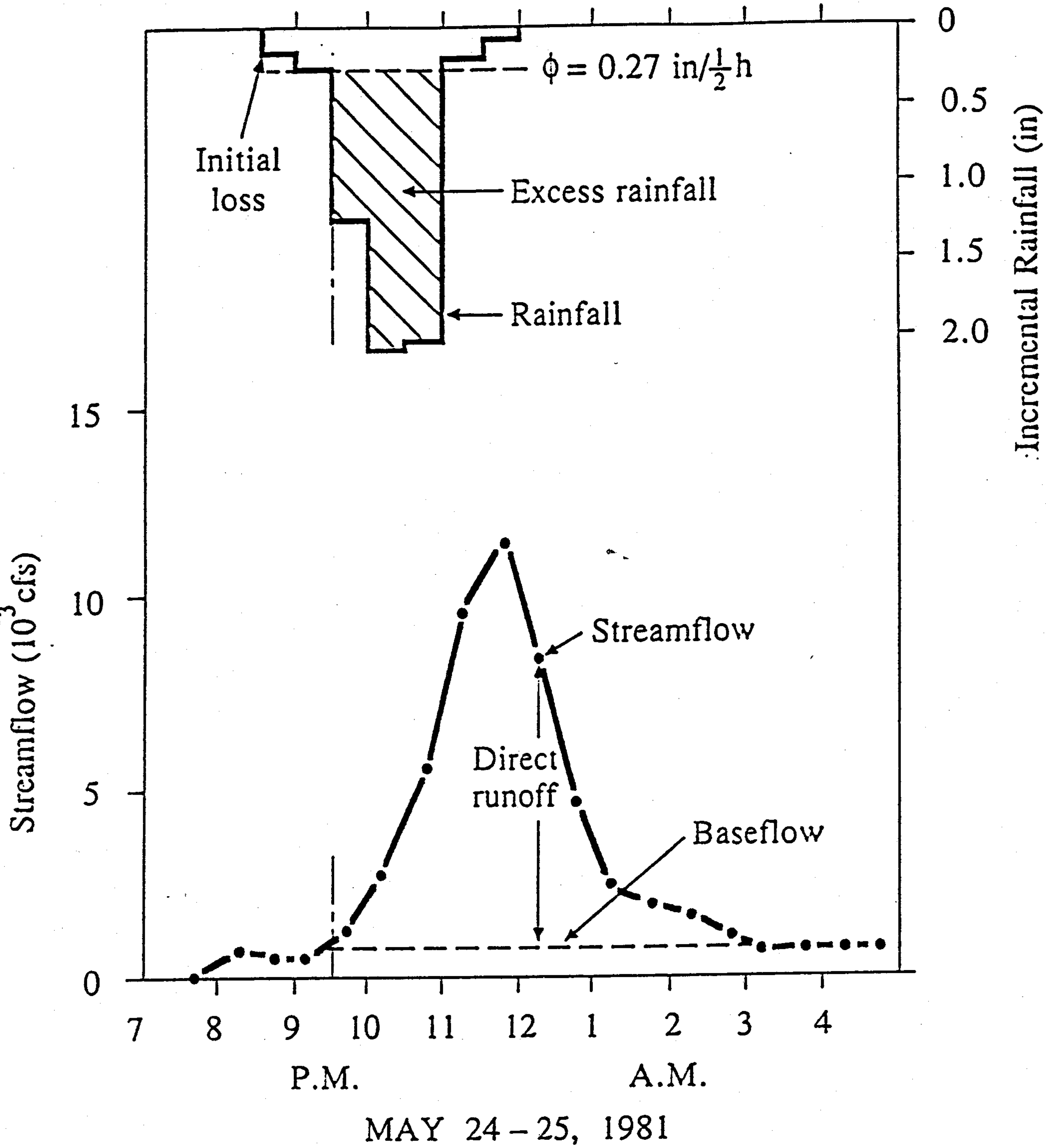


FIGURE 5.3.1

Rainfall and streamflow for the storm of May 24–25, 1981, on Shoal Creel Austin, Texas. (Chow et al., 1988)

2. Routing Hydrographs

* Hydrographs are attenuated as a flood wave moves downstream due to storage effects (Fig 10-1).

* Reservoirs, created to provide storage, reduce the peak flows downstream.

* Several methods exist to route hydrographs through stream reaches (ex. Muskingum).

3. Prediction of Streamflow (rainfall-runoff relationships)

A. Peak flow equations

<u>Rational Method</u>

$$Q_Y = K C_Y I_{tc,Y} A \text{ where}$$

Q_Y = peak flow rate (m^3/s or cfs) for average recurrence interval (ARI) of Y years

C_Y = runoff coefficient (dimensionless) for ARI of Y years

$I_{tc,Y}$ = average rainfall intensity (mm/hr or in/hr) during t_c hours and ARI of Y years

t_c = time of concentration in hours

A = drainage basin area (km^2 or ha or acres; see following K factor)

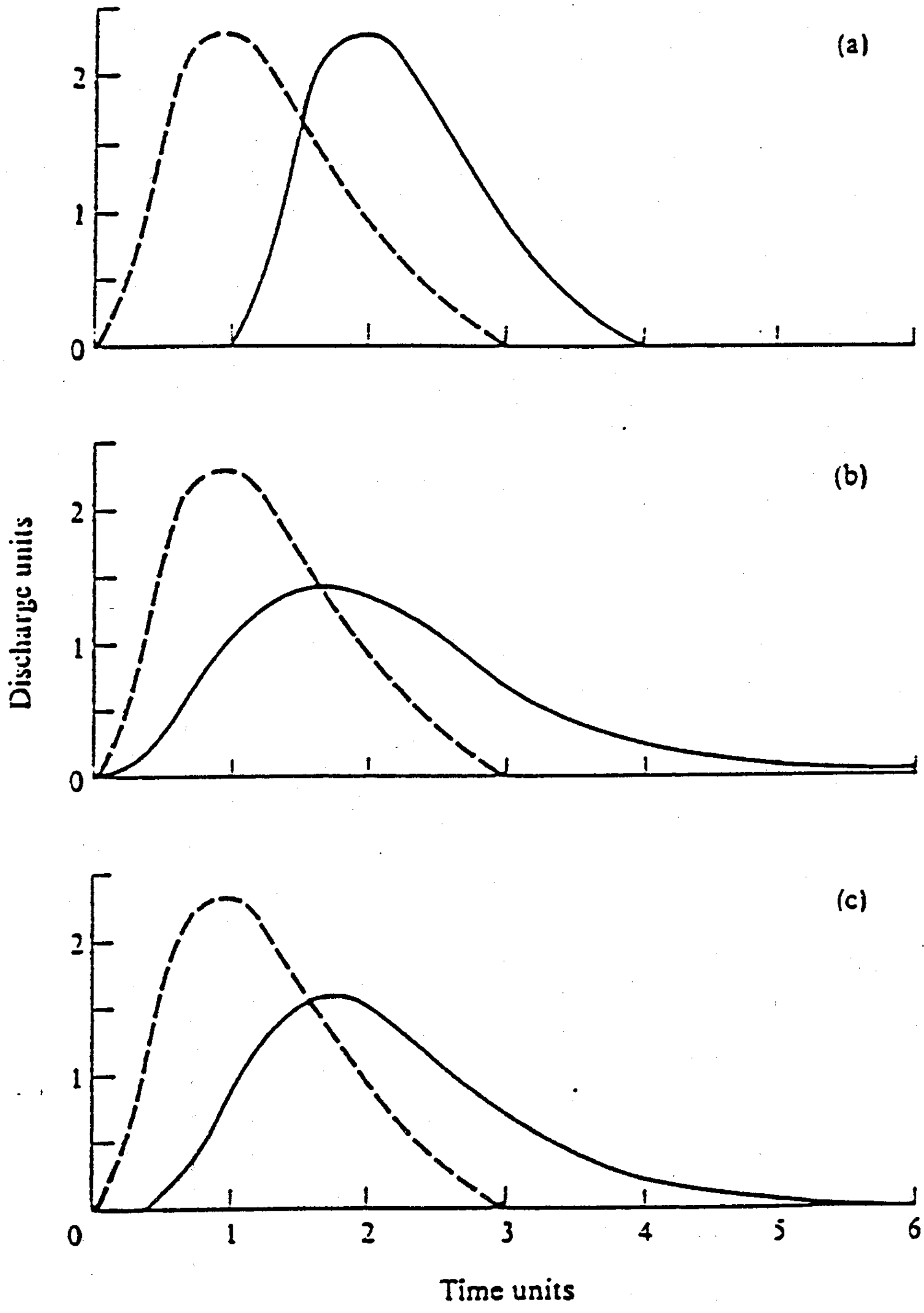


Figure 10-1 Comparison of computed outflow hydrographs (solid line) from a reach of channel for a given inflow (dashed line). The diagrams represent the effects of (a) simple translation or uniform progressive flow, (b) true reservoir action, and (c) "average" river channel storage, a combination of translation and reservoir action. (From W. B. Langbein 1940, *EOS, American Geophysical Union Transactions*, vol. 21, pp. 620–627. Copyrighted by American Geophysical Union.)

K = a unit conversion factor;
= 0.278 for areas in km^2
= 0.00278 for areas in hectares
= 1.0 for areas in acres

* Generally valid for areas less than 25 km^2
* Valid for small areas less than 300 acres
* t_c is equal to time for water from most remote part of basin to reach the outlet (many formulas available to estimate).

Design using the rational method:

1. Estimate t_c
2. Select a frequency or return period for the storm (see later section on frequency analysis)
3. Compute an area-weighted value of C from sources such as Table 15.1.1
4. Calculate or determine the average storm intensity from published intensity-duration-frequency (IDF) curves (e.g., Figure 3.2)
5. Use rational formula to calculate peak discharge
6. Use hydraulic design techniques to size channel or pipe needed to convey the water

Example - Two adjacent fields contribute runoff to a collector whose size is to be determined (see sketch). Use the provided IDF curve (Fig. 3.2) and runoff coefficient chart (Table 15.1.1) to find the peak flow for a 10-year event with a 25-minute rainfall duration.

TABLE 15.1.1
Runoff coefficients for use in the rational method

	Return Period (years)						
Character of surface	2	5	10	25	50	100	500
Developed							
Asphaltic	0.73	0.77	0.81	0.86	0.90	0.95	1.00
Concrete/roof	0.75	0.80	0.83	0.88	0.92	0.97	1.00
Grass areas (lawns, parks, etc.)							
Poor condition (grass cover less than 50% of the area)							
Flat, 0–2%	0.32	0.34	0.37	0.40	0.44	0.47	0.58
Average, 2–7%	0.37	0.40	0.43	0.46	0.49	0.53	0.61
Steep, over 7%	0.40	0.43	0.45	0.49	0.52	0.55	0.62
Fair condition (grass cover on 50% to 75% of the area)							
Flat, 0–2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2–7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, over 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Good condition (grass cover larger than 75% of the area)							
Flat, 0–2%	0.21	0.23	0.25	0.29	0.32	0.36	0.49
Average, 2–7%	0.29	0.32	0.35	0.39	0.42	0.46	0.56
Steep, over 7%	0.34	0.37	0.40	0.44	0.47	0.51	0.58
Undeveloped							
Cultivated Land							
Flat, 0–2%	0.31	0.34	0.36	0.40	0.43	0.47	0.57
Average, 2–7%	0.35	0.38	0.41	0.44	0.48	0.51	0.60
Steep, over 7%	0.39	0.42	0.44	0.48	0.51	0.54	0.61
Pasture/Range							
Flat, 0–2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2–7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, over 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Forest/Woodlands							
Flat, 0–2%	0.22	0.25	0.28	0.31	0.35	0.39	0.48
Average, 2–7%	0.31	0.34	0.36	0.40	0.43	0.47	0.56
Steep, over 7%	0.35	0.39	0.41	0.45	0.48	0.52	0.58

Note: The values in the table are the standards used by the City of Austin, Texas. Used with permission

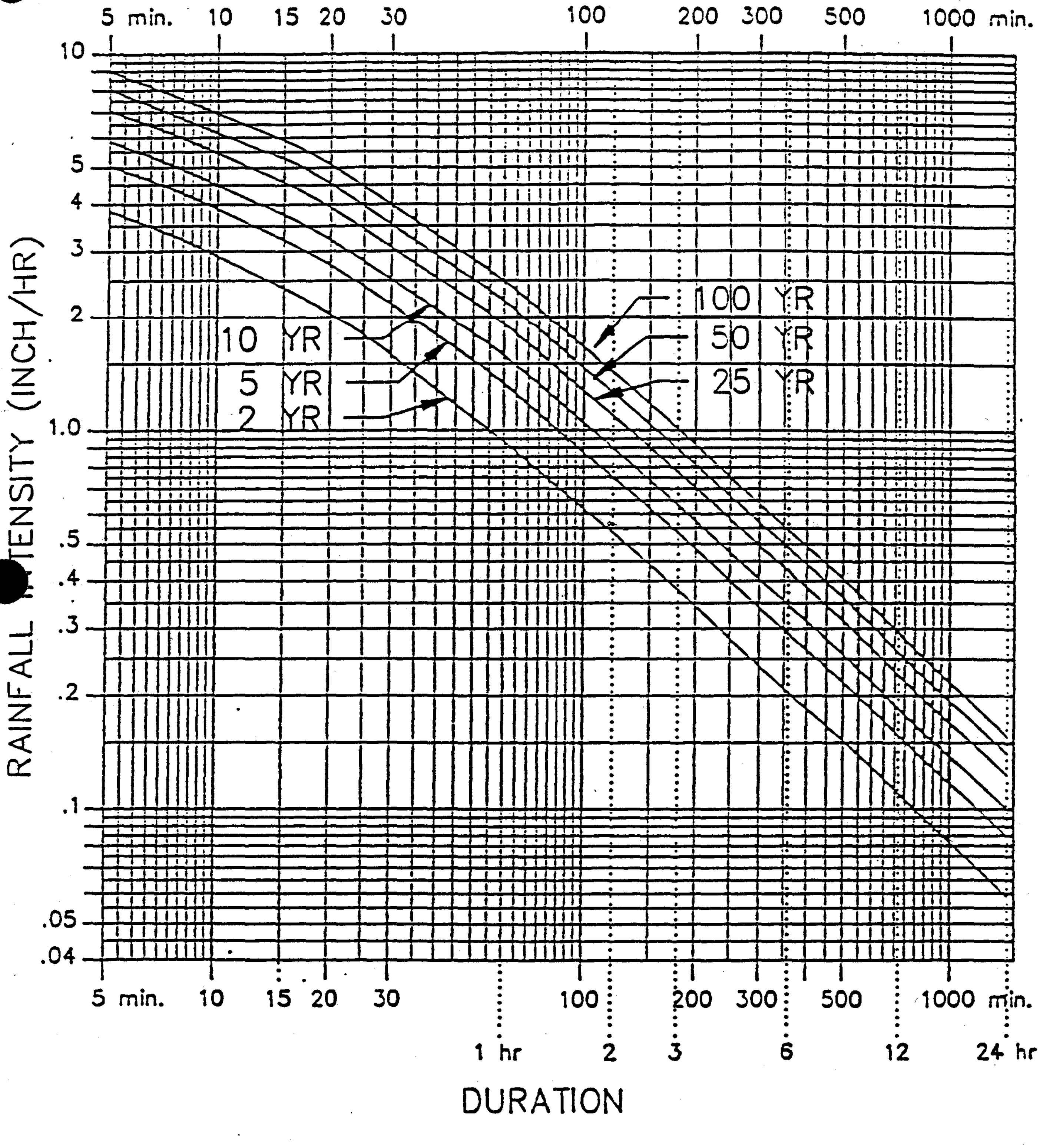
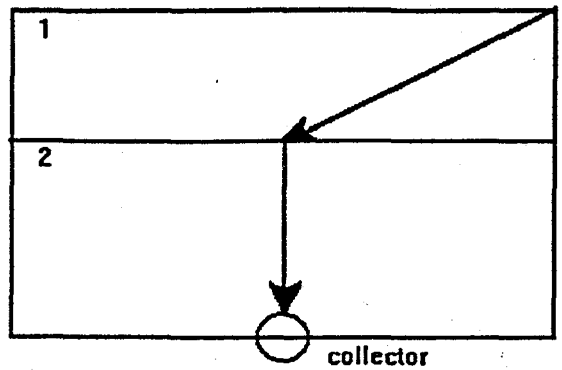


Figure 3.2
Rainfall Intensity-Duration-Frequency Relation
(Phoenix Metro Area)

$A^1 = 2$ ha
$t_c^1 = 15$ min.

$A^2 = 4$ ha
$t_c^2 = 10$ min.



Area 1 is a park with grass covering over 75% of the area, with an average ground slope of 0.05. Area 2 is steep cultivated land.

1. The time for the water from the farthest corner to reach the collector is:
   $t_c = 15 + 10 = 25$ minutes

2. The return period was given as 10 years.

3. From Table 15.1.1, the coefficients for Areas 1 and 2 are 0.35 and 0.44 respectively. Since we want to size the pipe for the total runoff the coefficients are weighted by their contributing areas.

$$C_{10} = \frac{(2)(0.35)+(4)(0.44)}{2+4} = 0.41$$

4. The rainfall intensity for a 25 minute duration can be found from Figure 3.2 and is 2.8 in/hr or 71.1 mm/hr.

5. The total area is 4+2=6 ha. The peak flow is:
   $Q_Y = 0.00278(0.41)(71.1)(6) = \mathbf{0.49\ m^3/s}$

SCS Curve Number Method

* More complicated than rational method
* Any size homogeneous watershed(s)
* Curve numbers from published tables and charts depend on soil type, land use, and soil moisture

USGS Regional Regression Equations

* Typical form : $Q_T = XA^yP^z$ where
  Q_T = peak flow for return period T
  A = Basin area
  P = Mean annual precipitation
  X, Y, Z = numbers determined by regression

* Other parameters often seen in these equations: mean basin elevation, mean annual temperature, basin slope, and channel length

* Usually have basin size limitations

B. Unit hydrograph (UH) methods

Drainage basin with gage at outlet: Direct method

1. Determine excess rainfall volume by measuring area under direct runoff hydrograph.

2. Divide this amount by basin area to get average excess precipitation for that storm.

3. Divide hydrograph points by precipitation to get unit hydrograph (discharge resulting from 1mm [1 inch] of excess rainfall evenly distributed over the basin in a given period of time).

Example - After a 3 hour storm, a gaging station downstream from a 77 km^2 drainage basin measures 254.9 m^3/s as a peak discharge and 3.7E6 m^3 as total runoff. Find the 3 hour unit hydrograph peak discharge. What would be the peak runoff and design flood volume if a 3 hour storm dropped 63.5 mm net (excess) precipitation?

The volume of 1 mm of runoff over 77 km^2 is

$$(0.001 \text{ m})(77E6 \text{ m}^2) = 77{,}000 \text{ m}^3$$

The runoff ratio is 3.7E6/77,000 = 48.052

The unit hydrograph peak discharge is

$$254.9/48.052 = 5.3 \text{ m}^3/\text{s}$$

For a 63.5 mm storm, peak runoff would be

$$(63.5 \text{ mm})(5.3 \text{ m}^3/\text{s}/1 \text{ mm}) = 336.85 \text{ m}^3/\text{s}$$

The design flood would contain

$$(63.5 \text{ mm})(77{,}000 \text{ m}^3/1 \text{ mm}) = 4.89E6 \text{ m}^3$$

UH assumptions

* The excess rainfall has a constant intensity within the effective duration.

* The excess rainfall is evenly distributed over the drainage area.

* The base time is constant for all storms of the given duration.

* The shape of the hydrograph is the same for all storms of the given duration.

* Only the total amount of rainfall varies from storm to storm.

Ungaged drainage basins: synthetic unit hydrographs

* Snyder Unit Hydrograph
* Clark Unit Hydrograph
* SCS Dimensionless Unitgraph

Application of UH's

Each X-depth, N-duration storm (where X is depth [mm, inches] and N is time [min., hrs.]) will produce an excess runoff response in the shape of the N-duration unit hydrogaph with all the ordinates multiplied by X. Superposition of multiple storms will create a runoff hydrograph for the entire sequence of storms (Fig. 4-18).

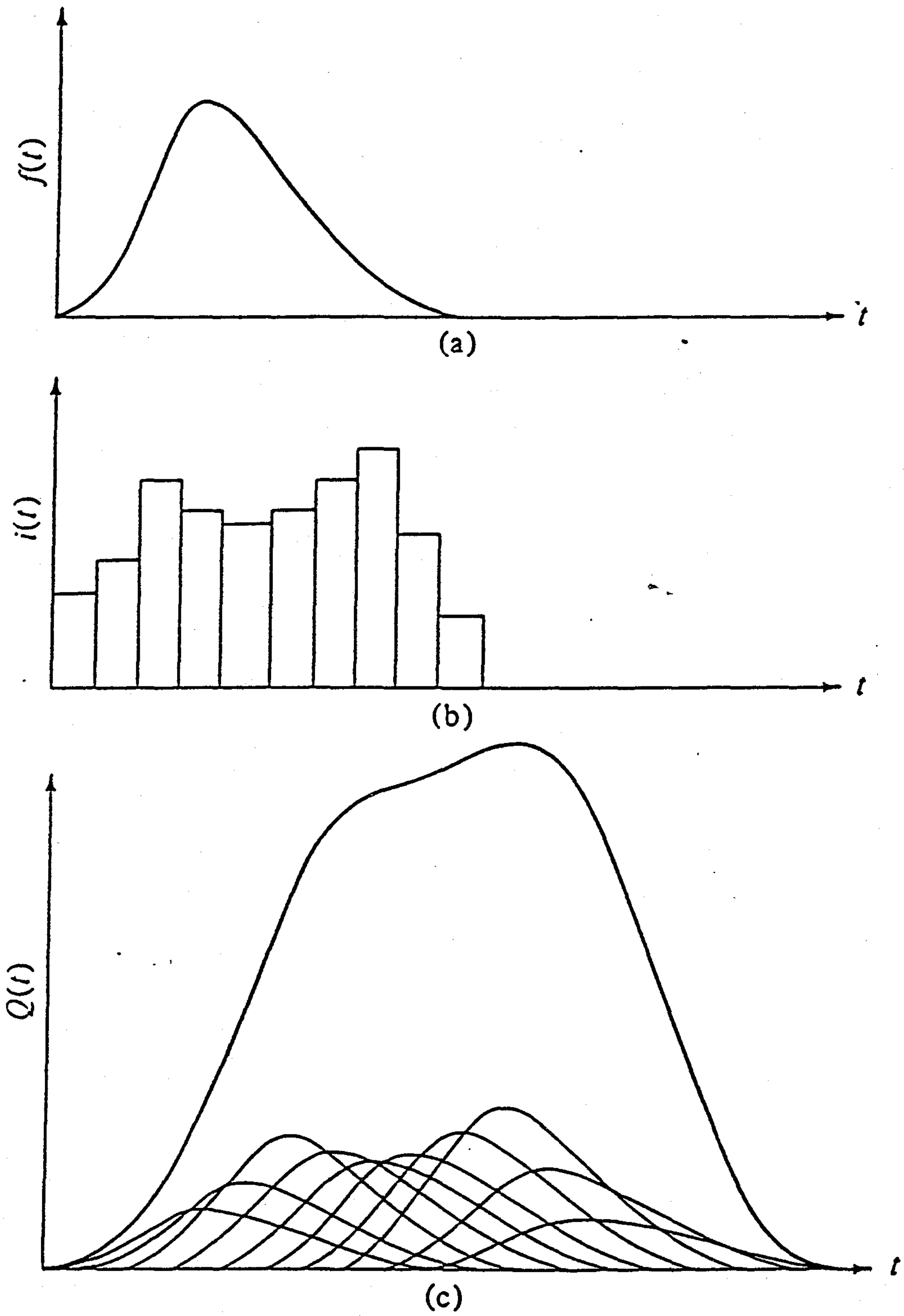


Fig. 4-18. Unit hydrograph description of the runoff process. (a) Unit hydrograph; (b) a sequence of 1-min storms; (c) superposition of runoff hydrographs for each of the 1-min storms. (After John C. Schaake, Jr., "Synthesis of the Inlet Hydrograph," Tech. Rept. No. 3, Department of Sanitary Engineering and Water Resources, Baltimore, Md., 1965.)

Frequency Analysis

1. Hydrologic processes are complex - they are usually described in probabilistic terms.

2. Probability basics:

   On a given coin toss, P(heads) = 0.5
   P(tails) = 0.5

3. What is the "100-year discharge"?

   * Given a probability of an event F occurring in any given year, $P(F)=0.01=1\%$. The average return period (sometimes called recurrence interval) is defined as

   $$T=1/P(F)=1/0.01=100 \text{ years}$$

   * The probability that F will occur in any year is

   $$P(F)=1/T$$

   * The probability that F will <u>not</u> occur in any year is

   $$P(F)=1\text{-}P(F)=1\text{-}1/T$$

   * The probability that F will not occur for n successive years is

   $$P_1(F)xP_2(F)x\ldots xP_n(F)=P(F)^n=(1\text{-}1/T)^n$$

* The probability (**Risk**) that F will occur at least once in n successive years is

$$R = 1-(1-1/T)^n$$

Example - What is the probability of the 100-year storm occurring in 50 years?

$$\begin{aligned} R &= P(\text{100-year storm in 50 years}) \\ &= 1-(1-1/100)^{50} \\ &= 0.395 \\ &= 39.5\% \end{aligned}$$

Example - What is the probability of the 100-year storm occurring in 100 years?

$$\begin{aligned} R &= P(\text{100-year storm in 100 years}) \\ &= 1-(1-1/100)^{100} \\ &= 0.634 \\ &= 63.4\% \end{aligned}$$

4. Frequency of extreme values from gage information

* Several methods use relatively complex statistics

* One simple method uses "plotting positions" (several formulas can be used)

➤ One of most common formulas is by Weibull:

$$P(X \geq x_m) = m/(n+1)$$

where P is an estimate of the probability of values being equal to or less than the ranked value, and m is the rank of a value in a list of n values total.

Example - Construct a frequency distribution for the given rainfall data using the Weibull plotting position method.

Data - Annual rainfall for Richmond, VA, 1906-1928

m	Rainfall (in.)	Rainfall (mm.)	$m/(n+1) \times 100\%$
1	53	1350	4.2
2	52	1320	8.3
3	51	1300	12.5
4	49	1240	16.7
5	49	1240	20.8
6	47	1190	25.0
7	47	1190	29.2
8	44	1120	33.3
9	43	1090	37.5
10	43	1090	41.7
11	43	1090	45.8
12	41	1040	50.0
13	40	1020	54.7
14	38	970	58.3
15	38	970	62.5
16	37	940	66.7
17	37	940	70.8
18	36	910	75.0

19	36	910	79.7
20	34	860	83.3
21	34	860	87.5
22	31	790	91.7
23	31	790	95.8

The points are plotted in Fig. 5-7 as exceedence probability (left-hand scale) versus inches of rainfall.

REFERENCES

Bras, R.L., 1990, *Hydrology: An Introduction to Hydrologic Science*, Addison-Wesley Publishing Co., Reading, MA.

Chow, V.T., Maidment, D.R., and Mays, L.W., 1988, *Applied Hydrology*, McGraw-Hill Publishers, New York, NY.

Haan, C.T., Johnson, H.P., and Brakensiek, D.L. (eds.), 1982, *Hydrologic Modeling of Small Watersheds*, American Society of Agricultural Engineers Monograph No. 5, The American Society of Agricultural Engineers, St. Joseph, MI.

Ponce, V.M. (1989). *Engineering Hydrology*, Prentice Hall, Inc., Englewood Cliffs, NJ.

Viessman, W., Jr., Knapp, J.W., Lewis, G.L., and Harbaugh, T.E. (1977). *Introduction to Hydrology*, Second Edition, Harper & Row, Publishers, New York, NY.

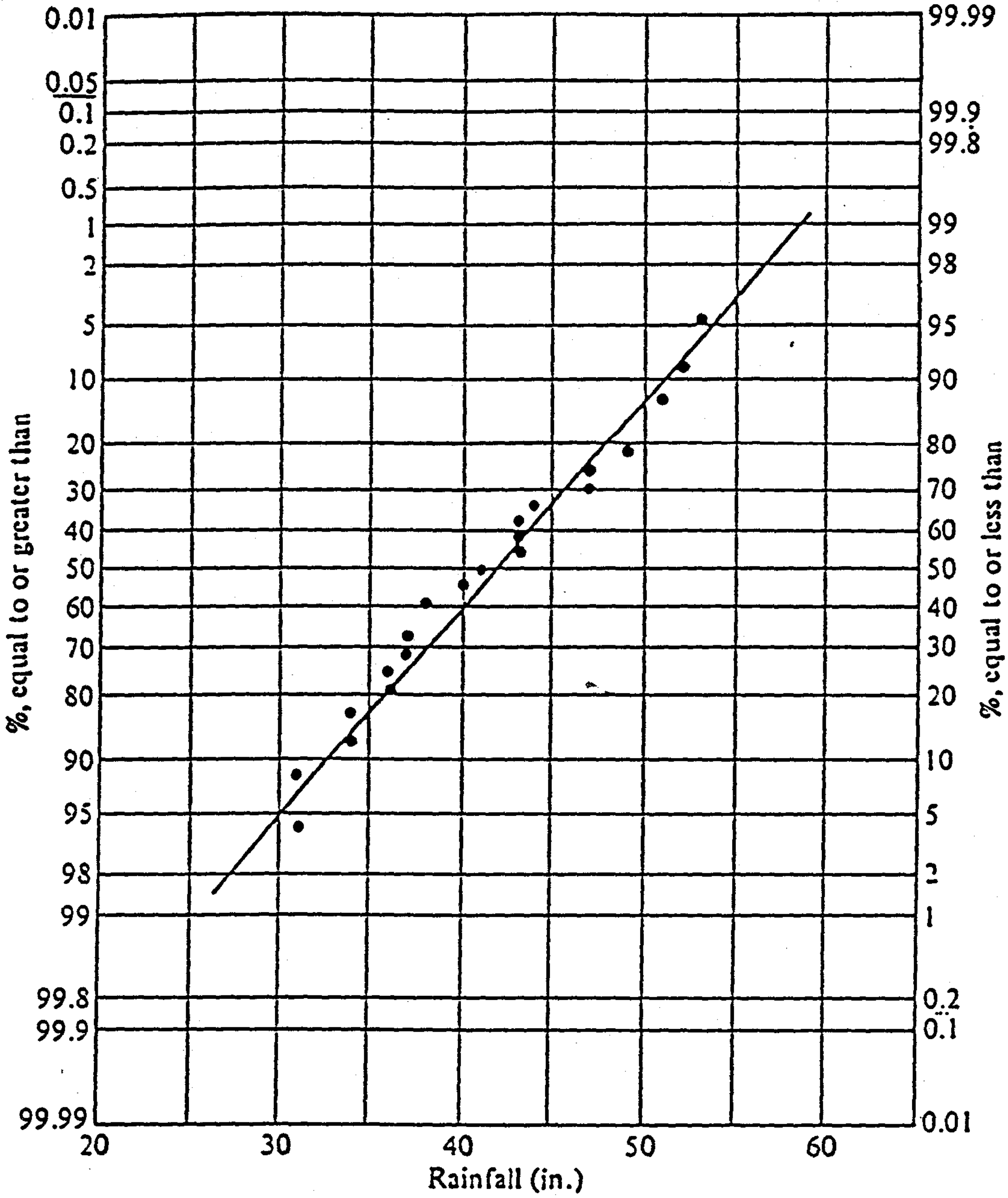


Fig. 5-7. Annual rainfall for Richmond, Virginia, 1906–1928, plotted on normal probability paper.

Required Procedures for Flood Hydrology in Maricopa County

The determination of flood hydrology for designing stormwater facilities in Maricopa County is to be performed according to the procedures set forth in the *Drainage Design Manual for Maricopa County, Volume I, Hydrology* (hereinafter referred to as the *Hydrology Manual*).

Deviations from the procedures in the *Hydrology Manual* require prior approval from the jurisdictional agency and/or the Flood Control District of Maricopa County before proceeding with the determination of design hydrology.

(ref: 2. Flood Control District of Maricopa County. 1996, Volume II, Hydraulics)

Table 2.1
Hydrology Design Criteria (Ref. 2)

Drainage Feature	Peak Frequencies		
	10 Year	50 Year	100 Year
Street with Curb and Gutter (longitudinal flow)	Runoff contained within street curbs. For collector and arterial streets one 12-foot dry driving lane must be maintained in each direction.	N/A	Runoff to be contained below the finished floor of building. Q_{max} = 100 cfs V_{max} = 10 fps d_{max} = 8 inches
Street without Curb and Gutter (longitudinal flow)	Runoff contained within the roadside channels with the water surface elevation below the subgrade.	N/A	Same as Street with Curb and Gutter.
Street with Storm Drain System (longitudinal flow)	Pipes or roadside channels are added if the 10-year runoff exceeds street capacity.	N/A	Storm drains are needed if 100-year runoff inundates the building's finished floor.
Cross Road Culvert for Collector and Arterial Streets	N/A	Runoff to be conveyed by culvert under road with no flow overtopping the road. V_{max} = 15 fps V_{min} = 3.0 fps	Runoff to be conveyed by culvert and by flow over the road with a maximum depth over the road of 6 inches.
FEMA Floodplain Channel (1)	N/A	N/A	100-year peak storm
Channel to Convey Offsite Flow Through Development	N/A	N/A	100-year peak storm
Lowest floor elevation for buildings within a FEMA Floodplain Area	N/A	N/A	Lowest floor elevation to be a minimum of 1 foot above the regulatory flood elevation.
Lowest floor not in a FEMA Designated Floodplain	N/A	N/A	The lowest floor will be free from inundation for the 100-year peak storm event.
Retention Basin	N/A	N/A	100-year 2-hour storm for determining on-site retention volume.

(1) Per ARS 48-3609.A, ADWR has established that during the course of the Master Planning process, the 100-year runoff will be used to delineate a floodplain for major channels with discharges more than 500 cfs and should be processed through the local government, ADWR, and FEMA.

BASICS OF

HYDRAULICS

Lecture 2

TABLE OF CONTENTS

HYDRAULICS OF OPEN CHANNELS

Acknowledgements:

The following material was developed with the assistance of Brian Roberts of Water Resource Consultants, Inc., Fairfax, VA (703) 978-8620.

Types of Flow

Uniform vs. Varied Flow

Uniform Flow - Depth of flow is the same at every section along the length of the channel.

Varied Flow - Depth of flow changes along the length of the channel.

Gradually Varied Flow

Rapidly Varied Flow

Steady vs. Unsteady

Steady Flow - Depth of flow at a given cross section does not change or can be assumed constant during a given time interval.

Unsteady Flow - Depth of flow changes over time at a given cross section.

Steady, Uniform Flow

For many applications we can assume steady, uniform flow for open channel hydraulics. However, this rarely occurs in nature.

Subcritical vs. Supercritical

Subcritical Flow

Relatively deep

Low velocity

Mild slope

Supercritical

Shallow flow

High velocity

Steep slope

Manning's Equation

$$V = (K_c/n)\, R^{2/3}\, S_f^{1/2}$$

where: K_c = 1 for metric, 1.49 for English units

V = average velocity, feet/sec (meters/sec)

n = manning's roughness coefficient

R = hydraulic radius = A/P cross sectional area divided by wetted perimeter, feet (meters)

S_f = friction slope of channel, approximated as average bed slope for uniform flow conditions

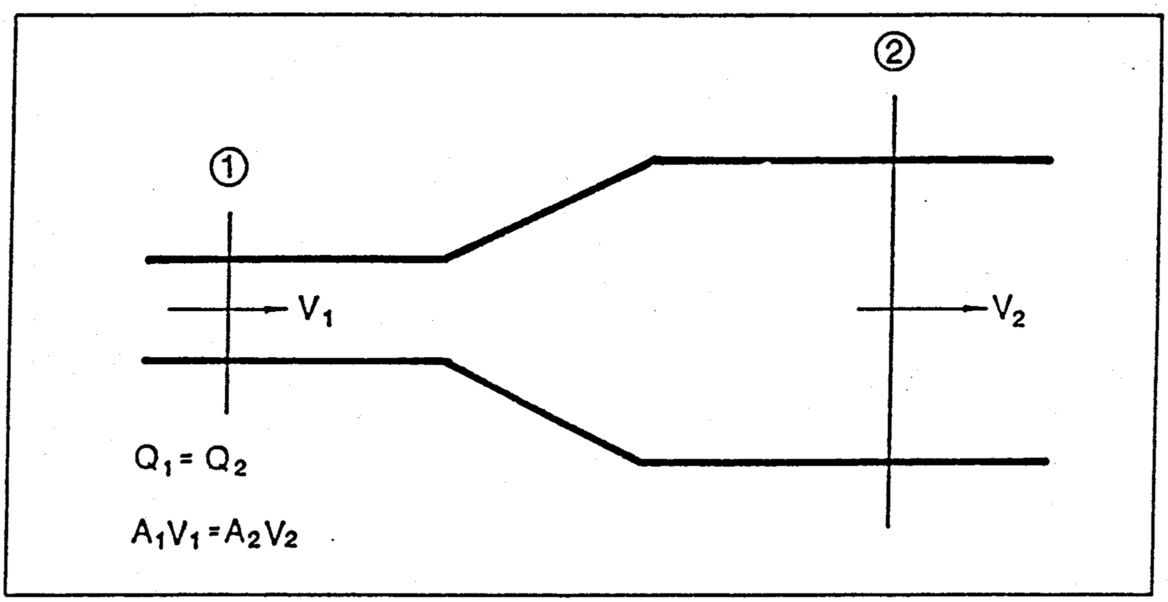
Continuity Equation

$$Q = AV$$

where: Q = Discharge, cfs (cms)

A = Flow area, ft^2 (m^2)

V = Mean velocity, f/s (m/s)



Manning's equation can be combined with the Continuity to compute discharge as:

$$Q = (K_c/n)\ A\ R^{2/3}\ S_f^{1/2}$$

The Manning Equation assumes steady, uniform flow. It can be used to compute **Normal Depth, d_n.**

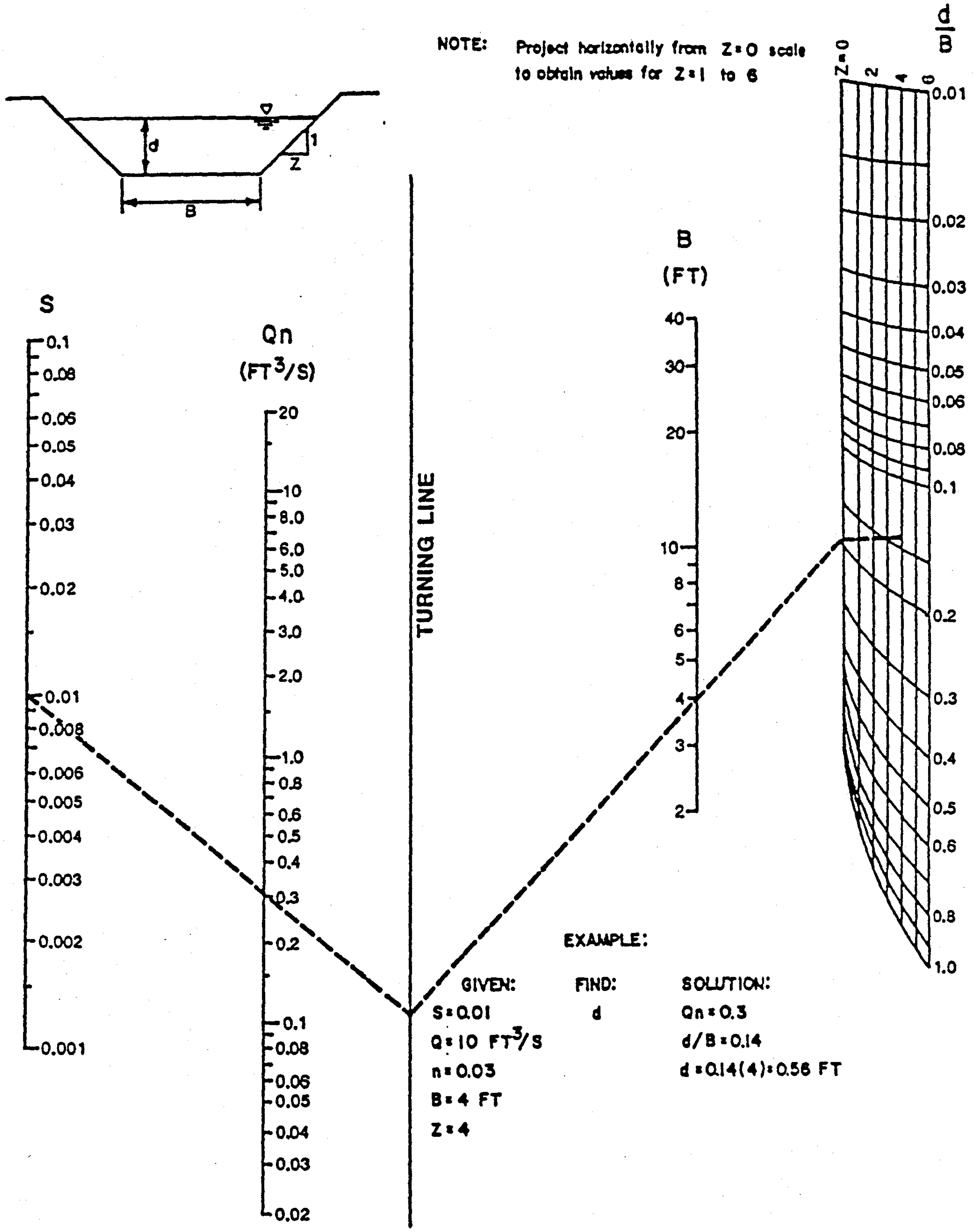


Chart 1: Solution of Manning's Equation for Trapezoidal Channels

Manning n values

Manning's n is generally considered constant.

However, roughness will increase for shallow flow where height of roughness features approaches flow depth (Riprap).

Vegetated Channels

Roughness is a function of height of vegetation and stiffness.

SCS developed classification based on Retardance.

Other studies (Kouwen) have provided equations based on Retardance which is more accurate for very stiff vegetation and mild slopes.

Channel Bends

$$\Delta d = V^2 T/(g R_c)$$

where: V = mean velocity, ft/s (m/s)

T = surface width of channel, ft (m)

g = gravitational acceleration, 32.2 ft/s^2, (9.806 m/s^2)

R_c = mean radius of the bend, ft (m)

Table 1: Manning's Roughness Coefficients (From HDS-3)

	Manning's n range [3]
I. Closed conduits:	
A. Concrete pipe	0.011–0.013
B. Corrugated-metal pipe or pipe-arch:	
1. 2⅔ by ½-in. corrugation (riveted pipe): [3]	
a. Plain or fully coated	0.024
b. Paved invert (range values are for 25 and 50 percent of circumference paved):	
(1) Flow full depth	0.021–0.018
(2) Flow 0.8 depth	0.021–0.016
(3) Flow 0.6 depth	0.019–0.013
2. 6 by 2-in. corrugation (field bolted)	0.03
C. Vitrified clay pipe	0.012–0.014
D. Cast-iron pipe, uncoated	0.013
E. Steel pipe	0.009–0.011
F. Brick	0.014–0.017
G. Monolithic concrete:	
1. Wood forms, rough	0.015–0.017
2. Wood forms, smooth	0.012–0.014
3. Steel forms	0.012–0.013
H. Cemented rubble masonry walls:	
1. Concrete floor and top	0.017–0.022
2. Natural floor	0.019–0.025
I. Laminated treated wood	0.015–0.017
J. Vitrified clay liner plates	0.015
II. Open channels, lined [4] (straight alinement): [5]	
A. Concrete, with surfaces as indicated:	
1. Formed, no finish	0.013–0.017
2. Trowel finish	0.012–0.014
3. Float finish	0.013–0.015
4. Float finish, some gravel on bottom	0.015–0.017
5. Gunite, good section	0.016–0.019
6. Gunite, wavy section	0.018–0.022
B. Concrete, bottom float finished, sides as indicated:	
1. Dressed stone in mortar	0.015–0.017
2. Random stone in mortar	0.017–0.020
3. Cement rubble masonry	0.020–0.025
4. Cement rubble masonry, plastered	0.016–0.020
5. Dry rubble (riprap)	0.020–0.030
C. Gravel bottom, sides as indicated:	
1. Formed concrete	0.017–0.020
2. Random stone in mortar	0.020–0.023
3. Dry rubble (riprap)	0.023–0.033
D. Brick	0.014–0.017
E. Asphalt:	
1. Smooth	0.013
2. Rough	0.016
F. Wood, planed, clean	0.011–0.013
G. Concrete-lined excavated rock:	
1. Good section	0.017–0.020
2. Irregular section	0.022–0.027
III. Open channels, excavated [4] (straight alinement, [5] natural lining):	
A. Earth, uniform section:	
1. Clean, recently completed	0.016–0.018
2. Clean, after weathering	0.018–0.020
3. With short grass, few weeds	0.022–0.027
4. In gravelly soil, uniform section, clean	0.022–0.025
B. Earth, fairly uniform section:	
1. No vegetation	0.022–0.025
2. Grass, some weeds	0.025–0.030
3. Dense weeds or aquatic plants in deep channels	0.030–0.035
4. Sides clean, gravel bottom	0.025–0.030
5. Sides clean, cobble bottom	0.030–0.040
C. Dragline excavated or dredged:	
1. No vegetation	0.028–0.033
2. Light brush on banks	0.035–0.050
D. Rock:	
1. Based on design section	0.035
2. Based on actual mean section:	
a. Smooth and uniform	0.035–0.040
b. Jagged and irregular	0.040–0.045
E. Channels not maintained, weeds and brush uncut:	
1. Dense weeds, high as flow depth	0.08–0.12
2. Clean bottom, brush on sides	0.05–0.08
3. Clean bottom, brush on sides, highest stage of flow	0.07–0.11
4. Dense brush, high stage	0.10–0.14

	Manning's n range [1]
IV. Highway channels and swales with maintained vegetation [1,7] (values shown are for velocities of 2 and 6 f.p.s.):	
A. Depth of flow up to 0.7 foot:	
1. Bermudagrass, Kentucky bluegrass, buffalograss:	
a. Mowed to 2 inches	0.07–0.045
b. Length 4–6 inches	0.09–0.05
2. Good stand, any grass:	
a. Length about 12 inches	0.18–0.09
b. Length about 24 inches	0.30–0.15
3. Fair stand, any grass:	
a. Length about 12 inches	0.14–0.08
b. Length about 24 inches	0.25–0.13
B. Depth of flow 0.7–1.5 feet:	
1. Bermudagrass, Kentucky bluegrass, buffalograss:	
a. Mowed to 2 inches	0.05–0.035
b. Length 4 to 6 inches	0.06–0.04
2. Good stand, any grass:	
a. Length about 12 inches	0.12–0.07
b. Length about 24 inches	0.20–0.10
3. Fair stand, any grass:	
a. Length about 12 inches	0.10–0.06
b. Length about 24 inches	0.17–0.09
V. Street and expressway gutters:	
A. Concrete gutter, troweled finish	0.012
B. Asphalt pavement:	
1. Smooth texture	0.013
2. Rough texture	0.016
C. Concrete gutter with asphalt pavement:	
1. Smooth	0.013
2. Rough	0.015
D. Concrete pavement:	
1. Float finish	0.014
2. Broom finish	0.016
E. For gutters with small slope, where sediment may accumulate, increase above values of n by	*0.002*
VI. Natural stream channels: [8]	
A. Minor streams [9] (surface width at flood stage less than 100 ft.):	
1. 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	0.035–0.05
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	*0.01–0.02*
2. Irregular sections, with pools, slight channel meander; increase values given in 1a–e about	*0.01–0.02*
3. 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
B. Flood plains (adjacent to natural streams):	
1. Pasture, no brush:	
a. Short grass	0.030–0.035
b. High grass	0.035–0.05
2. 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
3. Heavy weeds, scattered brush	0.05–0.07
4. Light brush and trees: [10]	
a. Winter	0.05–0.06
b. Summer	0.06–0.08
5. Medium to dense brush: [10]	
a. Winter	0.07–0.11
b. Summer	0.10–0.16
6. Dense willows, summer, not bent over by current	0.15–0.20
7. 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
8. 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
C. Major streams (surface width at flood stage more than 100 ft.): Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Values of n may be somewhat reduced. Follow recommendation in publication cited [8] if possible. The value of n for larger streams of most regular section, with no boulders or brush, may be in the range of	0.028–0.033

Freeboard

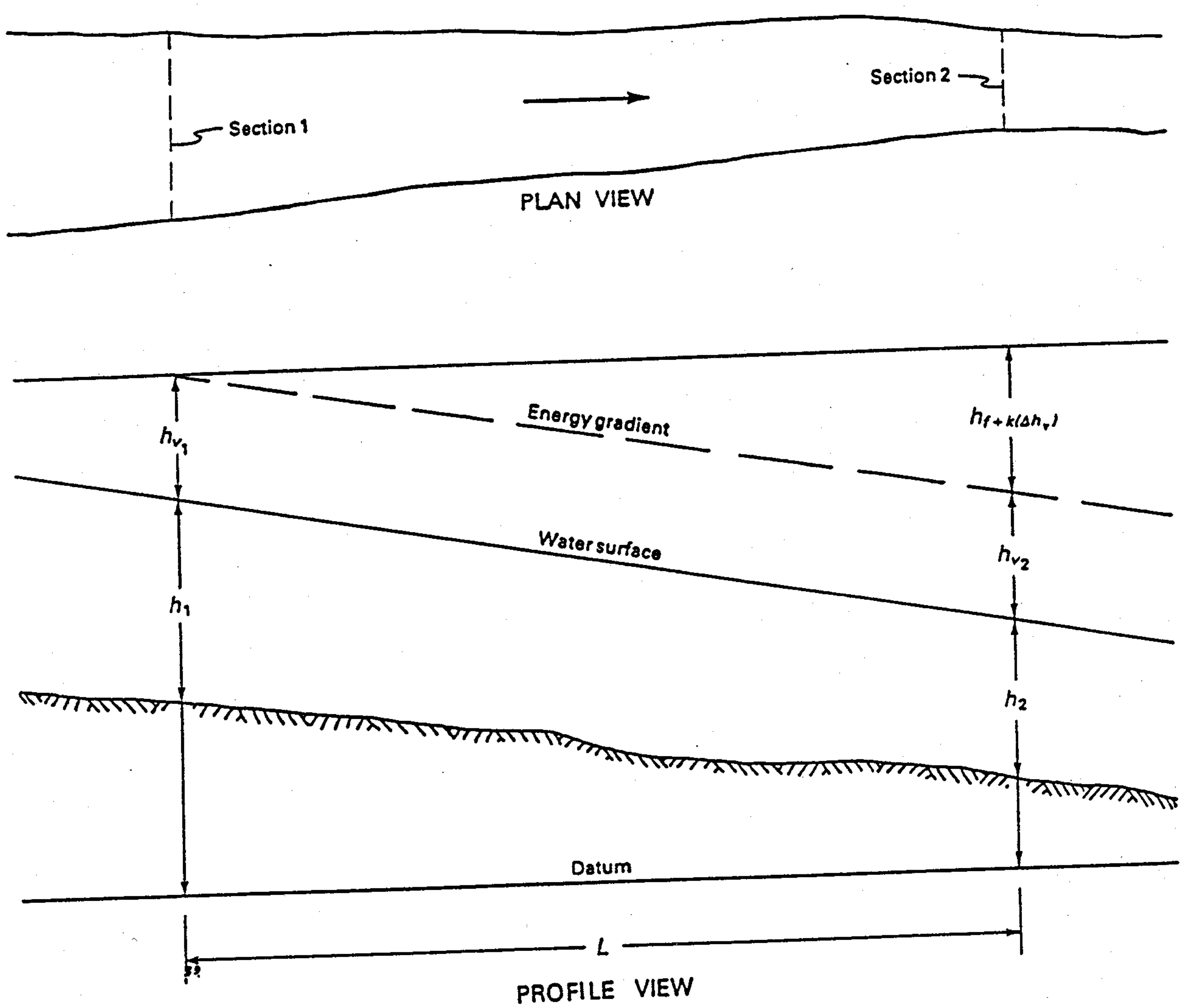
Vertical distance from water surface to top of channel for a design condition.

For permanent channels, minimum 0.5 feet to 1.0 feet (0.15 to 0.31 m)

For temporary channels, no freeboard.

For steep channels, freeboard up to flow depth.

Water Surface Profile Computations



Energy at Section 1 = Energy at Section 2 + Losses

Energy Equation

$$(h + h_v)_1 = (h + h_v)_2 + (h_f)_{1\text{-}2} + k(\Delta h_v)_{1\text{-}2}$$

where:

h = elevation of the water surface at a particular cross section

h_v = velocity head = $\alpha V^2/2g$

h_f = energy loss due to boundary friction

Δh_v = upstream velocity head minus downstream velocity head

$k(\Delta h_v)$ = energy loss due to contraction and expansions

k = coefficient for expansion and contraction

Friction loss

$$h_f = LQ_2/K_1K_2$$

where:

L = flow distance through the subreach, ft (m)

Q = total discharge, cfs (cms)

K = conveyance at the cross section = $(K_c/n)\ A\ R^{2/3}$

Velocity head

$$h_v = \alpha V^2/2g$$

where:

V = mean velocity in the section, f/s (m/s)

α = velocity head coefficient = 1.0 if the cross section is not subdivided

$= \Sigma (k_i^3/a_i^2)/(K_T^3/A_T^2)$

where:

k_i = conveyance of the sub section

a_i = area of the subsection

K_T = total conveyance of the section

A_T = total area of the section

Standard Step Method (Subcritical Flow)

1. Determine discharge for water surface profile.

2. Determine channel geometry, roughness, subdivisions, and subreach lengths.

3. Choose the water surface elevation, h_2, at the downstream end.

4. For the value of h_2 chosen, compute the corresponding area, conveyance, velocity head, and α values.

5. Assume a water surface elevation, h_1, for the upstream cross section.

6. For the value of h_1 chosen, compute the corresponding area, conveyance, velocity head, and a values.

7. Compute the friction loss between sections 1 and 2
$(h_f)_{1-2} = LQ^2/K_1K_2$

8. Determine the coefficient, K.

9. Solve the energy equation. If the result is acceptably balanced, go to step 12. If not, proceed to step 10.

10. If the energy equation is not balanced within an acceptable tolerance, choose a new value for the upstream water surface elevation, h_1.

11. Repeat steps 5 through 10 until the energy equation is satisfactorily balanced.

12. The solution moves one subreach upstream. The value of h_1 at the upstream reach becomes the value of h_2 at the downstream reach.

13. Repeat steps 4 - 12 for each subreach until the entire water surface profile has been computed.

Permissible Velocities for Channels with Erodible Linings

Soil Type	Clear Water ft/s (m/s)	Water Carrying Fine Silts, ft/s (m/s)	Water Carrying Sand and Gravel, ft/s (m/s)
Fine sand (noncolloidal)	1.5 (0.46)	2.5 (0.76)	1.5 (0.46)
Sandy loam (noncolloidal)	1.7 (0.52)	2.5 (0.76)	2.0 (0.61)
Silt loam (noncolloidal)	2.0 (0.61)	3.0 (0.91)	2.0 (0.61)
Ordinary firm loam	2.5 (0.76)	3.5 (1.07)	2.2 (0.67)
Volcanic ash	2.5 (0.76)	3.5 (1.07)	2.0 (0.61)
Fine gravel	2.5 (0.76)	5.0 (1.52)	3.7 (1.13)
Stiff clay (very colloidal)	3.7 (1.13)	5.0 (1.52)	3.0 (0.91)
Graded, loam to cobbles (noncolloidal)	3.7 (1.13)	5.0 (1.52)	5.0 (1.52)
Graded, silt to cobbles (colloidal)	4.0 (1.22)	5.5 (1.68)	5.0 (1.52)
Alluvial silts (noncolloidal)	2.0 (0.61)	3.5 (1.07)	2.0 (0.61)
Alluvial silts (colloidal)	3.7 (1.13)	5.0 (1.52)	3.0 (0.91)
Coarse gravel (noncolloidal)	4.0 (1.22)	6.0 (1.83)	6.5 (1.98)
Cobbles and shingles	5.0 (1.52)	5.5 (1.68)	6.5 (1.98)
Shales and hard pans	6.0 (1.83)	6.0 (1.83)	5.0 (1.52)

From HDS-3, "Design Charts for Open Channel Flow."

PERMISSIBLE VELOCITIES FOR GRASS-LINED CHANNELS

Channel Slope	Lining	Velocity*, ft/s (m/s)
0 - 5%	Bermudagrass	6 (1.83)
	Reed canarygrass Tall fescue Kentucky Bluegrass	5 (1.52)
	Grass-legume	4 (1.22)
	Red Fescue, Redtop Sericea lespedeza Annual Lespedeza Small grains Temporary vegetation	2.5 (0.76)
5 - 10%	Bermudagrass	5 (1.52)
	Reed canarygrass Tall fescue Kentucky bluegrass	4 (1.22)
	Grass-legume	3 (0.91)
Greater than 10%	Bermudagrass	4 (1.22)
	Reed Canarygrass Tall fescue Kentucky bluegrass	3 (0.91)

* For highly erodible soils, decrease permissible velocities by 25%

Source: Soil and Water Conservation Engineering, Schwab, et al. and American Society of Civil Engineers

STABLE CHANNEL DESIGN

The procedures discussed in this section pertain to the design of flexible channel linings. The riprap design procedures are for maximum discharges of 50 cfs. For larger discharges, see the section entitled Design of Riprap Revetment.

Types of Flexible Linings

Vegetated Linings

Temporary Linings

Riprap

Gabions

Channel Stability

Rigid (static) boundaries vs. moveable (dynamic) boundaries

Tractive Force Theory

Average Tractive Force (shear stress)

$$\tau = \gamma R S$$

where:

γ = unit weight of water, 62.4 lb/ft^3 (1000 kg/m^3)

R = hydraulic radius, ft (m)

S = average bed slope or energy slope

Maximum shear stress

$$\tau_d = \gamma \, d \, S$$

where $\quad d$ = maximum flow depth

Shear Stress distribution

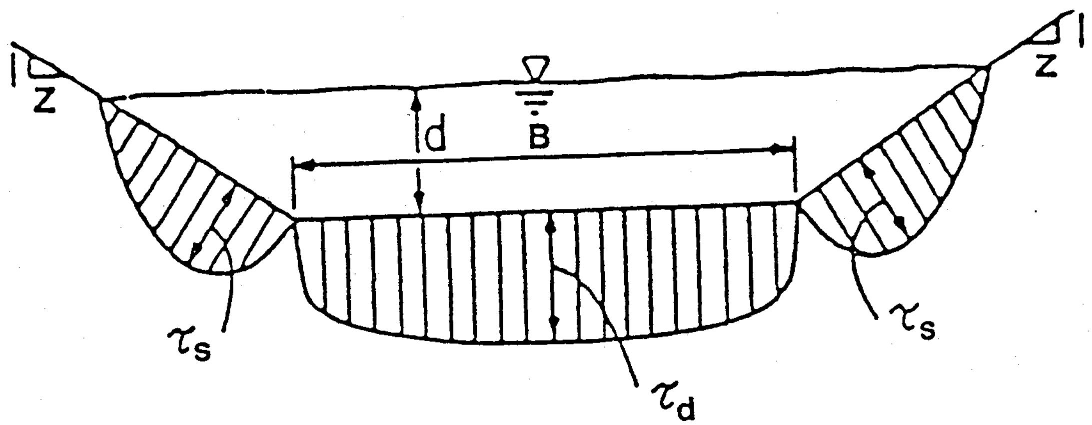


Figure 8: Typical Shear Stress Distribution (From HEC-15)

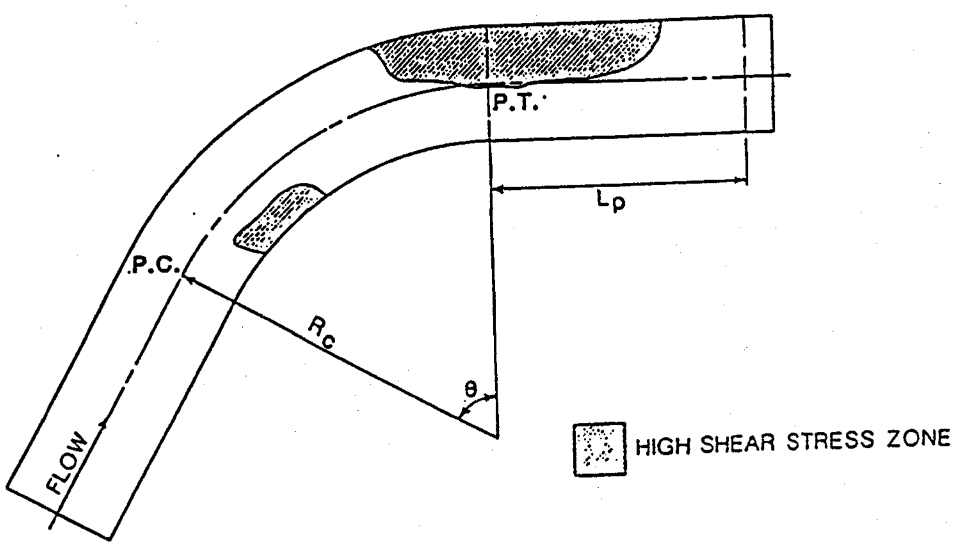


Figure 9: Location of High Shear Stress in Channel Bend (From HEC-15)

Bend Shear Stress

$$T_b = K_b \, T_d$$

where K_b is determined from Chart 10 as a function of R_c/B

where:

R_c = Channel curvature, ft (m)

B = Bottom width, ft (m)

The increased shear stress due to the bend extends a distance L_p downstream of the bend.

Side Slope Stability

As the side slope becomes steeper than 3:1 and approaches the angle of repose of the material, the side slope becomes less stable for riprap. The mean diameter of the stone, D_{50}, for the sides is a function of the bottom size stone.

$$(D_{50})_{sides} = K_1/K_2 \; (D_{50})_{bottom}$$

where:

K_1 = Ratio of shear stress on the sides and bottom (Chart 13)

K_2 = Tractive force ratio (Chart 14)

Flexible Lining Design Procedure

Step 1: Select a flexible lining and determine the permissible shear stress, τ_p, from Table 3.

Step 2: Estimate flow depth, d_i, for vegetation or flow depth range for non-vegetative linings, the channel shape, slope and design discharge(s).

Step 3: Determine the Manning's n value for estimated flow depth.

a. For non-vegetative linings, use Table 4.

b. For vegetation

(1) Calculate the hydraulic radius, R.

(2) Determine n from Charts 4 - 9.

Step 4: Calculate the flow depth, d, in the channel.

Step 5: Compare computed flow depth, d, with estimated flow depth, d_i. If d is outside the assumed range for non-vegetative linings or differs by more than 0.1 feet (say 0.03 m) from d_i for vegetation, repeat steps 2 through 4.

Step 6: Calculate the shear stress, τ_d. If $\tau_d > \tau_p$, the lining is not acceptable, repeat steps 1 through 5.

$$\tau_d = \gamma dS$$

Step 7: For channel bends:

a. Determine the factor for maximum shear stress on channel bends, K_b, from Chart 10. This is a function of the ratio of channel curvature to bottom width, R_c/B.

b. Calculate the shear stress in the bend, τ_b.

$$\tau_b = K_b \, \tau_d$$

If $\tau_b > \tau_p$, the lining is not acceptable, repeat steps 1 through 7.

c. Calculate the length of protection, L_p, downstream of the bend from Chart 11.

d. Calculate the superelevation

$$\Delta d = V^2 \, T/(g \, R_c)$$

Step 8: For riprap or gravel linings with steep side slopes (steeper than 3:1):

a. Determine the angle of repose for the rock size and shape from Chart 12.

b. Determine K_1, the ratio of maximum side shear to maximum bottom shear from Chart 13.

c. Determine K_2, the tractive force ratio from Chart 14.

d. Calculate the required D_{50} for the side slopes.

$$(D_{50})_{sides} = (K_1/K_2) \, (D_{50})_{bottom}$$

Step 9: For riprap on slopes steeper than 10%, check steep slope design procedure.

DESIGNER: ______________________________ DATE: ______________

PROJECT: ____

STATION: ____

DRAINAGE AREA: __________________ Acres (km^2)

DESIGN FLOW: Q ____ = ___________ ft^3/s (m^3/s)

DESIGN FLOW FOR TEMPORARY LINING: Q ____ = __________ ft^3/s (m^3/s)

CHANNEL SLOPE (S): ____________ ft/ft (m/m)

Lining	Q	τ_p (1)	d_i (2)	R (3)	n (4)	d (5)	$\tau_d = \gamma RS$ (6)	REMARKS

(1) Table 3
(2) For vegetation, estimate initial depth
For liners, select range from Table 4
(3) Vegetation only, Chart 2 for trapezoid channels
(4) For vegetation, Charts 5 - 9
For other liners, Table 4
(5) Normal depth, Chart 1 (d must be in d_i range)
(6) τ_d must be $\leq \tau_p$
(7) Check for steep sides slopes and channel bends

Figure 10: Worksheet for Flexible Lining Design

Table 2: Classification of Vegetal Covers as to Degree of Retardance (HEC-15)

Retardance Class	Cover	Condition
A	Weeping lovegrass	Excellent stand, tall (ave. 30") (76 cm)
	Yellow bluestem Ishcheemum	Excellent stand, tall (ave. 36") (91 cm)
B	Kudzu	Very dense growth, uncut
	Bermuda grass	Good stand, tall (ave. 12") (30 cm)
	Native grass mixture (little bluestem, bluestem, blue grass, blue gamma and other long and short midwest grasses)	Good stand, unmowed
	Weeping lovegrass	Good stand, tall (ave. 24") (61 cm)
	Lespedeza sericea	Good stand, not woody, tall (ave. 19") (48 cm)
	Alfalfa	Good stand, uncut (ave. 11") (28 cm)
	Weeping lovegrass	Good stand, unmowed (ave. 13") (33 cm)
	Kudzu	Dense growth, uncut
	Blue gamma	Good stand, uncut (ave. 13") (28 cm)
C	Crabgrass	Fair stand, uncut (10 to 48") (25 to 120 cm)
	Bermuda grass	Good stand, mowed (ave. 6") (15 cm)
	Common lespedeza	Good stand, uncut (ave. 11") (28 cm)
	Grass-legume mixture summer (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (6 to 8") (15 to 20 cm)
	Centipedegrass	Very dense cover (ave. 6") (15 cm)
	Kentucky bluegrass	Good stand, headed (6 to 12 ") (15 to 30 cm)
D	Bermuda grass	Good stand, cut to 2.5 inch height (6 cm)
	Common lespedeza	Excellent stand, uncut (ave. 4.5") (11 cm)
	Buffalo grass	Good stand, uncut (3 to 6") (8 to 15 cm)
	Grass-legume mixture fall, spring (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (4 to 5") (10 to 13 cm)
	Lespedeza sericea	After cutting to 2 inch height (5 cm) Very good stand before cutting
E	Bermuda grass	Good stand, cut to 1.5 inch height (4 cm)
	Bermuda grass	Burned stubble

Table 3: Permissible Shear Stresses for Lining Materials (From HEC-15)

Lining Category	Lining Type	Permissible Unit Shear Stress lb/ft	Permissible Unit Shear Stress Kg/m^2
Temporary	Woven paper net	0.15	0.73
	Jute net	0.45	2.20
	Fiberglass roving		
	single	0.60	2.93
	double	0.85	4.15
	Straw with net	1.45	7.08
	Curled wood mat	1.55	7.57
	Synthetic mat	2.00	9.76
Vegetative	Class A	3.70	18.06
	Class B	2.10	10.25
	Class C	1.00	4.88
	Class D	0.60	2.93
	Class E	0.35	1.71
Gravel riprap	1 - inch (2.5 cm)	0.33	1.61
	2 - inch (5.0 cm)	0.67	3.22
Rock riprap	6 - inch (15.2 cm)	2.00	9.76
	12 - inch (30.5 cm)	4.00	19.52
Bare soil	Non-cohesive	See Chart 3	
	Cohesive	See Chart 4	

Table 4: Manning's Roughness Coefficients (From HEC-15)

Lining Category	Lining Type	n values[1] Depth Ranges 0 - 0.5 ft 0 - 15 cm	0.5 - 2.0 ft 0.5 - 60 cm	> 2.0 ft > 60 cm
Rigid	Concrete	0.015	0.013	0.013
	Grouted riprap	0.040	0.030	0.028
	Stone masonry	0.042	0.032	0.030
	Soil cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare soil	0.023	0.020	0.020
	Rock cut	0.045	0.035	0.025
Temporary*	Woven paper net	0.016	0.015	0.015
	Jute net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.021	0.019
	Straw with net	0.065	0.033	0.025
	Curled wood mat	0.066	0.035	0.028
	Synthetic mat	0.036	0.025	0.021
Gravel riprap	1 - inch (2.5 cm)	0.044	0.033	0.030
	2 - inch (5 cm)	0.066	0.041	0.034
Rock riprap	6-inch (15 cm) D_{50}	0.104	0.069	0.035
	12-inch (30 cm) D_{50}	--	0.078	0.040

[1] Based on data in (5, 8, 13, 14, and 15)

Note: Values listed are representative values for the respective depth ranges. Manning's roughness coefficients, n, vary with the flow depth. See Appendix B.

* Some "temporary" linings becomes permanent when buried

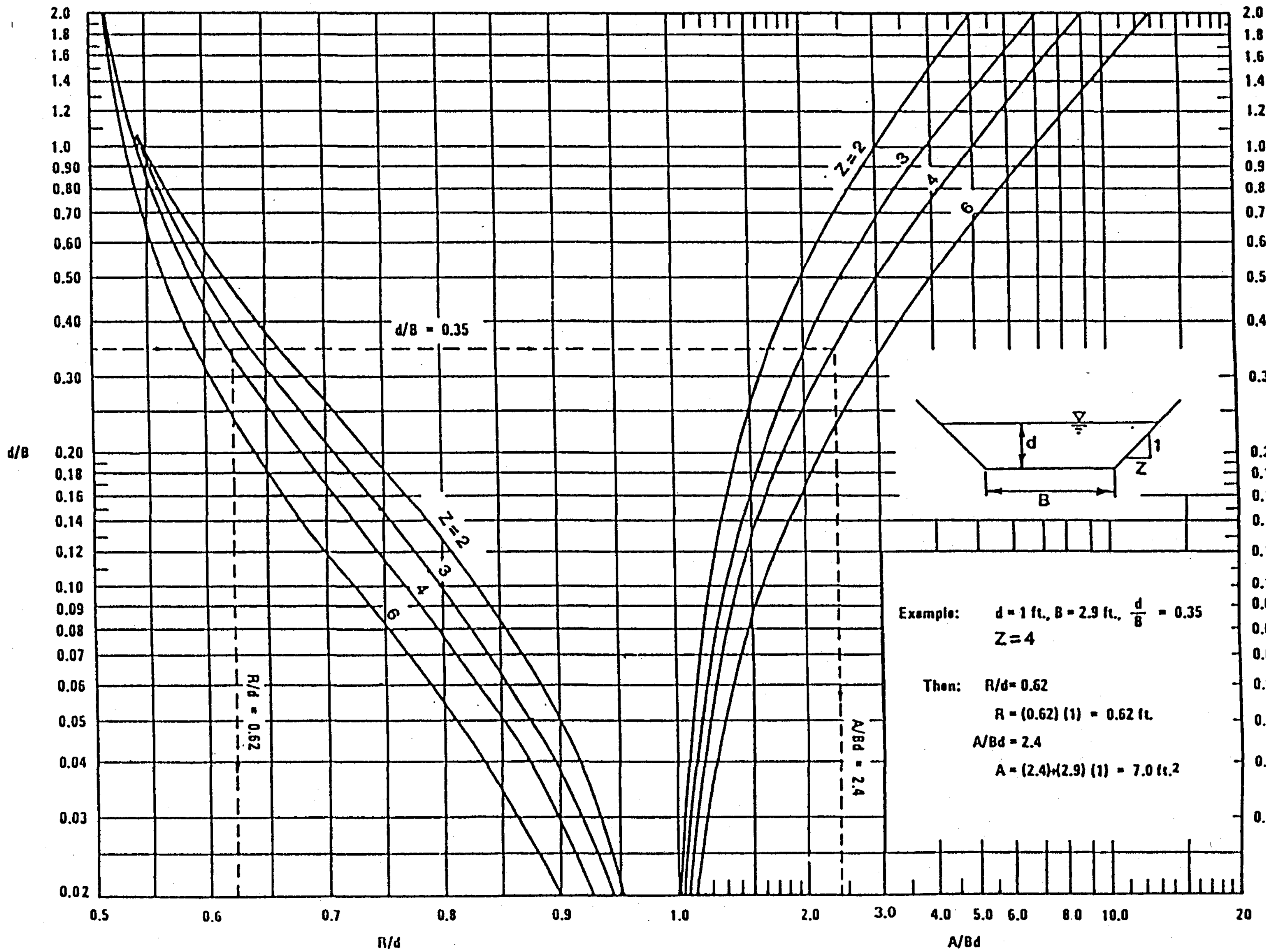


Chart 2: Geometric Design Chart for Trapezoidal Channels (From HEC-15)

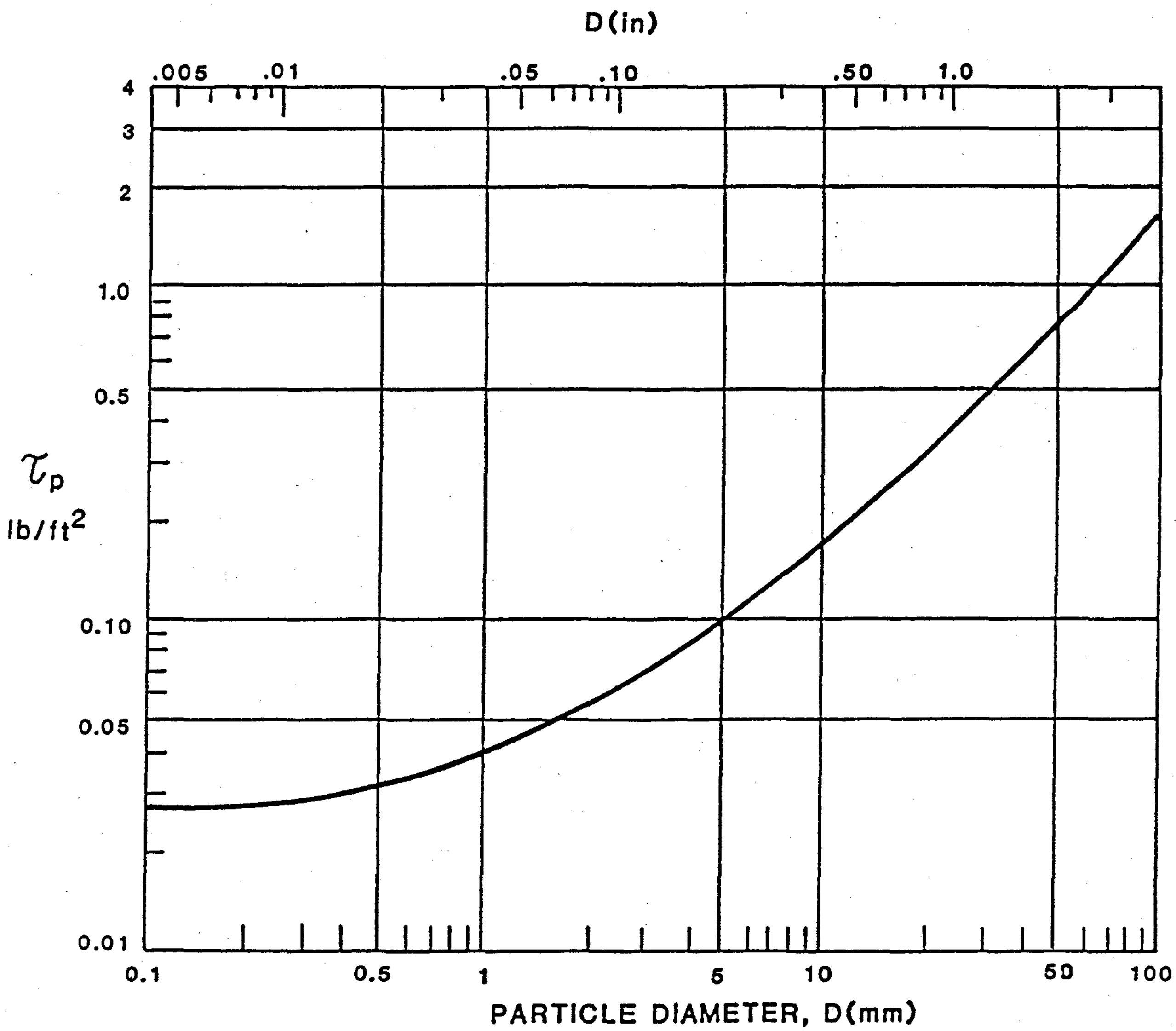


Chart 3: Permissible Shear Stress for Non-cohesive Soils (From HEC-15)

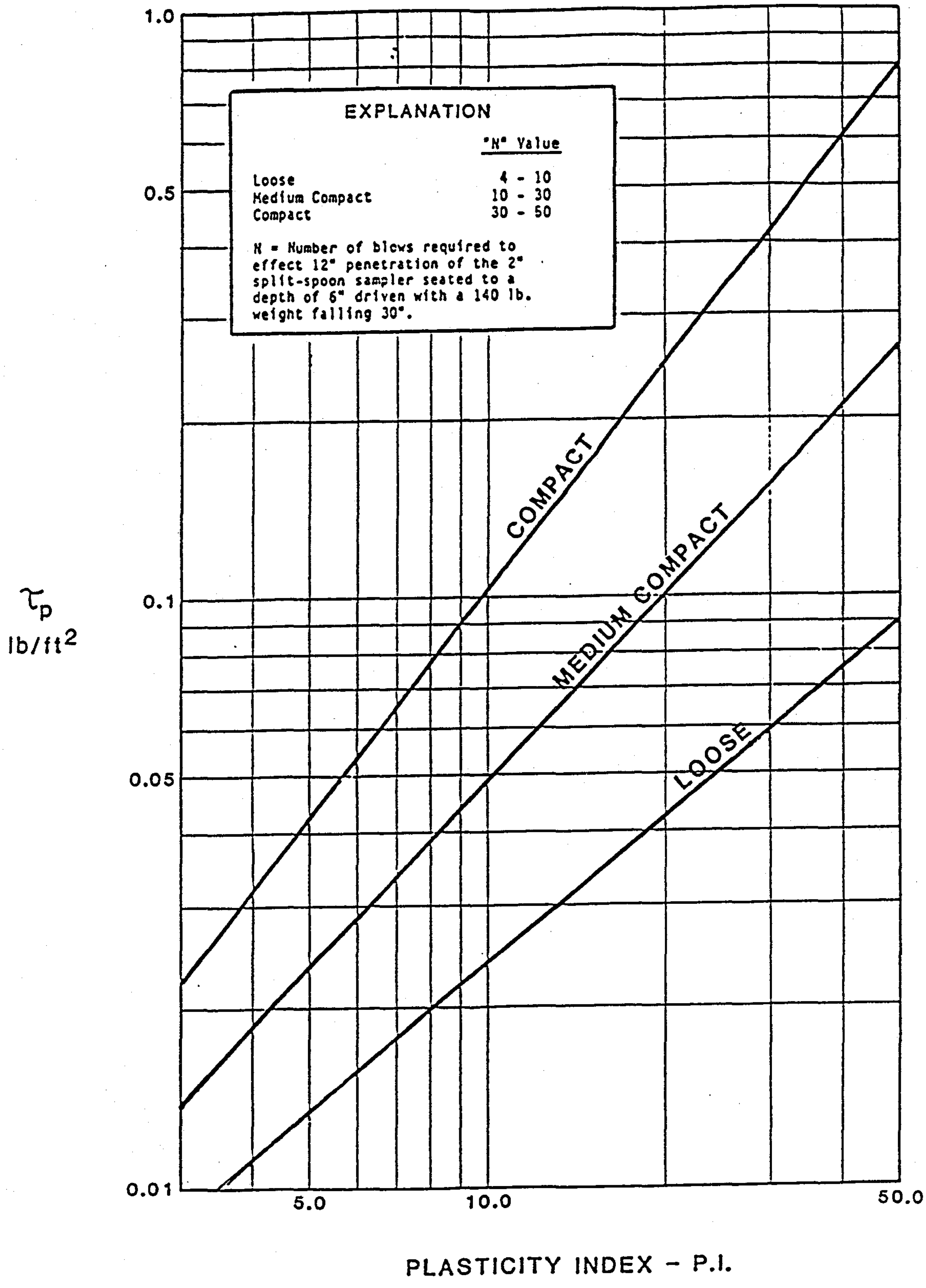


Chart 4: Permissible Shear Stress for Cohesive Soils (From HEC-15)

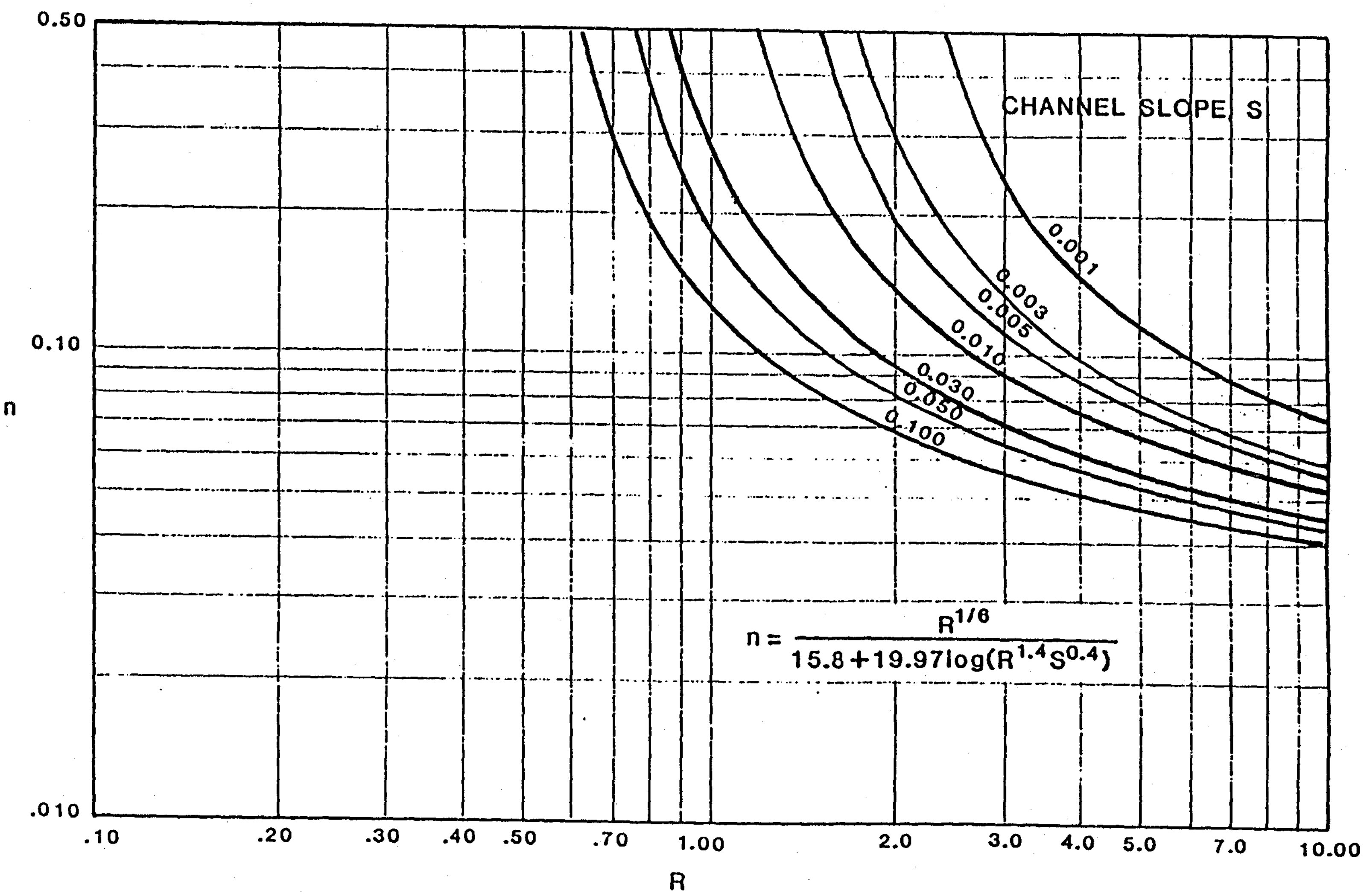


Chart 5: Class A Vegetation: Manning's n versus Hydraulic Radius, R (From HEC-15)

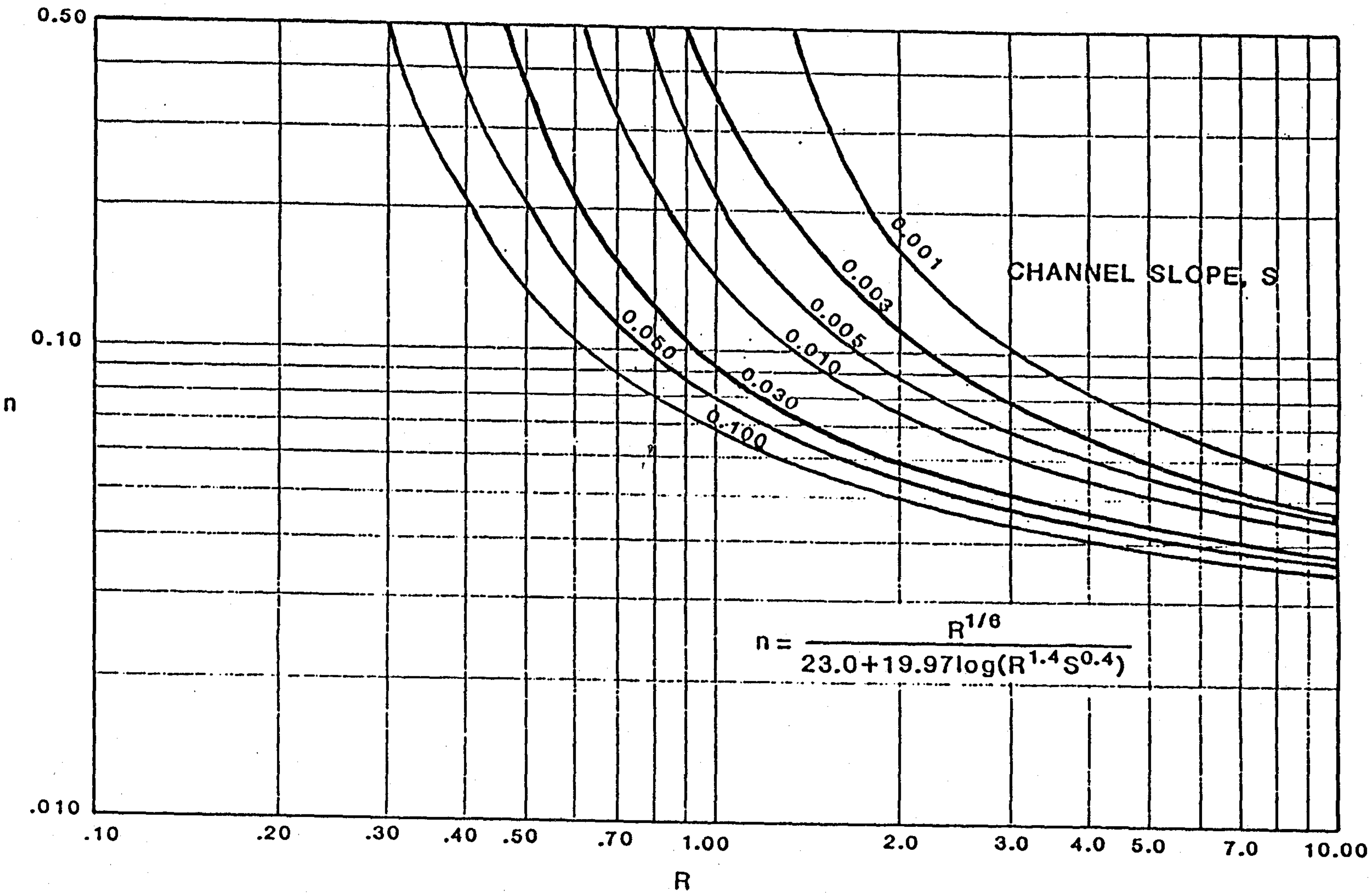


Chart 6: Class B Vegetation: Manning's n versus Hydraulic Radius, R (From HEC-15)

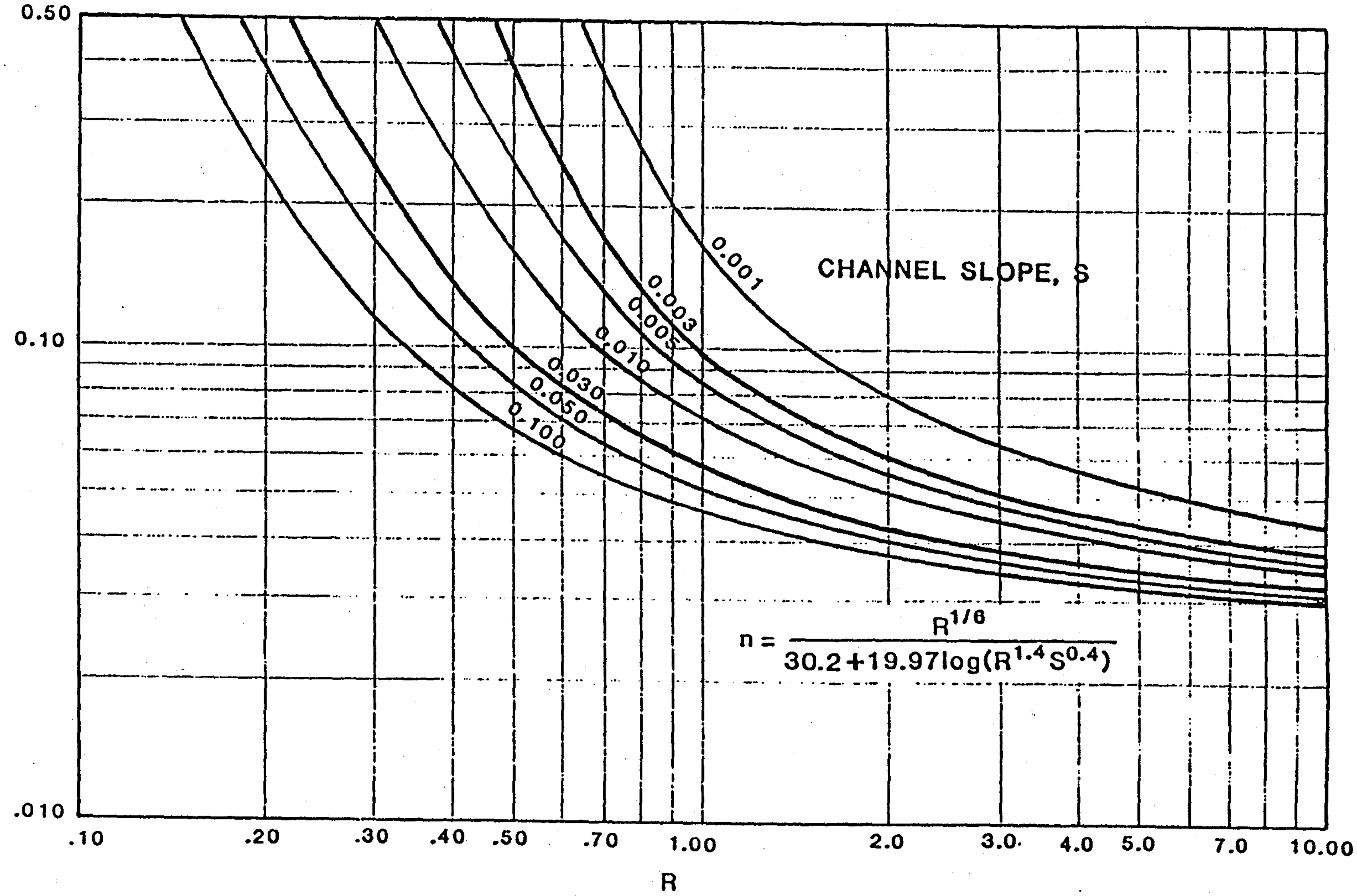


Chart 7: Class C Vegetation: Manning's n versus Hydraulic Radius, R (From HEC-15)

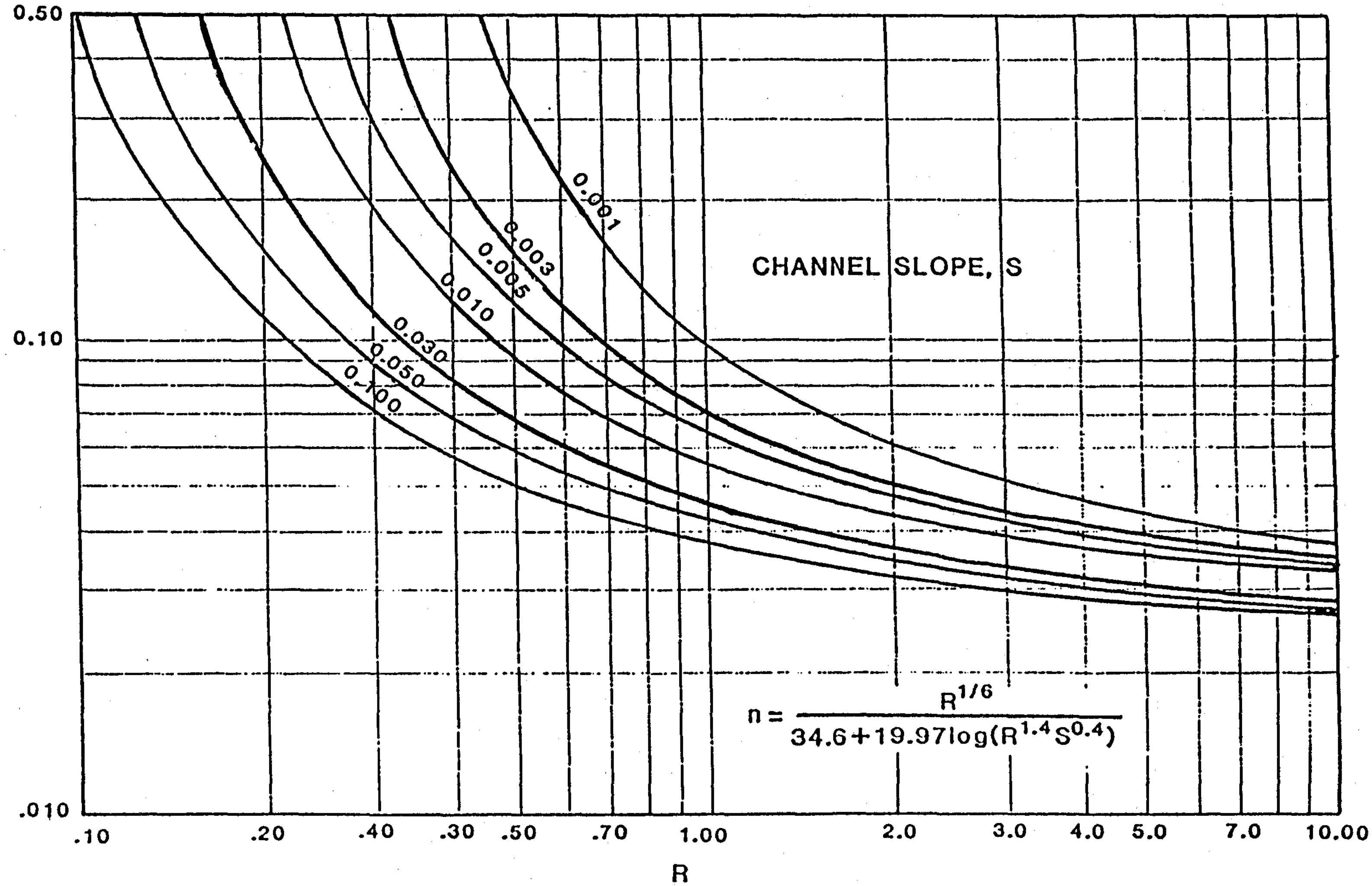


Chart 8: Class D Vegetation: Manning's n versus Hydraulic Radius, R (From HEC-15)

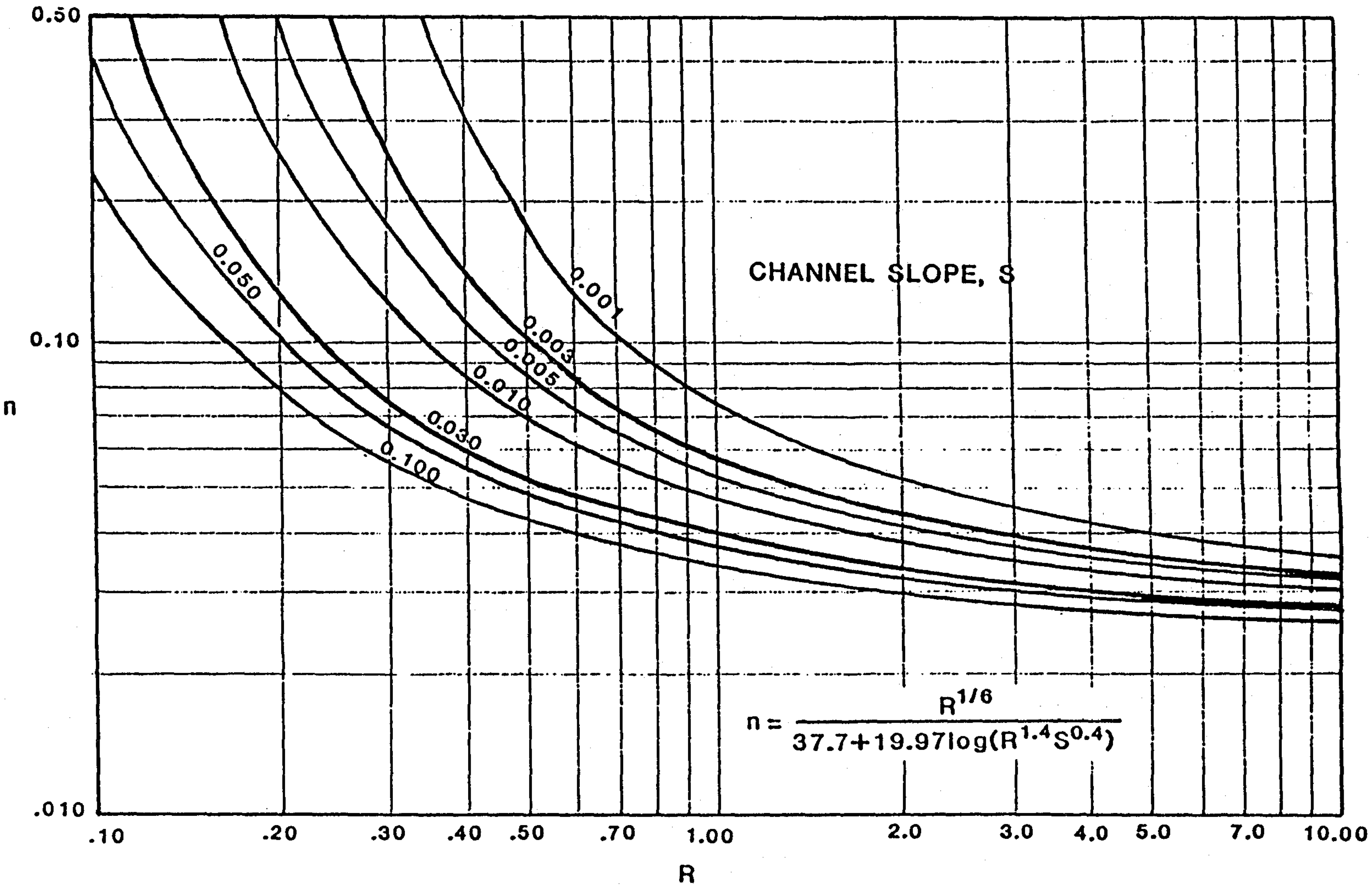


Chart 9: Class E Vegetation: Manning's n versus Hydraulic Radius, R (From HEC-15)

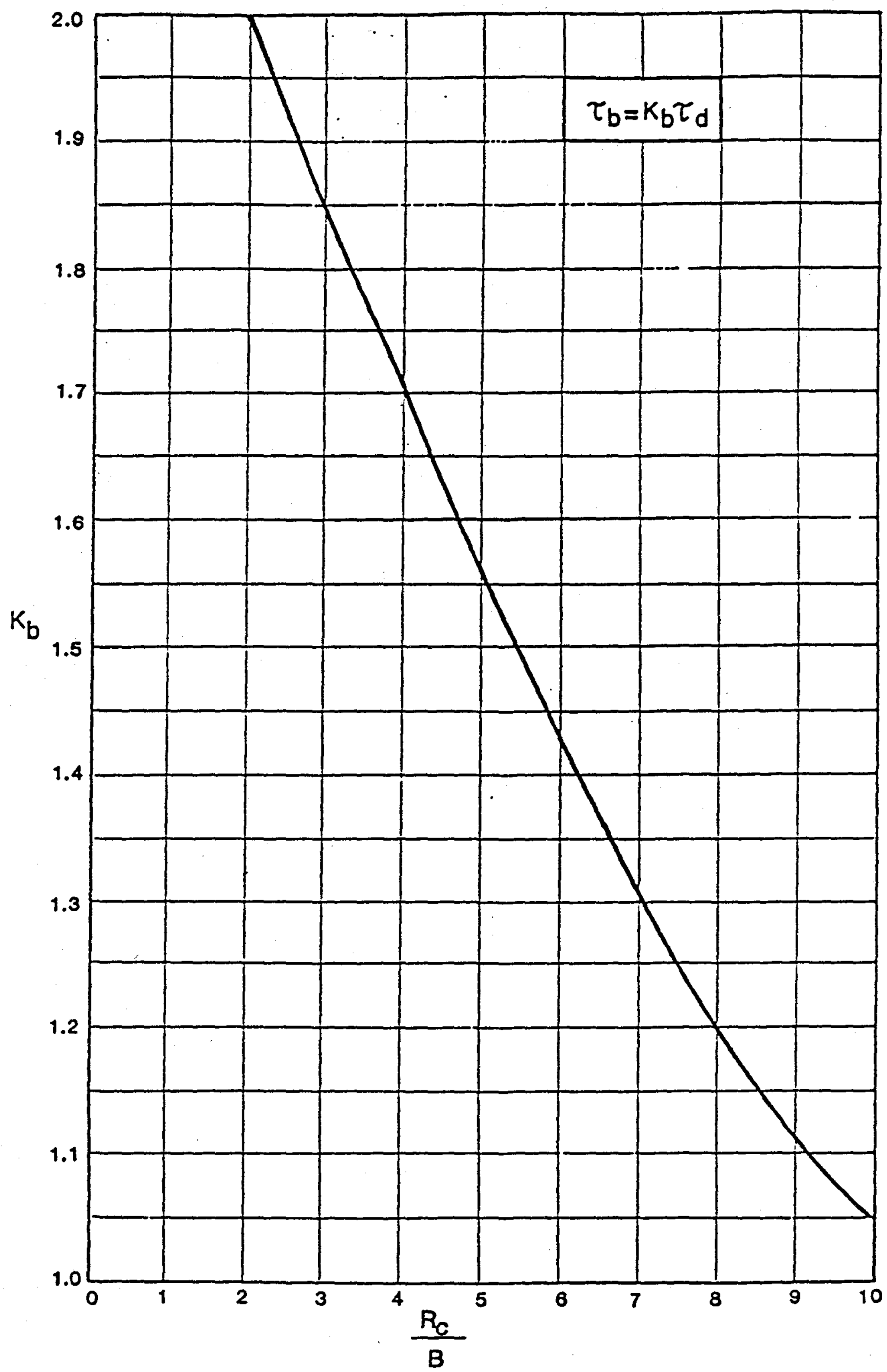


Chart 10: K_b Factor for Maximum Shear Stress on Channel Bends (From HEC-15)

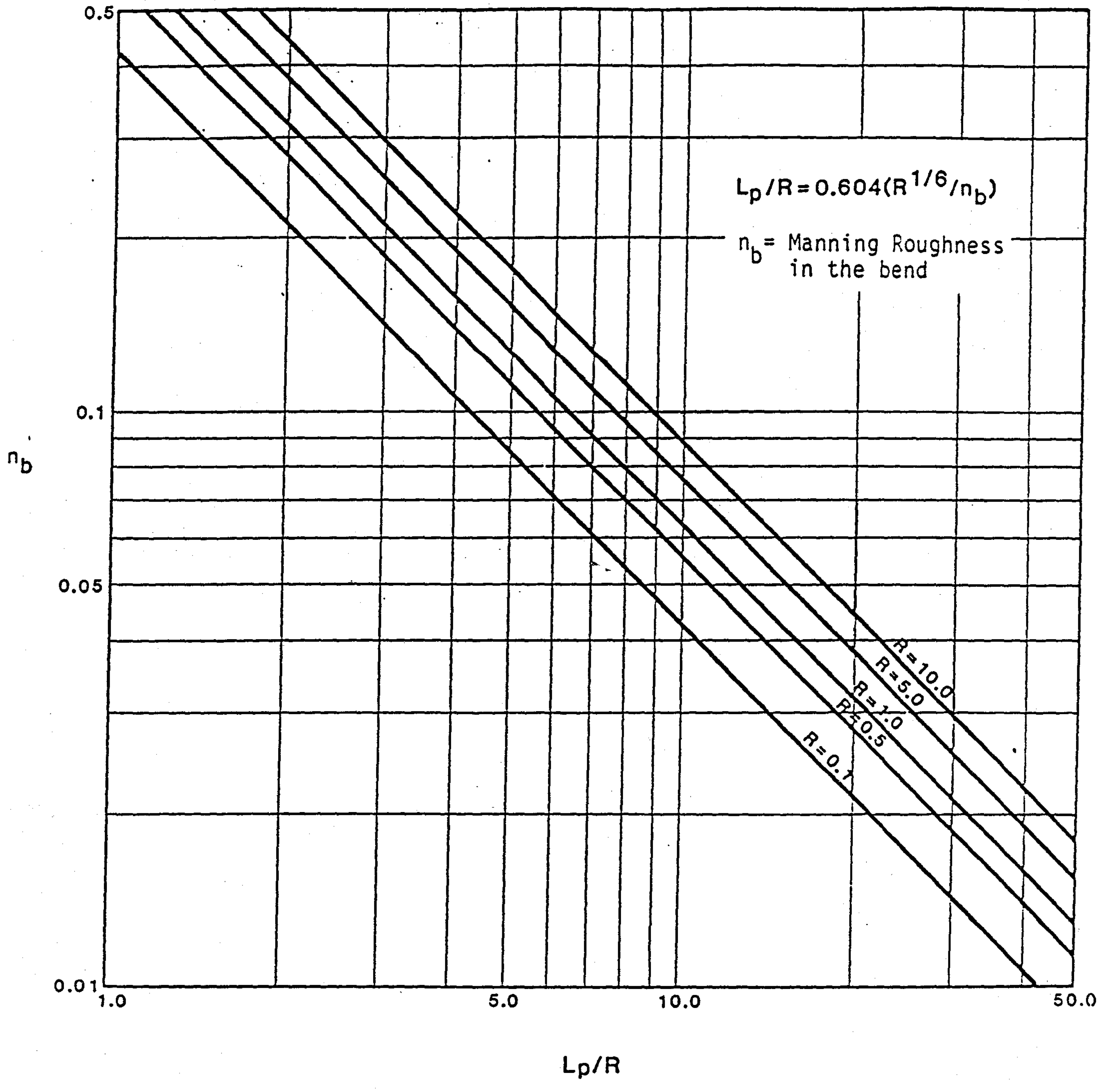


Chart 11: Protection Length, L_p, Downstream of Channel Bend (From HEC-15)

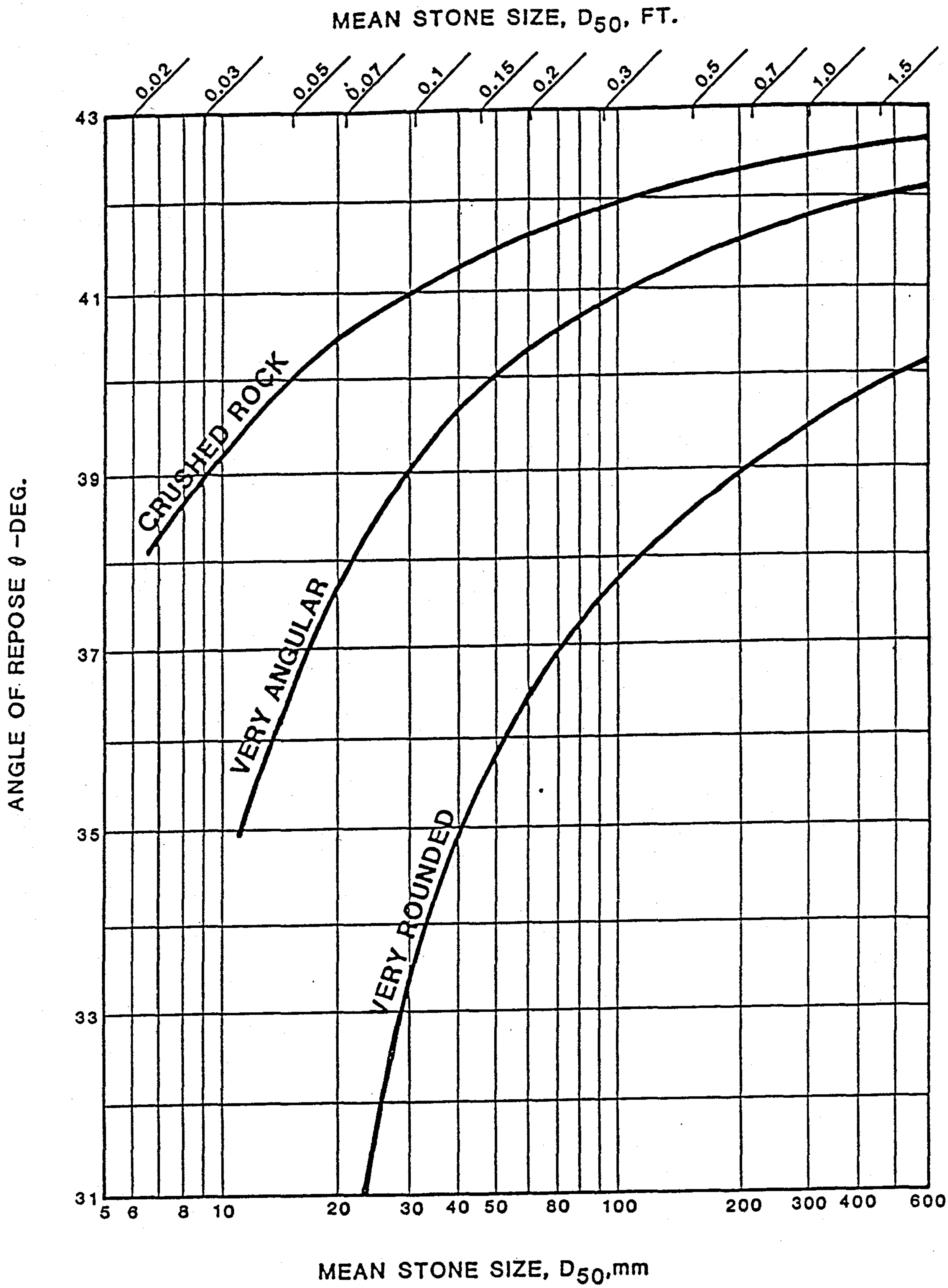


Chart 12: Angle of Repose of Riprap in Terms of Mean Size and Shape (From HEC-15)

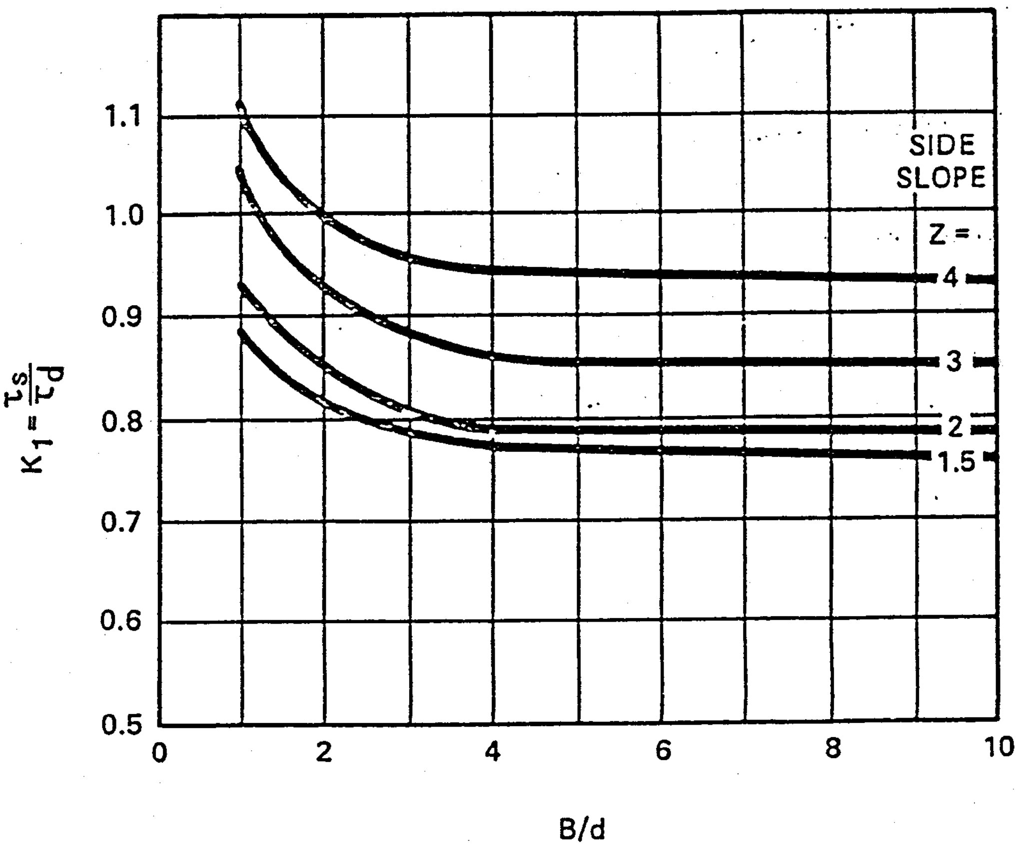


Chart 13: K_1 Factor, Channel Side Shear Stress to Bottom Shear Stress Ratio (From HEC-15)

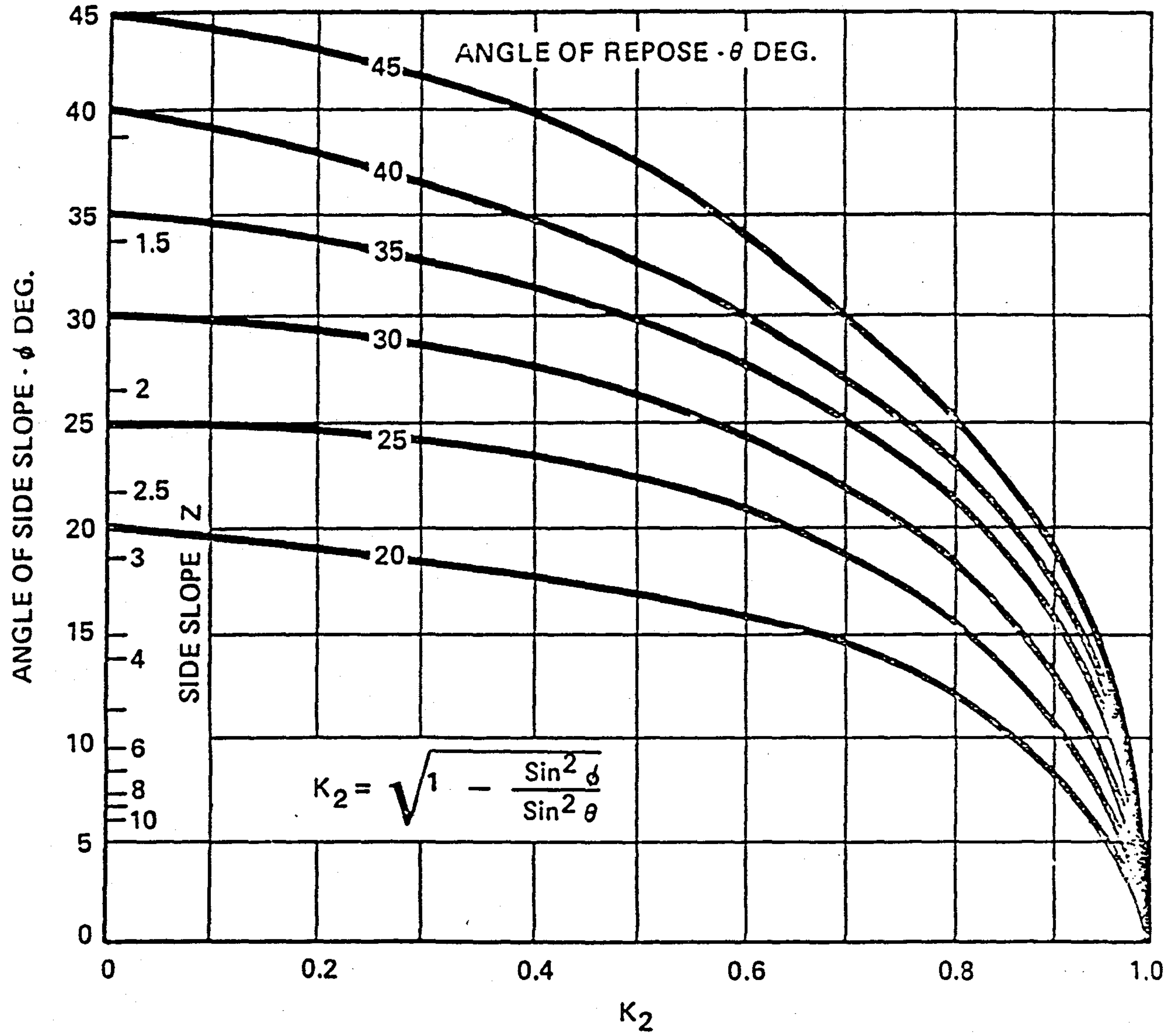


Chart 14: K_2 Factor, Tractive Force Ratio (From HEC-15)

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(703)978-8620

BASICS OF

FLUVIAL SYSTEMS

Lecture 3

FUNDAMENTALS OF FLUVIAL GEOMORPHOLOGY
An Engineering Perspective

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Colorado State University
Ref. 9

Webster's New World Dictionary defines ***fluvial*** as: *of, found in, or produced by a river or rivers*. The same reference defines ***morphology*** as: ***any scientific study of form and structure, as in physical geography, etc.*** With a little guess work, we can correctly extrapolate that fluvial geomorphology is the study of the form and structure of the surface of the earth (geo) as affected by flowing water. An equally important term is the ***fluvial system***. A system is an arrangement of things to form a whole. The primary goal on which we want to focus in this section is that whether you are considering a major lock and dam project, building a bridge across a river, or planning a bank stabilization project, you are working with a system and the complete system must be considered.

BASIC CONCEPTS

Five basic geomorphic concepts that should be considered in working with watersheds and rivers are: 1) the river is only a portion of a system, 2) the system is dynamic, 3) the system behaves with complexity, 4) geomorphic thresholds exist, and when exceeded, can result in abrupt changes, and 5) geomorphic analyses provide a historical prospective and the engineer must be aware of the time scale.

The Fluvial System

Schumm (1977) provides an idealized sketch of a fluvial system (Fig. 1). The parts are referred to as:

Zone 1 - the upper portion of the system which is the watershed or drainage basin; this portion of the system functions as the sediment supply.

Zone 2 - the middle portion of the system which is the river; this portion of the system functions as the sediment transfer zone.

Zone 3 - the lower portion of the system may be a delta, wetland, lake, or reservoir; this portion of the system functions as the area of deposition.

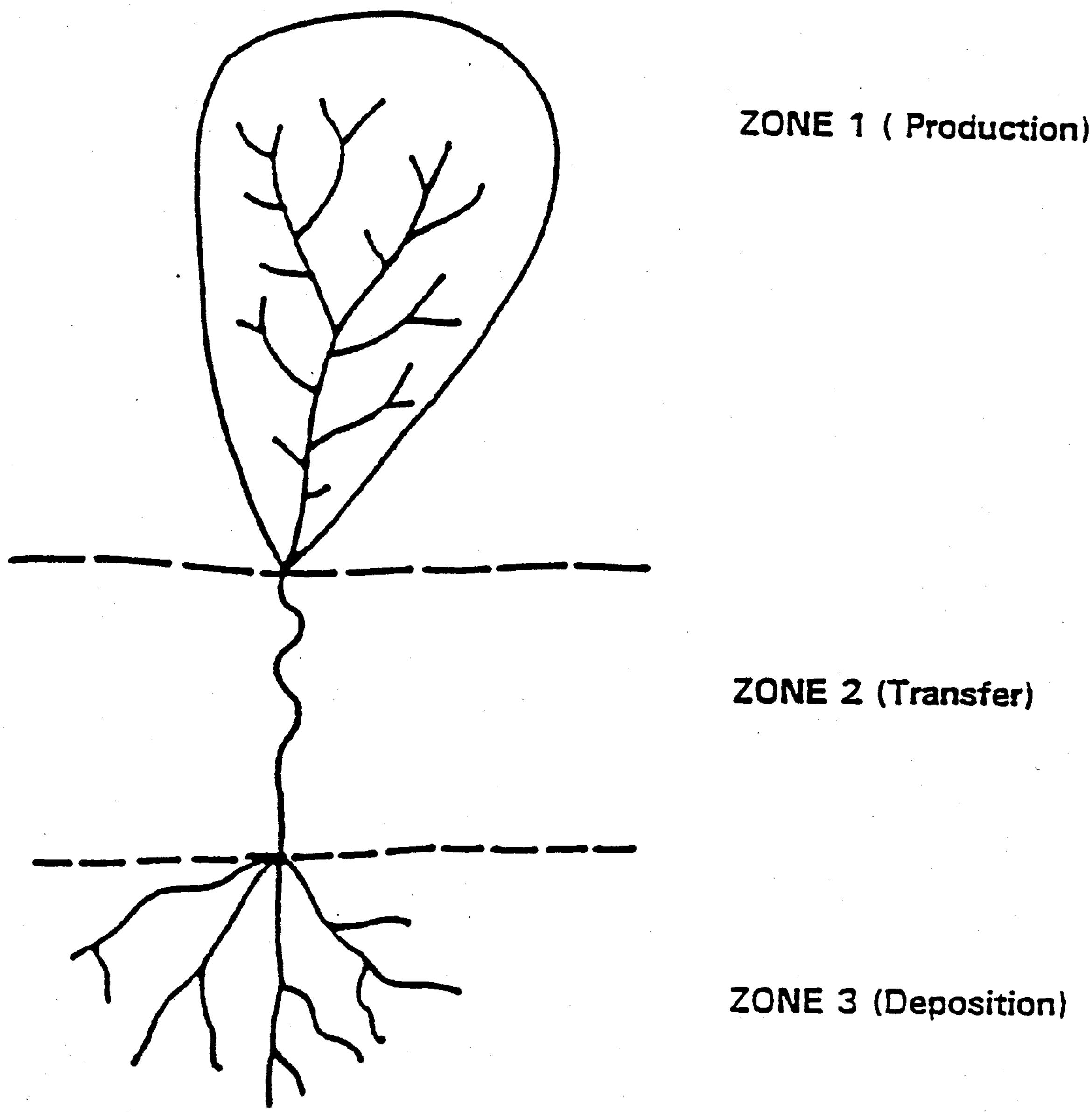

Figure 1. The fluvial system.

These three zones are idealized, because in actual conditions sediments can be stored, eroded, and transported in all zones. However, within each zone one of the processes is usually dominant. For our purposes in planning channel stabilization, we are primarily concerned with Zone 2, the transfer zone. We may need to riprap only a small length of stream bank (Zone 2) to solve a local instability problem; however, from a system viewpoint we must insure that our plan does not interfere with the transfer of sediment from upstream (Zone 1) to downstream (Zone 3). In channel stabilization planning we must not neglect the potential effects that may occur throughout the system.

The fundamental concept that a stream is a portion of a large and complex system may have been most eloquently stated by Dr. Hans Albert Einstein:

> *If we change a river we usually do some good somewhere and "good" in quotation marks. That means we achieve some kind of a result that we are aiming at but sometimes forget that the same change which we are introducing may have widespread influences somewhere else. I think if, out of today's emphasis of the environment, anything results for us it is that it emphasizes the fact that we must look at a river or a drainage basin or whatever we are talking about as a big unit with many facets. We should not concentrate only on a little piece of that river unless we have some good reason to decide that we can do that. But, I think the most important part that has to be done now is to actually go out in the field.*

The System Is Dynamic

In each of the idealized zones described above, a primary function is listed. Zone 1 is the sediment source which implies that erosion of sediment occurs. Zone 2 is the transfer zone which implies that as rainfall increases soil erosion from the watershed, some change must result in the stream to enable transfer of the increased sediment supply. Zone 3 is the zone of deposition and change must occur as sediment builds in this zone, perhaps the emergence of wetland habitat in a lake then a change to a flood plain as a drier habitat evolves. The function of each zone implies that change is occurring in the system, and that the system is dynamic.

From an engineering viewpoint some of these changes may be very significant, for example, loss of 100 feet of stream bank may endanger a home or take valuable agricultural land. From a geomorphic viewpoint, these changes are expected in a dynamic system and change does not represent a departure from a natural equilibrium system.

In 1948, Mackin gave the following definition of a graded stream:

> *A graded stream is one in which, over a period of years, slope is delicately adjusted to provide, with available discharge and with prevailing channel characteristics, just the velocity required for the transportation of the load supplied from the drainage basin. The graded stream is a system in equilibrium.*

Mackin did not say that a stream is in equilibrium is unchanging and static. Mackin's definition of equilibrium or graded conditions is in terms of the function of the river - to transfer the sediment supplied from the watershed. Change may be occurring in the stream bank, erosion may result and bank stabilization may be necessary, even on the banks of a stream in equilibrium. In planning stabilization measures, we must realize that we are forced to work in a dynamic system and we cannot disrupt the system while we are "fixin' the bank".

Complexity

Landscape changes are usually complex (Schumm and Parker, 1973). We are working in a system and we have defined a system as an arrangement of things to form a whole. Change to one portion of the system may result in complex changes throughout the system.

When the fluvial system is subjected to an external influence such as channelization of a portion of a stream, we can expect change to occur throughout the system. Channelization usually increases stream velocity and this would allow our stream to transfer more sediment, resulting in erosion upstream and deposition downstream of the portion of the stream that was channelized. Some of the Yazoo Basin streams that were channelized in the 1960s responded initially, but an equilibrium has not yet been reestablished in 1991 as repeated waves of degradation, erosion, and aggradation have occurred.

Thresholds

Geomorphic thresholds may be thought of as the straw that broke the camel's back. In the fluvial system this means that progressive change in one variable may eventually result in a abrupt change in the system. A the river erodes a few grains of soil from the toe of the river bank, no particular response will be noticed. If that continues with no deposition to balance the loss, the bank may eventually fail abruptly and dramatically due to undermining. As will be discussed later in this course, the amount of flow impinging along a riprap bank stabilization may vary considerably with no apparent effect on the stabilization; however, at some critical point the stone will begin to move and disastrous consequences can result.

In these examples the change was a gradual erosion of a few grains of soil and a variability of stream velocity, both which could be considered to be with the natural system. This type of threshold would be referred to as an intrinsic threshold. Perhaps the threshold was exceeded due to an earthquake or caused by an ill-planned bank stabilization project, these would be referred to as an extrinsic threshold. The planner must be aware of geomorphic thresholds, and the effect that their project may have in causing the system to exceed the threshold.

The previous definition of a graded stream by Mackin suggests that channel systems have a measure of elasticity that enables change to be absorbed by a shift in equilibrium. The amount of change a system can absorb before that natural equilibrium is disturbed depends on the sensitivity of the system, and if the system is near a threshold condition, a minor change may result in a dramatic response.

Time

We all have been exposed to the geologists view of time. The Paleozoic Era ended only 248 million years ago, the Mesozoic Era ended only 65 million years ago, and so on. Fortunately, we do not have to concern ourselves with that terminology. What we should be aware of is that the geologist temporal perspective is much broader that the temporal perspective of the engineer. Neither profession is good or bad because of the temporal perspective; just remember the background of person or the literature with which you are working.

Geomorphologists usually refer to three time scales in working with rivers: 1.) geologic time, 2.) modern time, and 3.) present time. Geologic time us usually expressed in thousands or millions of years and in this time scale only major geologic activity would be significant. Formation of mountain ranges, changes in sea level, and climate change would be significant in this time scale. The modern time scale describes a period of tens of years to several hundred years, and has been referred to as the graded time scale (Schumm and Lichty, 1965). During this period a river may adjust to a graded condition, adjusting to watershed water and sediment discharge. The present time is considered to be a shorter period, perhaps one year to ten years. There is no hard and fast rules that govern these definitions, and are only offered for consideration. Design of a major project may require less than ten years, and numerous minor projects are designed and built within the limitations of present time. Project life often extends into graded time. From a geologists temporal point of view, engineers built major projects in an instant of time, and expect the projects to last for a significant period.

In river related projects, including bank stabilization, time is the enemy. As engineers and planners we must learn all we can from that enemy by adopting a historical perspective for each project that we undertake.

RELATIONSHIP IN RIVERS

The previous discussion presents several definitions from the dictionary and five basic concepts. A final definition of geomorphology suggested in the previous paragraph is that geomorphology presents a framework for developing a historical perspective. Another definition, although given in jest, may be the one most remembered after this next section. *Geomorphology is the triumph of terminology over common sense.*

Classification

Several primary methods of river classification are presented in the following paragraphs, and these methods can be related to fundamental variables and processes controlling rivers. One important classification is either alluvial or non-alluvial. An ***alluvial*** channel is free to adjust dimensions such as size, shape, pattern, and slope in response to change and flow through the channel. The bed and banks of an alluvial river are composed of material transported by the river under present flow conditions. Obviously, a ***non-alluvial river*** is not free to adjust. An example of a non-alluvial river is a bedrock controlled channel. In other conditions, such as in high mountain stream flowing in very coarse glacially deposited materials or significantly controlled by fallen timber would suggest a non-alluvial system.

Another classification methodology by Schumm (1977) includes consideration of the type of sediment load being transported by the stream, the percentage of silt and clay in the channel bed and banks, and the stability of the channel. ***Sediment load*** refers to the type or size of material being transported by a stream. The total load can be divided into the ***bed sediment load*** (bedload) and the ***wash load*** (suspended load). The bedload is composed of particles of a size found in appreciable quantities in the bed of the stream, and the wash load are those finer particles that are found in small quantities in the shifting portions of the bed (ASCE, 1977).

For purposes of this classification system, a stable channel complies with Mackin's definition of a graded stream. An unstable stream may be either ***degrading*** (eroding) or ***aggrading*** (depositing). In the context of the definition of a graded stream being in balance between sediment supplied and sediment transported, an aggrading stream has excess sediment discharge and a degrading stream has a deficite of sediment discharge. ***Sediment discharge*** is the rate at which the sediment load is being supplied or transported through a reach.

Table 1 presents a summary of this classification system and provides a description of the response of the river segment to instability and a description of the stable segment. It is very important to note that the work on which this classification was based was conducted in the Mid-western U.S.; therefore, the classification system represents an interpretation of empirical data and extrapolation of the classification beyond the data base should be done cautiously.

Review of Table 1 reveals the term ***sinuosity***, and this calls for a discussion of ***channel planform***. Channel planform is another major type of channel classification. Channel planform is generally described as either braided, meandering, or straight. The ***braided pattern*** is characterized by a division of the river bed into multiple channels, most braided streams are relatively high gradient and relatively coarse streams. Most streams are not straight, yet the characteristics of a straight stream are very similar to the more common ***meandering stream*** (Ritter, 1978).

Table 1. Classification of Alluvial Channels (after Schumm, 1977)

Mode of sediment transport and type of channel	Channel sediment (M) (percent)	Bedload (percentage of total load)	Channel stability		
			Stable (graded stream)	Aggrading (excess sediment discharge)	Degrading (deficiency of sediment discharge)
Suspended load	>20	<3	Stable suspended-load channel. Width/depth ratio <10; sinuosity usually >2.0; gradient, relatively gentle	Depositing suspended load channel. Major deposition on banks cause narrowing of channel; initial streambed deposition minor	Eroding suspended-load channel. Streambed erosion predominant; initial channel widening minor
Mixed load	5-20	3-11	Stable mixed-load channel. Width/depth ratio >10, <40; sinuosity usually <2.0, >1.3; gradient moderate	Depositing mixed-load channel. Initial major deposition on banks followed by streambed deposition	Eroding mixed-load channel. Initial streambed erosion followed by channel widening
Bed load	<5	>11	Stable bed-load channel. Width/depth ratio >40; sinuosity usually <1.3; gradient, relatively steep	Depositing bed-load channel. Streambed deposition and island formation	Eroding bed-load channel. Little streambed erosion; channel widening predominant

As shown in Figure 2, straight reaches often contain accumulations of bedload called alternate bars that are positioned successively down the river on opposite sides. A line that connects the deepest parts of the river is the ***thalweg***. Between the alternate bars, the thalweg is at a location which is relatively shallow and this is referred to as the ***riffle or crossing***. Adjacent to the alternate bars is the deepest portion of the thalweg, and this location is called the ***pool***. Meandering rivers have similar features, frequently with more dramatic appearance. In the meandering river, the thalweg swings widely to the outside bank of the bend or bendway, and the pool may become deep enough to cause mass failure of the bank. The sinuosity of a meandering stream is defined as the ratio of channel length (L_c) to the straight-line or valley length (L_v). Think of sinuosity as the ratio of the distance the fish swims to the distance the crow flies. Sinuosity may also be defined as the ratio of the valley slope (S_v) to the channel slope (S_c).

$$P = L_c/L_v = S_v/S_c \qquad (1)$$

Schumm and Meyer (1979) presented the channel classification shown in Figure 3 which is based on channel planform, sediment load, energy, and relative stability. As with any classification system, Figure 3 implies that river segments can be conveniently subdivided into clearly discernable groups. In reality, a continuum of channel types exist and the application of the classification system requires judgement.

Investigation by Lane (1957) and Leopold and Wolman (1957) showed that the relationships between discharge and channel slope can define thresholds for indicating which rivers tend to be braided or meandering, as shown in Figures 4 and 5. Lane's relationship is somewhat more realistic because an intermediate range is included; however, both relationships are very similar in the variables used and the appearance of the graphs. Rivers that are near the threshold lines may exhibit segments that transitions between the two plan forms.

An example of the use of this type of relationship is provided by Schumm and Beathard (1976). The Chippewa River is a tributary to the Mississippi River entering downstream of St. Paul, Wisconsin. The Chippewa River is the second largest river in Wisconsin with a drainage area of approximately 9,500 square miles. From the confluence with the Mississippi River upstream for a distance of about 16.5 miles, the main channel is braided with the characteristically broad and shallow cross section and numerous shifting sand bars. Bankfull width in this lower reach is about 1000 feet. Upstream for approximately 42 miles, the channel is meandering with a sinuosity of approximately 1.49 and with a channel bankfull width of about 640 feet. The braided reach has a gradient of 0.00033 and the meandering reach has a gradient of .00028. The braided reach supplies more sediment than the meandering reach.

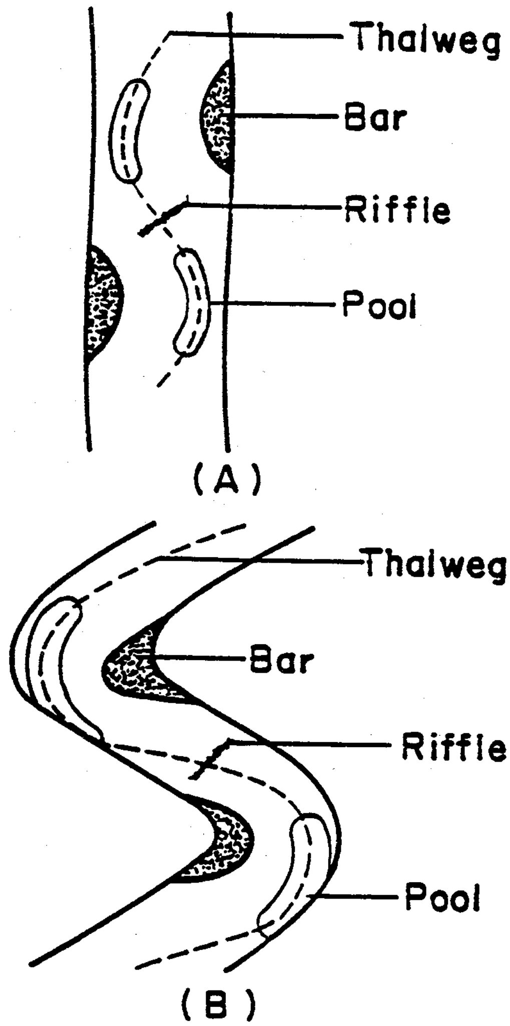

Figure 2 **Features associated with (A) straight and (B) meandering rivers.**

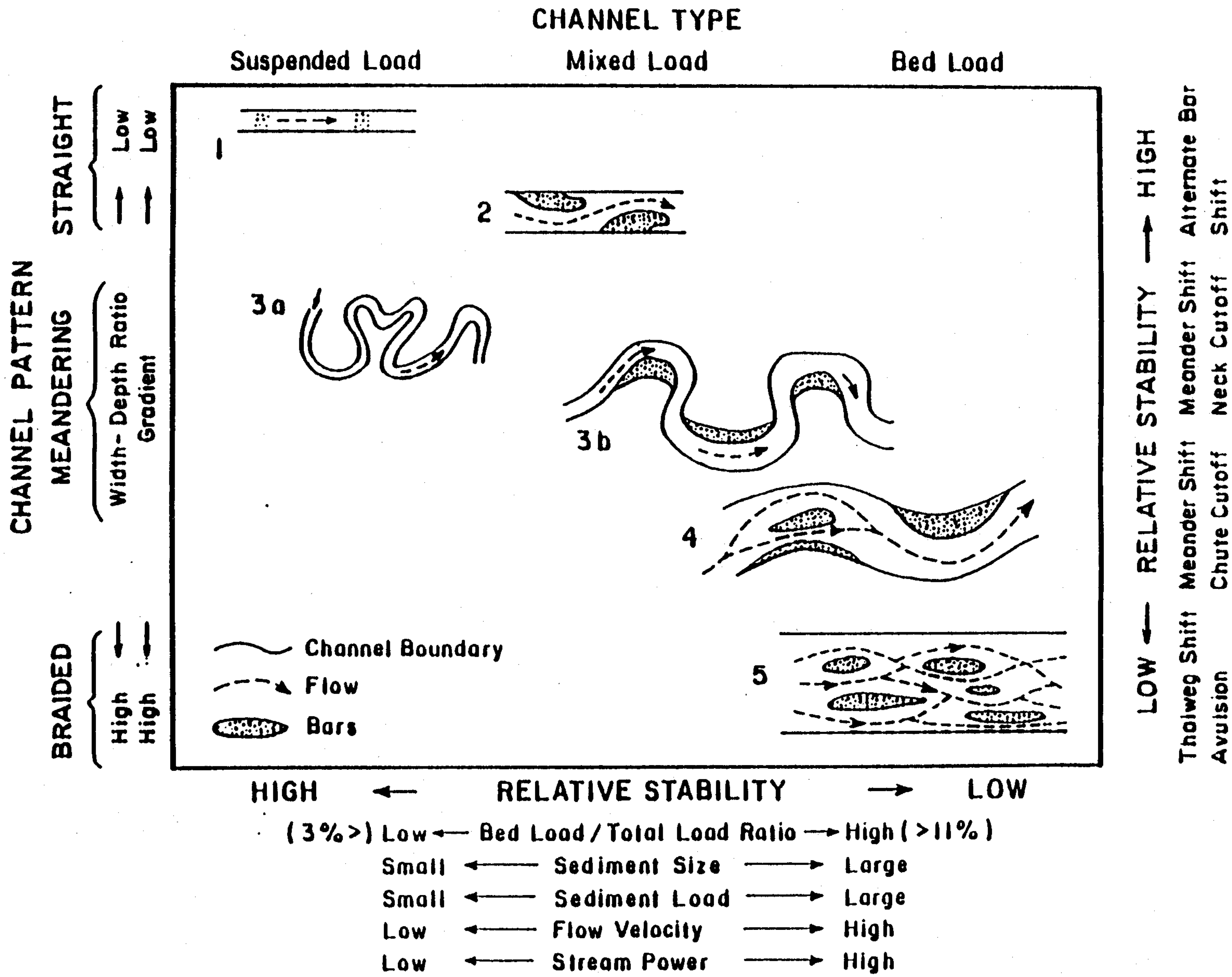

Figure 3 Channel classification based on pattern and type of sediment load.

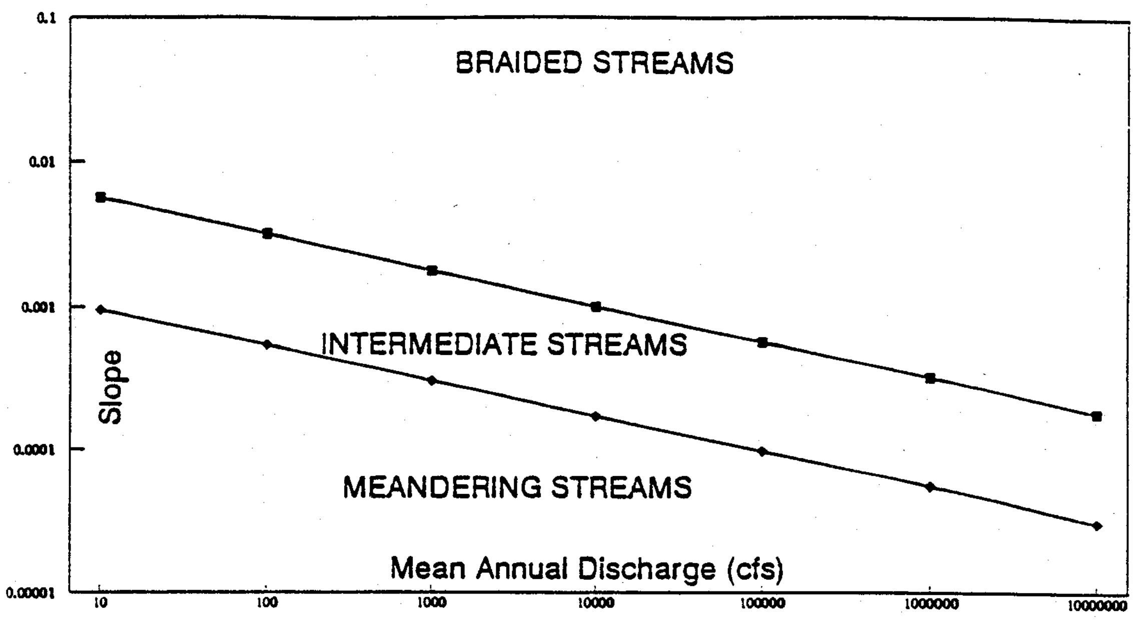

Figure 4 **Lane's (1957) relation between channel patterns, channel gradient and mean discharge.**

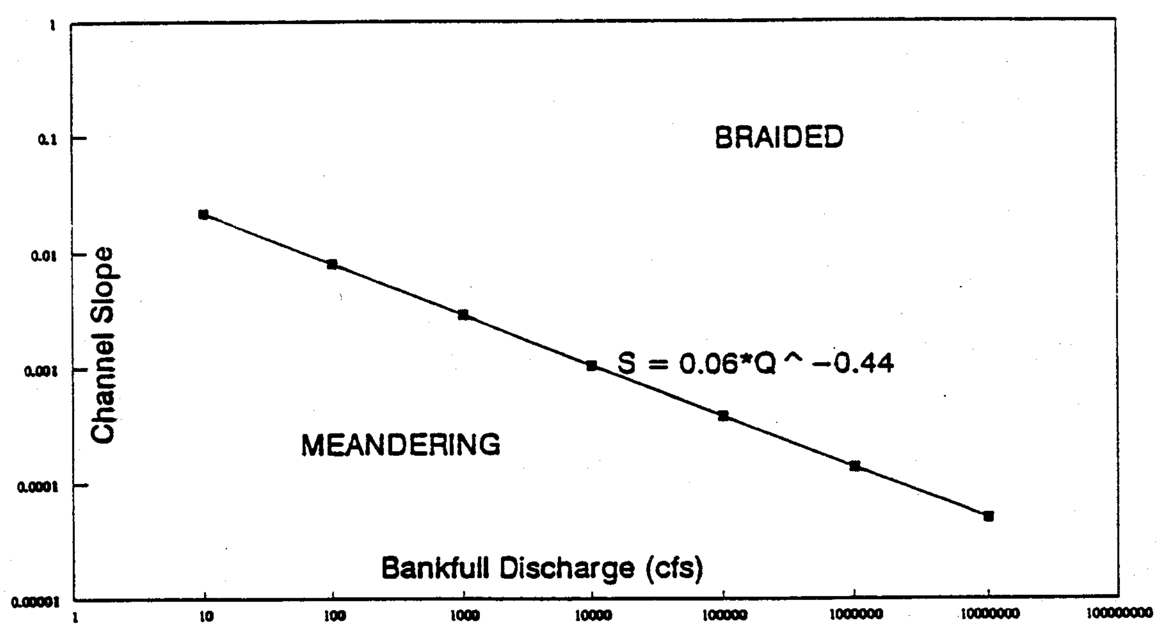

Figure 5 **Leopold and Wolman's (1957) relation between channel patterns, channel gradient, and bankfull discharge.**

Other than academic interest in this situation, the Chippewa River is a major contributor of sediment to that portion of the Mississippi River. If a reasonable and cost effective method of shifting the Chippewa River to a meandering condition could be found, then costly dredging on the Mississippi could be reduced. Figure 6 is a repeat of Lane's relationship showing the plotting position of the upstream meandering reach (M) and the downstream braided reach (B). Lane's relationship shows that both reaches plot in the intermediate zone and indicates that either grade control or sediment production could be effective in changing the planform and reducing the sediment discharge to the Mississippi River.

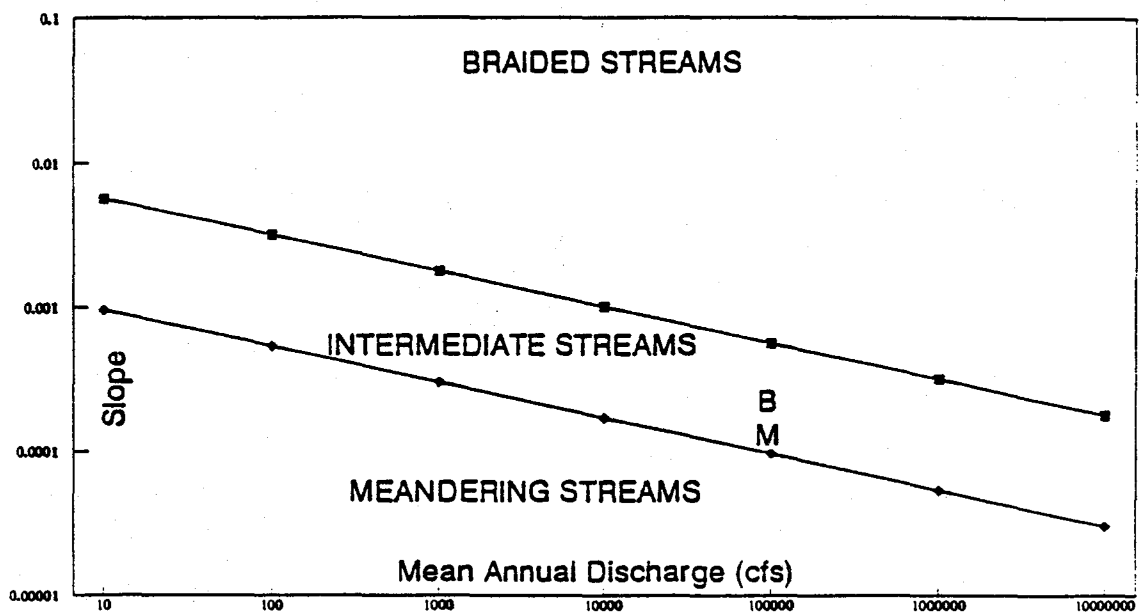

Figure 6. Lane's relationship showing the position of the meandering (M) and braided (B) reaches of the Chippewa River.

Another set of empirical relationships is related to meander geometry. The definitions on which these relationships are based are shown in Figure 7. Leopold, et al. (1964) reported the relationship between ***meander wave length*** (L) and ***channel width*** (w), ***meander amplitude*** (A) and ***channel width*** (w), and ***meander wave length*** (L) and ***bendway radius of curvature*** (R_c) as defined by Leopold and Wolman (1960). The relationships are:

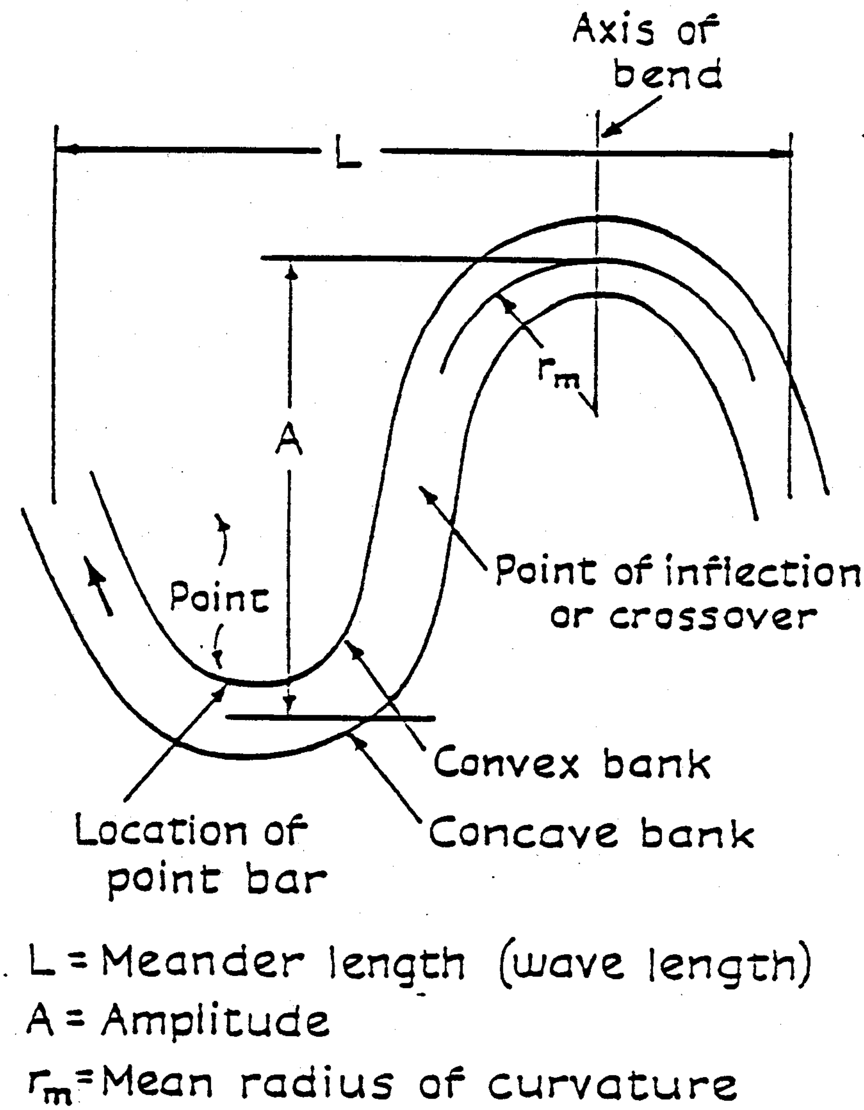

Figure 7. **Definition sketch for channel geometry.**

$$L = 10.9\, w^{1.01} \qquad (2)$$

$$A = 2.7\, w^{1.1} \qquad (3)$$

$$L = 4.7\, R_c^{0.98} \qquad (4)$$

Leopold, et al. (1964) stated that the exponent for the relationships are approximately unity, and these relationships can be considered linear. Also, they pointed out that channel meander form is affected by the cohesiveness of the channel boundaries. Dury (1964) found that meander wave length is related to the mean annual flood (Q_{ma}):

$$L = 30\, Q_{ma}^{0.5} \qquad (5)$$

Schumm (1960 and 1977) investigated the effect of the percentage silt and clay (M) in the stream boundaries and reported the following relationship for meander wave length:

$$L = 1890\, Q_m^{0.34}\, M^{-0.74} \qquad (6)$$

where Q_m is the average annual flow. The width to depth ratio (F) is also related to the percentage silt and clay:

$$F = 255\, M^{-1.08} \qquad (7)$$

Channel slope (S) was found to be related to the mean annual discharge (Q_m) and percentage silt and clay:

$$S = .60\, M^{-0.38}\, Q_m^{-0.32} \qquad (8)$$

Figure 8 shows the variability of the meander wavelength of Rio Bogota along the 80 km reach upstream of the Alicachin gates. In general, meander wavelength decreases in an upstream direction and the data indicates that most values are between 1000 ft. and 5000 ft. in length. Figure 9 is a comparison between meander wavelength and channel width. The actual Rio Bogota data is represented by the letter B. The relationship proposed by Leopold and Wolman (1960) is shown as a straight line, and comparison of these shows that the actual meander wavelength of Rio Bogota is generally much greater than the values that are predicted by Leopold and Wolman (1960). The relationship proposed for a prediction of wavelength as a function of channel width does not apply well to Rio Bogota. Leopold and Wolman (1960) also proposed a relationship to predict meander wavelength as a function of the bend radius of curvature. Figure 10 shows that the most of the actual Rio Bogota data (B) is represented by their relationship. These morphologic relationships for wavelength as a function channel width or radius of curvature demonstrate both the usefulness and limitations of empirical relationships. Unless the watersheds, climate, soil materials, geology, discharge, and other important variables are relatively similar between the Rio Bogota and the streams from which Leopold and Wolman (1960) collected the data to develop those relationships, it is unlikely that their predictions can directly apply.

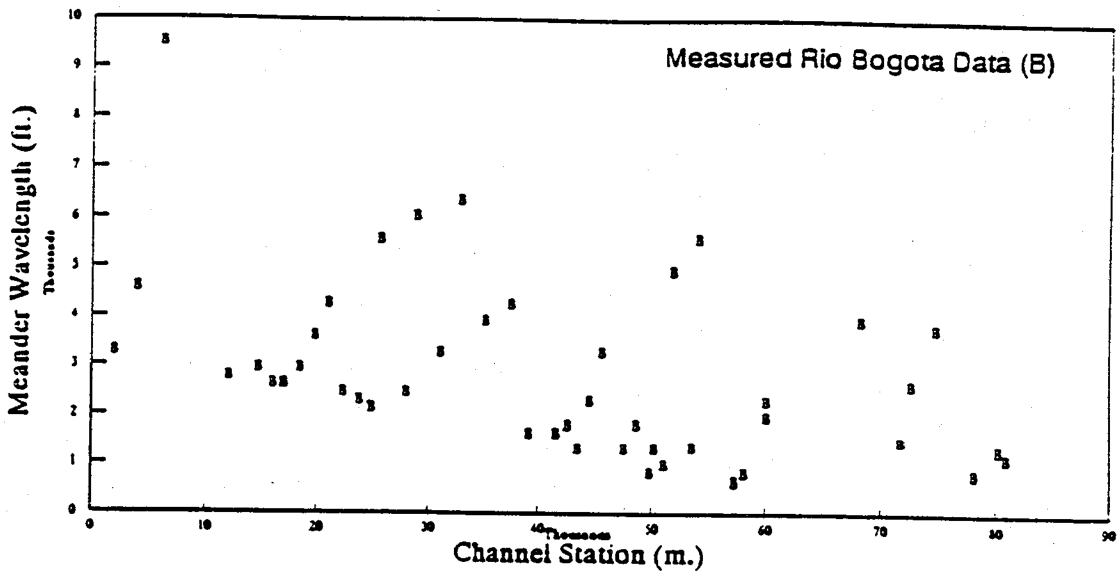

Figure 8 This figure shows the variability of meander wavelength along the 80 km study reach.

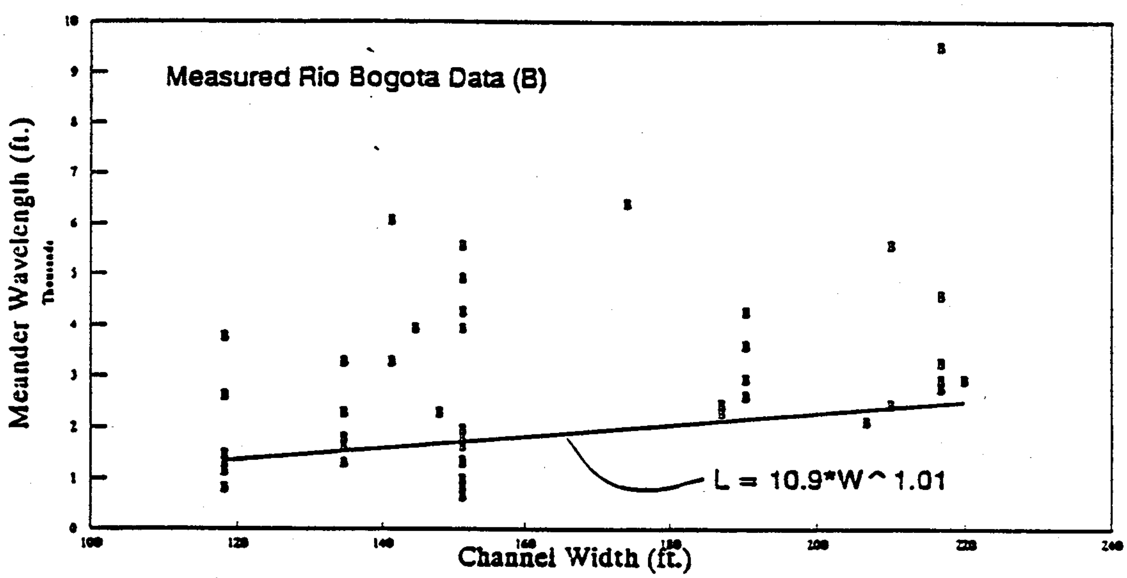

Figure 9 The figure illustrates the actual relationship between Rio Bogota channel width and the meander wave length. The predicted relationship from Leopold et al. (1980) of $L = 10.9\ W^{1.01}$ generally under predicts the wave length.

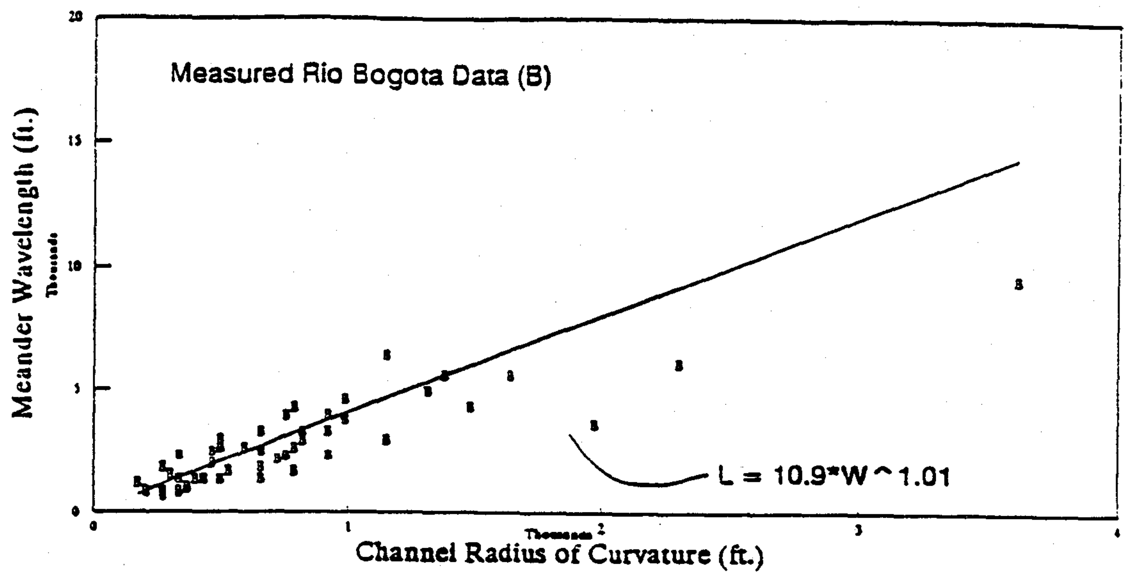

Figure 10 This graph depicts actual Rio Bogota wave length as a function of radius of curvature. Data is shown by the letter B. The straight line is a prediction relationship by Leopold and Wolman (1960).

Schumm (1977) developed a prediction relationship for meander wavelength which includes the average stream discharge and the weighted mean percentage of silt and clay in the channel bed and banks. He reasoned that the erosion resistance and bank slope stability would have a significant affect on channel morphology. Figure 11 was developed using the relationship between meander wavelength, average discharge, and the weighted mean percentage silt and clay. Average discharge for Rio Bogota was used along with hypothetical percentages of silt and clay. Although the percentage of silt and clay in the channel bed and banks is not known, the range of values suggests that this relationship could be applicable to Rio Bogota. Based on observed river systems, soil properties can be extremely important.

Similarly, Schumm (1977) also proposed relationships between channel sinuosity and the weighted mean percentage silt and clay, and for the ***width to depth ratio***. Figure 12 is the relationship which portrays sinuosity, and this relationship indicates that sinuosity increases with the silt and clay content. The %M is the percentage of silt and clay in the perimeter of the channel weighted as the percentage of perimeter length in which the material occurs. As the %M increases from 20% to 50%, sinuosity was observed to increase from 2.0 to 2.5 which encompasses the maximum Rio Bogota sinuosity.

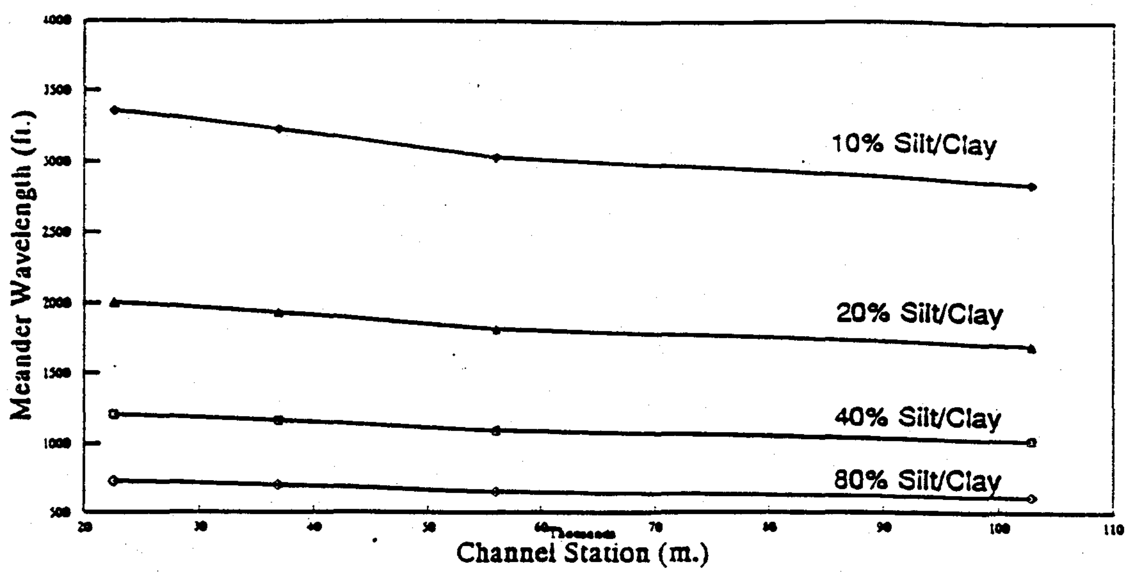

Figure 11 **The relationship found by Schumm (1977) between the percentage silt/clay in the channel bed and banks and meander wave length is shown for average flow on the Rio Bogota and for varying percentages of silt/clay.**

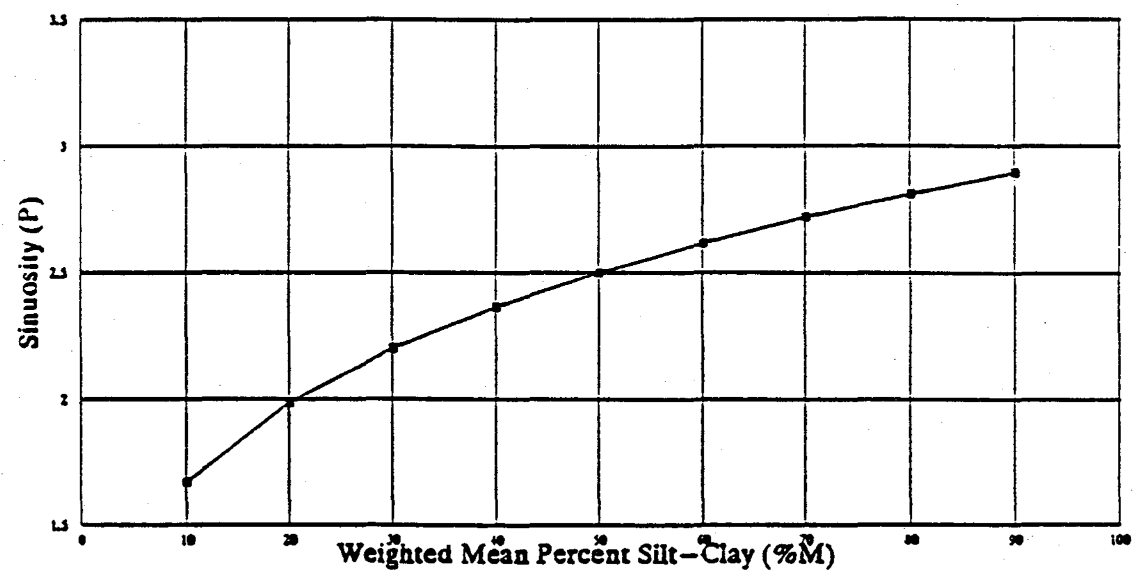

Figure 12 **The relationship found between sinuosity (P) and the weight mean percentage silt/clay in the channel bed and banks (%M) for several alluvial rivers in the Great Plains of the USA. (Schumm, 1977)**

Figure 13 shows that as the %M increases from 20% to 50%, the width to depth ratio was observed to decrease from 10 to approximately 4 which encompasses the range of Rio Bogota values. These two figures demonstrate that the range of existing sinuosity and width to depth ratio may not be that unusual because of the high silt clay content.

Figure 14 demonstrates the relationship observed between channel slope, average discharge, and the percentage silt and clay. Also shown are the reported present slope values for Rio Bogota and the proposed project slope values. This graph suggests that the lower slopes are observed at locations of high erosion resistance (80% silt and clay). Although this is implied, all that can be deduced from this graph is that silt and clay usually deposits in low energy, slow moving flow, and that the proposed and existing Rio Bogota slopes are very low.

Conclusions that can be reached from the analyses of morphologic variables are as follows:

a. Rio Bogota meander wavelength values are generally greater than those predicted by the Leopold and Wolman (1960) relationship.

b. The Leopold and Wolman (1960) relationship to predict meander wavelength as a function of radius of curvature is adequate.

c. Consideration of a relatively high weighted mean percentage silt and clay (%M) in the Rio Bogota valley provides a reasonable explanation for the observed values of wavelength, sinuosity, and width/depth.

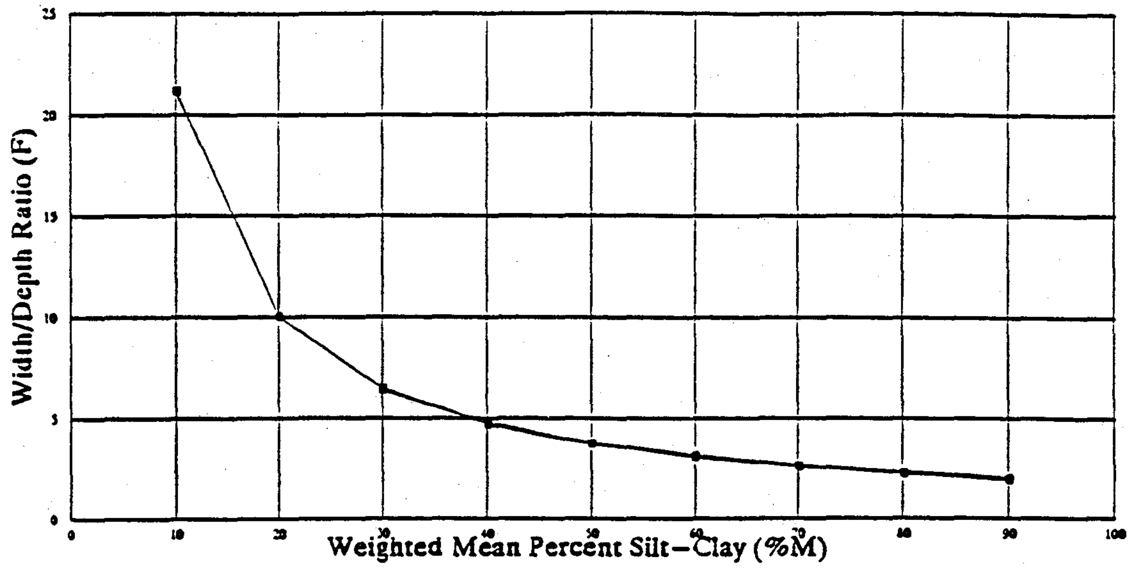

Figure 13 **This figure depicts the relationship between the width/depth ratio (F) and the weighted mean percentage of silt and clay.**

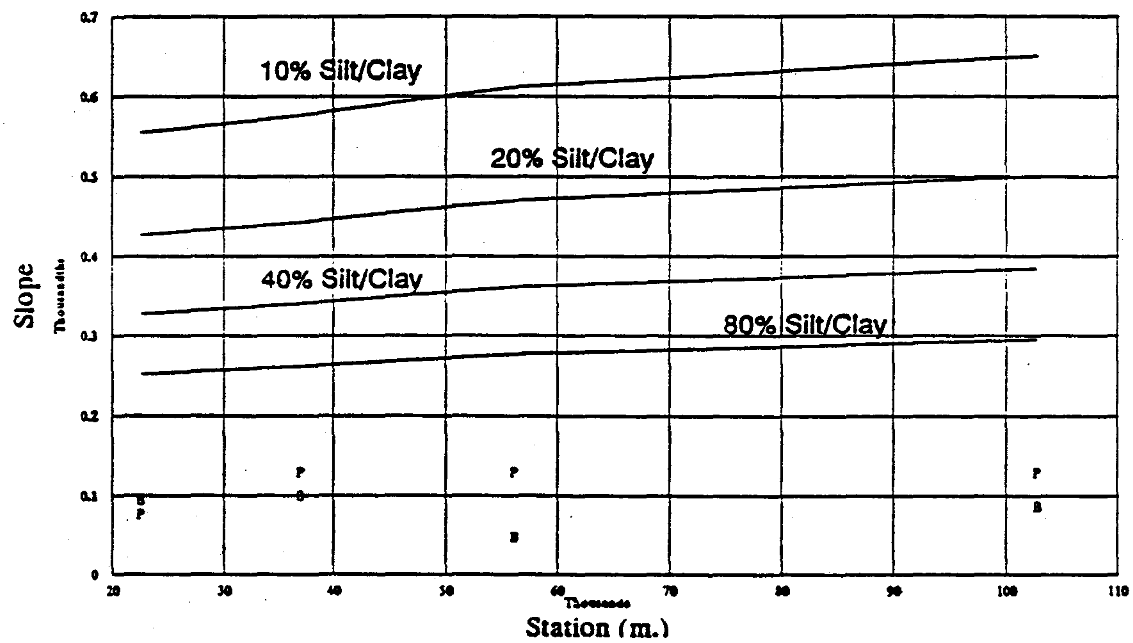

Figure 14 **The actual slope for locations along the Rio Bogota: B = actual slope, P = project slope. The lines depict predicted slope values for Rio Bogota average flow and for varying percentages of slit/clay using a relationship by Schumm (1977) from Australian and US rivers.**

LANDFORMS

Up until now in this paper, we have talked about concepts, introduced a few new words, and have presented empirical relationships and have talked about the lack of universal applicability of empirical relationships. Please understand that empirical relationships are of considerable value, but be cautious. Now it is time to give you a brief introduction into what you may see when you go to the field. The following discussion will be confined primarily to depositional landforms along meandering rivers, and a little information concerning terraces.

A ***floodplain*** is the alluvial surface adjacent to a channel that is frequently inundated (Figure 15). This is a simple definition of a floodplain; however, the concept that the bankfull discharge is the sole discriminator between channel-forming and floodplain-building process is especially difficult. Although much of the literature up until the 1970s suggested that the mean annual flood was the bankfull discharge, Williams (1978) clearly showed that out of thirty-five floodplains he studied in the U.S., the bankfull discharge varied between the 1.01- and 32-year recurrence interval. Only about a third of those streams had a bankfull discharge between the 1- and 5-year recurrence interval discharge. Knowledge of alluvial landforms will allow a more informed determination of bankfull than depending solely on the magnitude of the flood.

Table 2 and Figure 15 together provide a quick summary of some of the alluvial landforms found along a meandering stream. From the perspective of a bank stabilization planner, it is extremely important to know that all the materials along the bank and in the floodplain are not the same. The materials are deposited under different flow conditions, for example, ***backswamps*** and ***channel fills*** will usually be fine-grained and may be very cohesive. This is because both landforms are deposited away from the main flow in the channel, in a lower energy environment. ***Natural levee*** deposits are coarser near the channel and become finer away from the channel as the energy to transport the larger particles dissipates.

Point bars represent a sequence of deposition in which the coarser materials are at the bottom and the finer materials at the top. From the viewpoint of the channel stabilization planner, the more erosion resistant materials may then be silts and clays deposited at the top and very erosive sand may comprise the toe of the slope. Therefore, if the channel you are attempting to stabilize is eroding into an old point bar deposit, you may encounter several problems. Along the same line of thinking, an abandoned channel fill may appear on the eroding bank as a clay plug. The very erosion resistant plug must be considered in planning.

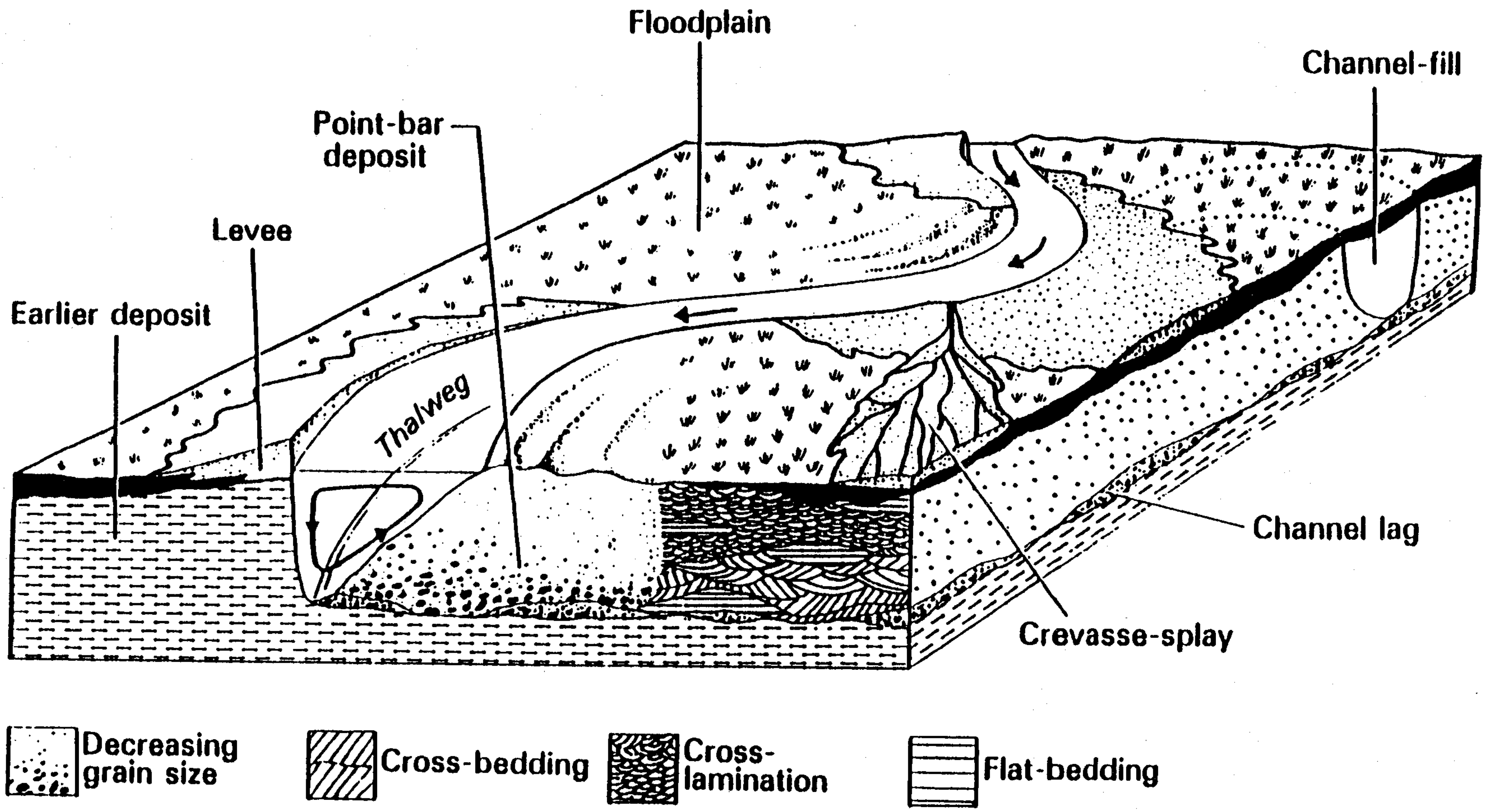

Figure 15 **The classical point bar model for meandering river; a crevasse is a break in the levee (Collinson, 1978 after Allen, 1970).**

Table 2. Classification of Valley Sediments

Place of Deposition (1)	Name (2)	Characteristics (3)
Channel	Transitory channel deposits	Primarily bedload temporarily at rest; for example, alternate bar deposits.
	Lag deposits	Segregation of larger of heavier particles, more persistent that transitory channel deposits, and including heavy mineral placers.
	Channel fills	Accumulations in abandoned or aggrading channel segments, ranging from relatively coarse bedload to plugs of clay and organic muds filling abandoned meanders.
Channel margin	Lateral accretion deposits	Point and marginal bars which may be preserved by channel shifting and added to overbank floodplain by vertical accretion deposits at top; point-bar sands and silts are commonly trough cross-bedded and usually form the thickest members of the active channel sequence.
Overbank flood plain	Vertical accretion deposits	Fine-grained sediment deposited from suspended load of overbank floodwater, including natural levee and backswamp deposits; levee deposits are usually horizontally bedded and rippled fine sand, grading laterally and vertically into point-bar deposits. Backswamp deposits are mainly silts, clays and peats.
	Splays	Local accumulations of bedload materials, spread from channels on to adjacent floodplains; splays are cross-bedded sands spreading across the inner floodplain from crevasse breaches.
Valley margin	Colluvium	Deposits derived chiefly from unconcentrated slope wash and soil creep on adjacent valley sides.
	Mass movement deposits	Earthflow, debris avalanche and landslide deposits commonly intermix with marginal colluvium; mudflows usually follow channels but also spill overbank.

Different types of bank instability can also arise depending on how the materials were deposited. Consider a point bar deposit with a sandy base that has been deposited over a backswamp clay deposit. This can result in sub-surface flow at the sand-clay interface which can cause the granular material to be washed out of the bank and failure to occur some distance back from the channel. Stabilization could include proper drainage of the top of the bank to deprive the failure mechanism of the percolating groundwater source.

In addition to the landforms briefly described in Table 2, we should introduce ***terraces***. Terraces are abandoned floodplains that were formed when the river flowed at higher level than at present (Ritter, 1978). Terraces are produced by incision of the floodplain (Schumm, 1977). In other words, the stream channel has downcut leaving the previous floodplain, and is establishing a new, lower floodplain. The appearance of a terrace or a series of terraces in a surveyed cross-section may be as broad stair steps down to the stream. The steps may be broad and continuous throughout the entire length of the stream segment, or may be discontinuous and could be only a few feet in width. The key importance of identifying a terrace is that something has caused the stream to incise. When did it happen and why? Also, if you have enough information, you may see that the slope along the top of the terrace (the tread) is not the same as the present water surface of the stream. If the tread slope is steeper than the present thalweg, what has caused this to happen?

One explanation could be that upstream dam construction reduced the sediment discharge. This could cause incision (degradation) of the stream which results in terrace formation; however, if the stream adjusts to a lower sediment supply, the slope would be less. The important point is to begin learning something about geomorphology and what to look for in the field.

CLOSING

In planning a project along a river or stream, awareness of even the fundamentals of geomorphology allows you to begin to see the relationship between form and process in the landscape. Go into the field and take notes, sketch, take pictures - and above all, think. When you are in the field, look at the variation in particle size of the sediment, and think about the relationship between particle size and the local energy of the flow. Then you begin to have some understanding and can perhaps begin to predict what sort of landform may result if your project alters the flow patterns. Look at your surroundings in the field, you might find yourself with a shovel or geologists pick in your hand. Try to establish a connection between what you see (form) and why it is there (process). Then you are beginning to think like a geomorphologist. As Dr. Einstein said in the closing comments of his retirement symposium: *It is in the field where we can find out whether our ideas are applicable, where we can find out what the various conditions are that we have to deal with, and where we can also find out what the desired improvements are.*

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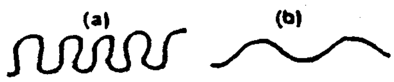
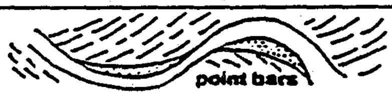
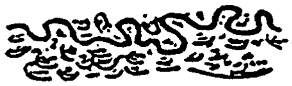
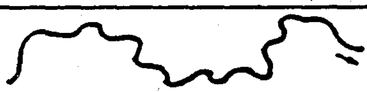
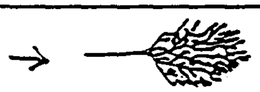
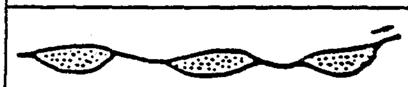
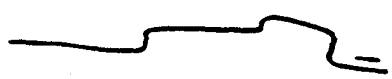
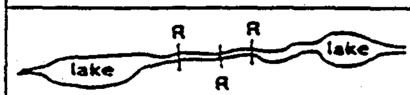
CHANNEL APPEARANCE	CHANNEL TYPE	TYPICAL ENVIRONMENT	TYPICAL BED AND BANK MATERIALS
	(a) Regular serpentine meanders (b) Regular sinuous meanders	Lacustrine plain	Uniform cohesive materials
	Tortuous or contorted meanders, no cutoffs	Misfit stream in glacial spillway channel	Uniform cohesive materials
	Downstream progression	Sand-filled meltwater channel	Slightly cohesive top stratum over sands
	Unconfined meanders with oxbows, scrolled	Sandy to silty deltas and alluvial floodplains	Slightly cohesive top straturn over sands
	Confined meandering	Cohesive top strata over sand substratum in steep-walled trench	Slightly cohesive top stratum over sands
	Entrenched meanders	Hard till or uniform rock	Till, boulders, soft rock
	Meanders within meanders	Underfit streams in large glacial stream spillways	Cohesive materials
	Irregularly sinuous meanders	Thin till over bedrock in plains	Hard and softer materials
	Wandering	Foothills and mountain valleys	Cobble-veneered sand
	Anastomosing	Foothills, plains. Sand bed or gravel paved rivers	Sand and gravel
	Classical braided	Glacial outwash. Foothills	Sand and gravel
	Dichotomic	Alluvial cones and fans	Gravel, sand, silt
	Irregular channel splitting	Large rivers in bedrock	Alternate sand, gravel and rock
	Rectangular channel pattern	Jointed rocks, mostly flat-lying sedimentary rocks	Rock
	Lakes and rapids (R)	Till-veneered Shield terrain	Till, cobbles, boulders, hard rock

Figure 2-11. Some forms of stream planform (after Mollard and Janes 1984)

STUDY PLAN AND PROJECT ALTERNATIVE FORMULATION

Lecture 4

THIRD INTERNATIONAL SYMPOSIUM ON RIVER SEDIMENTATION

The University of Mississippi, March 31-April 4, 1986 pgs 1169-1173

PRACTICAL FLOOD-CONTROL CHANNEL DESIGN
Adapted for Flood Control Short Course by D. Williams

John J. Ingram[1]

Technical guidelines for the design of a flood-control channel are provided in this paper. The basis behind this design approach is that a channel is part of a system and not an entity to itself. Use of this study plan is advantageous as it permits an assessment at various stages of a project's design and progression to more detailed studies as may be required for complex projects.

Introduction

In past practice the hydraulic design study plan for flood-control channels was primarily concerned with the conveyance of a stream and gave little consideration to sedimentation and channel stability.

It is now evident that channels which are modified for flood protection must not only be designed for conveyance but also for stability, since channel responses may lead to a loss of conveyance, increased property damage, and excessive channel maintenance. With this understanding, the flood-control channel design guidelines recommended by this paper assist the designer in understanding the potential for channel response and in planning channel modifications to function with, rather than against, the natural channel dynamic tendencies.

The basis behind this flood-control channel design approach is that a channel is part of a system and not an entity to itself. This system may be looked upon as being made up of three zones (Schumm 1977):

sediment production zone,
sediment transfer zone,
sediment deposition zone

For a channel's system, these zones may successively be thought of as the drainage basin, the channel, and the channel's exit.

Physical controlling parameters that are important to the stability of these areas are (Simons, Li and Associates 1982):

topography	soils	geology
vegetation	climate	hydrology
hydraulics	sediment transport	land use

Just as Lane (1955) described the sediment balance for a channel's reach (S_c $Q_w \propto D_{50} Q_s$; where S_c is the channel slope, Q_w is the stream discharge, D_{50} is the median sediment size, and Q_S is the sediment discharge), the relative types and magnitudes of these physical parameters are important in a channel system also.

With the system's attempt toward a balance of these parameters, a change in one parameter at one location in the system can lead to a change in other parameters at that and other locations. It is the variation of these parameters that make the channel system dynamic. Therefore, when dealing with a channel modification project, a spatial and temporal concern must exist for system response beyond the project reach for the project life.

Approach

The flood-control channel design study plan now recommended contains four study phases:

historical analysis of the river system

a ranking of proposed flood-control channel modifications

a qualitative analysis of the channel's response following modification

a quantitative analysis of a proposed project's stability

Use of this study plan is advantageous as it permits as assessment of the project at various levels of design (i.e., predesign through detailed design).

Other advantages of this approach include its applicability to several channel types and its assistance in channel maintenance planning.

Historical Analysis

The designer of a local flood protection project must become familiar with the projects stream system.

The first step for this is to conduct a historical analysis of the system. Using data collected from a review of historical records and a field reconnaissance, a channel-evolution model is produced, showing geometric changes along the stream over time and changes in the physical controls that govern channel regime.

From this analysis the reviewer can determine:

(a) if the stream system is in quasi-equilibrium

(b) if the stream is close to a threshold condition of channel-form change

(c) the potential for channel stability following the implementation of a proposed channel modification scheme.

Historical Records Review

Before conducting a field reconnaissance, the designer should become familiar with changes in the physical controls of the river system through an examination of the historical records.

These records may be comprised of topographic maps, aerial photographs, cross-section survey data, hydraulic and hydrologic records, information on existing flow control structures, soil records, environmental data, land use information, and any other available historical data (see Simons and Senturk - 1976 for a checklist of data needs for a complete river system analysis).

This examination of the historical records is most useful if it includes measurements of changes in parameters such as channel sinuosity, channel width, gully development, and erosion and deposition on the channel bed and banks.

As such parameters are being examined, adjustments to the channel geometry following major discharge events, changes in land use, diversion of flow for irrigation, channel constrictions due to bridge crossings, and other changes should be noted.

These notations will strongly indicate the processes that have had a major influence on changes of the channel form and the equilibrium status of the channel. These notations will also direct the designer toward recommendations for stability control of the modified channel system.

Field Reconnaissance

The completion of data collection for the historical analysis comes with a field reconnaissance of the stream. As this reconnaissance is made, the observer must remain cognizant of the physical controls that influence channel stability and how to identify and interpret these controls.

This reconnaissance is most beneficial when it is made up of observations from the land and the air with a team of **specialists in hydraulics, hydrology, geology, ecology, and economics.**

Such a team is beneficial to the study since interdisciplinary factors impact the feasibility of the project.

Observations should give consideration to the channel system's present stability, the impact of modifications upon the system's future stability, the post construction maintenance requirements for the system, and the economic justification for the project.

The field reconnaissance is considered complete once an appropriate amount of data has been collected that will allow a reasonable assessment of the channel's stability relative to the complexity of the project.

Channel Stability Assessment

A stable channel implies there is a balance between the physical process that influence the stream's boundary, that is, the stream is in a state of equilibrium. However, due to temporal variability in the magnitudes of the physical processes, stream systems can at best be in a state of quasi-equilibrium.

To assess the equilibrium of a channel, the collected historical office and field data are used. These data are most helpful in they consist of the temporal and spatial variation of a channel's cross section, slope, and alignment.

Other data to be reviewed may be change in land use and hydrology. By reviewing these parameters, an assessment of the channel's present equilibrium status can be made.

The channel stability assessment is performed in both a qualitative and quantitative fashion.

The qualitative assessment is based upon the field reconnaissance team's observation and experience-based interpretations of the collected data. The quantitative assessment can be performed under the direction of this team using procedures such as those proved by Lane(1957), Leopold and Wolman (1957), Parker (1976), Simons and Senturk (1976), and Schumm (1977).

As a result of this assessment, modifications for flood control can more easily be ranked such that we can work with, rather than against, the system.

Ranking Flood-Control Alternatives

Recognizing a stream system's present stability status and its potential response to various flood-control alternatives will enhance the selection of a reasonable flood-control measure.

With reference to channel stability and response, Harrison (1981) suggests that the selection of an alternative that affects the natural regime of the system the least is best.

Or, stated another way, the flood-control alternative that encroaches upon the natural channel regime the least should be considered first. This guidance is based upon the reasoning that a channel's natural regime is the most stable.

Ranking various alternatives will be dependent upon individual project situations; however, in the general case, modifications may be ranked as follows:

(a) selective clearing and snagging

(b) use of levees with little or no channel modification

(c) excavating a floodway for high flows

(d) excavation of berms on a natural channel bank

(e) channel enlargement and realignment

Some projects become more complex when multiple modifications are to be incorporated in the design. For these projects, the ranking of alternatives may be more complex than the simple listing just provided; however, the general guideline to attempt to have as little impact upon the channel system's natural regime can still be applied.

Flood-control alternatives sometimes need to be ranked according to project design constraints rather than in accordance with a channel's regime.

Other desirable constraints are to provide the greatest level of safety to human life, the greatest level of flood protection, and the least channel maintenance requirements.

Additional constraints include a sponsor's desired improvement method and/or the economic justification of a channel modification.

Qualitative Response Analysis

A qualitative analysis provides insight to a channel system's response to natural or imposed changes. It therefore provides a means by which alternative channel system modifications may be compared. This analysis is useful as it can provide information on what may be expected prior to conducting a quantitative analysis. Such a study is also beneficial when sufficient data and time are not available for a quantitative analysis.

When data are available for a quantitative analysis, a qualitative study can help determine which quantitative studies would be most desirable.

For some projects (in particular, those projects which are not complex), the information provided from this analysis, in conjunction with the experience of the assessment team, may be sufficient for the selection of a flood-control protection alternative.

Tools for a qualitative analysis have been presented by Leopold and Maddock (1953), Lane (1957), Santos and Simons (1972), and Schumm (1977).

These have been used by Simons and Senturk (1976) to form relationships between the following parameters: depth of flow, water discharge, channel width, sediment discharge, channel width to depth ratio, channel slope, sediment grain size, channel sinuosity, valley slope, stream power, fine material concentration, and the fall diameter for the median bed material size.

Using relationships such as these will assist in determining the impact of a change in one parameter at one location in a system upon other parameters at that and other locations.

Note that since a channel is part of a system, the qualitative analysis should be performed for the project reach and upstream and downstream of that reach. Example applications of making qualitative analysis on river systems may be found in Simons and Senturk (1976) and Schumm (1977).

Quantitative Analysis

A quantitative analysis can be an aid at several levels of a project's study. For instance, such an analysis can benefit an assessment of a project's feasibility as well as assist in a project's detailed design.

It must be realized, however, that the usefulness of a quantitative tool is only as good as the available data used and the assumptions of the chosen tool. Therefore, the usefulness of the analysis is dependent upon the information available, the selected quantitative tool, and the level of information required to make a reasonable assessment for the project.

Tools

Two basic approaches have been considered for the development of quantitative tools: rational and empirical.

The rational approach is founded on theory, while the empirical approach is founded on laboratory and field observations.

Several quantitative tools have been developed by way of these two approaches and caution must be taken in their application. This precaution is due to dissimilarity in the assumptions of these methods and therefore also in their results.

Therefore, the selection of a method should be based upon its applicability for systems similar to the one being studied and its sufficiency to allow a sound assessment for the level of design for which this tool is being applied.

Some tools that may be considered for a channel design study include: allowable mean velocity, tractive stress, tractive power, modified regime (Simons and Senturk 1976 and USDA 1977), Neill's regime procedure (Neill 1984), and a sediment balance procedure (refer to a sediment routing numerical model and/or sediment transport text).

If one of these tools does not appear to be appropriate, and sufficient data are available, an empirically based regionalized procedure could be developed (Schumm, Harvey, and Watson 1984).

Tool Selection

The selection of a tool depends on the questions asked, the information known, and the detail required in the answer. If a tool is selected using these constraints, the most reasonable answer for the current level of study will more likely occur.

At the pre-design level, quantitative tools may supplement the interpretation of the qualitative tools, and therefore, direct the designer toward a reasonable modification scheme selection.

As a study progresses to the detailed design level, more extensive analyses may be conducted using more data with geomorphic, hydrologic, hydraulic, and sediment routing numerical models. When these tools are used, the present and projected stability of the system must be considered. This consideration should account for the potential response upstream, downstream, and at a project's location for all experienced flows.

Acknowledgment

The information presented herein, unless otherwise noted, was obtained from research conducted under the Civil Works Program of the US Army Corps of Engineers by the Waterways Experiment Station. Permission was granted by the Chief of Engineers to publish this information.

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Hydrologic Engineering Investigations

Adapted from lectures of David Ford, Ph.D., P.E. in Ref. 8

Problem

A. Determine type, size, location, etc. of measures.

B. Criterion for best is maximum net benefit.

1. Net benefit = benefit - cost

   = Inundation reduction benefit - cost

   = Damage with project - damage without project - cost

C. How are flood damages estimated?

D. How do floodplain-management measures reduce damage?

Plan Evaluation Procedures

A. Deterministic

1. Use "design" event or historical record
2. Reliability unknown

B. Probabilistic

1. Infer risk from historical record
2. Reliability known

Deterministic

A. This approach to planning uses a "design" event or the historical record as the criteria for design.

1. Examples:

   a. Use the flood of record to size a levee.

   b. Use PMF to size a spillway.

   c. Use historical sequence to allocate reservoir storage.

2. Question: How reliable is the project if we design for the flood of record?

   What's the risk?

   Answer: We don't know.

Probabilistic

Analyze historical record to infer characteristics of long-term behavior.

Plan/design/operate for this long-term behavior.

1. Examples:

   a. Size levee so long-term damage reduction > cost.

   b. Operation reservoir so risk of power shortage < acceptable level.

EAD = Expected Annual Damages

A. We can compute EAD by computing area beneath plot of damage vs. annual probability of exceedence.

B. Probability - It's a measure of risk of occurrence or exceedence.

1. Magnitude: $0 \leq \text{probability} \leq 1$

2. Streamflow data - use Statistical analysis

3. No streamflow data, use:

   a. Regional frequency analysis

   b. Rainfall - runoff modeling

4. Discharge - Probability and elevation - discharge relationships are determined with hydrologic engineering techniques.

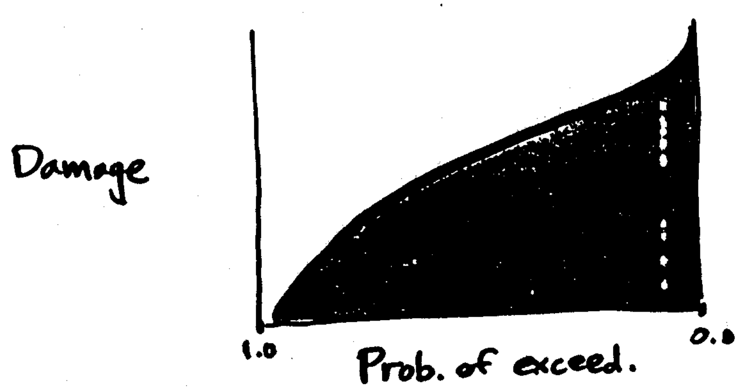
Alternative "Labels" for Frequency Curves

- Exceedence frequency = usually prob. * 100
- Exceedence probability = prob. of exceed.
- % - Chance exceedence = probability * 100.
- Return period = 1./probability
- Recurrence interval = 1./probability
- Exceedence interval = 1./probability

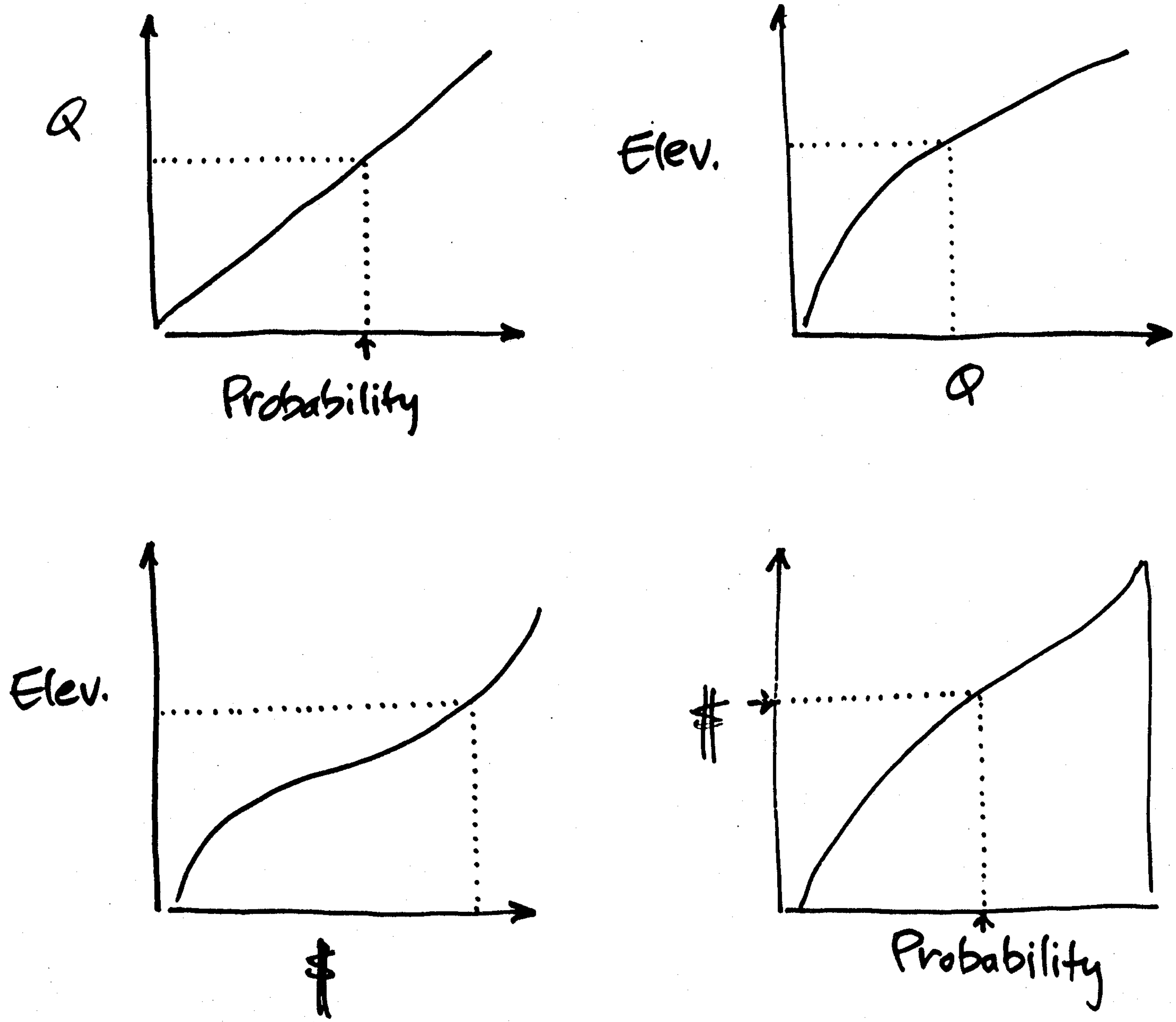
EAD

- $EAD = \int (\text{Damage})(\text{Probability of damage})$

- We can compute EAD by computing area beneath plot of damage vs. annual probability of exceedence.



EAD Computation



CONCEPTS AND CONSIDERATIONS IN FLOOD CONTROL CHANNEL DESIGN

Lecture 5

CONCEPTS AND CONSIDERATIONS
IN FLOOD CONTROL CHANNEL DESIGN

Adapted for Flood Control District short course from Ref. 8.

Alfred S. Harrison

Warren J. Mellema

A. Investigate stream's present regimen - "Get to Know Your Stream."

1. Hydrologic conditions

   a. Flow duration, peak Q, frequency.

      Will dams change them?

   b. Recent flood history. Recent super flood?

   c. Where are we in hydrologic cycle?

      Drought? Wet Cycle?

2. Channel alignment in valley

   a. Upstream and downstream from project.

   b. Best ideal -- examine aerial photo.

   c. Aerial reconnaissance flight is valuable.

   d. Evidence of channel meander - oxbows, meander scrolls, etc.

3. Profiles upstream and downstream from project site

   a. Valley and channel bed profiles

   b. U.S.G.S. topographic sheets - good source of rough profile data

   c. Investigate anomalies in profile

      - "Stairsteps" might show degrading or head-cutting trend

      - "Flat stretches" - possible sediment overload.

   d. Geological exploration

4. The channel

   a. Dimensions - depth, width, bank slopes

   b. Bed material samples for mechanical analysis - sandy, gravely, boulders, rock ledge, cohesive silt or clay, silt stones or shales, etc.

   c. Evidence of underlying material - shallow layer of erodible material over unerodible material.

      - Channel incised in clay valley fill but with sand just underneath (very important).

d. Classify channel as alluvial or non-alluvial

- Alluvial: channels usually in deep sandy beds.

- Alluvial: stream is flowing over a deep bed of the same material it is transporting, scouring, and depositing.

- Alluvial: channels a bed load function exists - special care needed in designing for sediment loads.

- Non-alluvial: has a thin layer of transported sediment with more resistant material underneath, i.e., sand on cohesive clay or silt, sand on silt stone, sand among cobbles or boulders.

- Cohesive bed material may scour, but it becomes wash load and does not redeposit to reform bed.

e. Bed roughness

- Vegetation?

- Shifting sand bars? Evidence of form roughness that changes with Q.

f. Banks

- Vertical or sloping? Indicates type material and/or degradation trend.

- Recent caving or slumping - possible degradation trend

- Sandy? Cohesive? Bank material indicates degree of lateral constraint on channel width

- Role of vegetation - erosion protection, roughness, ecology

- Evidence of seepage thru bank - cause of bank instability

- Are banks naturally stable against internal sloughing and low flow attack?

- Evidence of sand layer?

- Lateral rock controls

g. Valley cross section

- Is channel incised? Perched?

- Natural levees? Do they impede valley drainage?

5. Channel shifting - historical information on rapidity of channel shifting, migrating.

   How wide is current meander belt and what is frequency channel sweeps meander belt?

6. Human changes

    a. Existing channel improvements

        - How have they behaved?

        - What has worked and what has not?

    b. Clearing and irrigation - future trends?

7. Current channel trends

    a. Widening - look at bridges for added approach spans.

    b. Deepening - look at bank sloughing and head cutting on tributaries.

        Look at bridge piers and abutments for exposed footings.

8. Hydraulic roughness study

    a. Analyze other reaches

        - Where rating curve data are available

        - Special X-section and profile probably needed.

    b. Reproduce flood profiles with backwater computations.

c. Look at variable n with Q in wide, sandy channels

d. Composite "n" - banks and bed

e. Be alert for chance to analyze existing channel improvement

B. Preference for channel improvement schemes

"The best scheme interferes with the stream condition the least."

1. Order of preference

a. Levees set back - leave natural channel alone (ecological advantage as well)

b. Use natural channel but excavate adjacent floodway for flow capacity

c. Use as much of natural channel as possible for lower flows - excavate berms for capacity

d. Excavate new channel

2. If you straighten existing channel you incur added cost of grade control.

3. If you disturb existing banks you incur added cost of bank protection.

C. Improved channel alignment

1. Try to use as much of natural alignment as possible.

2. Minimize straightening.

3. Recognize ecological advantage of minimal channel work.

4. We recognize that for many projects, radical channel realignment will be necessary.

D. Improved channel cross section

1. IMPORTANT - Design the channel shape for low flow as well as high flow.

2. Q versus velocity plot is a simple but useful tool for comparing natural and design channel transport - erosion - deposition characteristics.

   Try to match design with natural as closely as practicable and up to as high a Q as practicable. (Where reservoirs will alter Q regime, compare velocity-duration curves.)

3. Improved channel too narrow (velocity too high).

   You can get away with this, but at expense of grade control and bank protection.

4. Improved channel too wide (velocity too low).

   You usually can't get away with this because:

   - Stream will silt in channel

   - High operation and maintenance cost to retain capacity

   - This applies to side slopes - too flat - stream will deposit berms - anticipate in hydraulic computations.

E. Improved channel grade

1. Deepening upstream along with straightening can restore natural grade.

2. Too much deepening at upstream end may require drop structures.

3. What material are you deepening in? Natural channel entirely in clay can be deepened into underlying sand --- TROUBLE.

   - Pot holes

   - Bank toe cutting

   - Need for grade control

F. Consider need for grade control.

No stable banks without grade control

1. Estimate potential for degradation.

   Good rule of thumb - extend natural grade upstream through the improvement to get difference between extended profile and design profile.

2. If slope steepened through project, grade control may be needed.

3. If project short enough, 1 or 2 ft. potential degradation might be tolerated by stream.

4. Upstream headcutting may require drop structure to avoid inducing damage.

5. Look for degradation potential outside (usually below) the project.

   a. Degradation trend (headcut) working its way from downstream

   b. Degrading base level at mouth of stream. (Degradation on main stem induces degradation up the tributary.)

6. Degradation sometimes can be tolerated where bank sloughing consequences not serious.

    a. Where improved flow capacity would be beneficial.

    b. Where improved drainage beneficial.

    c. Where costs of underpinning bridge piers and abutments can be tolerated.

    d. Where levees or other installation not adjacent to channel.

    e. Where upstream damage and increased degradation up tributaries can be tolerated.

G. Consider the need for bank protection.

Remember bed stability must be assured first. Consider and weigh factors that make bank unstable.

1. Raw bank versus seasoned bank.

    Time for seasoning - calculated risk.

2. Local scour at toe due to shifting sand bed (not necessarily a degradation trend).

3. High velocities generally.

4. Local high velocities

    - Bends

    - Bridges and structures

    - Channel contraction

5. Ice and trash attack

6. Sand lens in bank or bed

H. Special attention to sand exposed in banks. Loss of noncohesive sand means trouble.

1. Blanket by rolling cohesive material

2. Low rock toes when sand near bed

3. Riprap

4. Watch for berm excavated into sand - blanket and seed.

I. Control of side drainage

1. Avoid slope erosion

2. Bring in at controlled point

3. Headcutting on tributaries may require special structures

4. Perched channel, natural levees give special drainage problems

J. Seeding - treatment of levees and floodway

Ecological requirements might not be compatible with hydraulic design assumptions (trees, unmowed grass, etc.).

K. Post construction evaluation

1. As a construction cost

   a. Install one or more gages for stage measurement.

   b. Have reliable as-built or survey record X-sections with ties so they can be relocated.

   c. Establish bench marks along project for handy leveling to the water surface.

2. Inspection

   a. Designers seize every opportunity to observe project performance at low or high flow.

   b. Periodic engineering inspections

      - Hydraulic designers have role in inspection

      - If problems are apparent, survey X-sections as part of inspection or get locals to do so.

Table 6.8
Design Checklist for Artificial Channels

Item	Section Reference
Simplified Design Procedures	
* When simplified procedures can be used	6.6.1.1
When more thorough analysis is required	6.6.1.2
Initial Data	
* Existing structures	2.3
* Existing channel characteristics	2.3, 6.3.1, 6.3.3.1
* Existing grade control	2.3, 6.3.1, 6.3.3.3
* Existing flood performance characteristics	2.3
* Scour observations	6.5.6
* Existing stream development	2.3
* Land use changes	2.3
Flood history	Drainage Design Manual, Volume I
Rainfall/Runoff relationships	Drainage Design Manual, Volume I
Possible Components and Strategies	
* Channels	6.3.2, 6.5
* Alignment	6.3.3.3, 6.3.3.4
* Grade control structures	2.3, 6.3.3.3, Chapter 7
Consideration for Right-of-Way	
Migration	
* Water level	2.3
Economic and Alternative Analysis	
* Designation of significantly different concepts	
* Type of lining	6.3.2.1, 6.5
* Type of cross section	6.3.2.1, 6.3.3.3
* Channel alignment	6.3.1, 6.3.3.3
* Location of grade control(s)	6.3.2.5, 6.3.2.4, 6.3.3.3
* Hydrologic and hydraulic detail	6.3.3.2, Drainage Design Manual, Volume I
* Least total expected cost evaluation	Not in chapter
Extreme flood evaluation of components and alternatives	Not in chapter
Environmental considerations	2.3
* Documentation and comprehensive evaluation	2.3
* Safety requirements	6.3.2.5, Chapter 5
Hydraulic Analysis	
* Determination of control	6.3.2.4, Chapter 7
Determination of type of flow profile	6.4.1
* Normal depth calculations	6.3.3.2
* Water surface profile calculations	6.4.1.2
* Bridge hydraulics	Chapter 5
* Channel lining	6.5
Supercritical channel hydraulics	6.3.3.3
Superelevation	6.3.3.3
* Drop structure hydraulics	6.5
Physical hydraulic models	6.4
* Low flow channel	6.3.3.3
Sediment Transport Analysis	
Required when simplified design procedure cannot be used, reference to natural channels	Table 6.10, 7.1
Additional Considerations	
Permanent record	
Post construction data	
Normal inspection (references)	

* Required for Simplified Design Procedure

Table 6.9
Design Checklist for Natural Channels

Item	Section Reference
Initial Data	
Existing structures	
Channel characteristics	6.4.1.2, 6.4.1.3
Existing flood performance characteristics	6.4
Existing grade control	6.4
Scour observations	6.3.2.1, 6.5.6
Existing stream development	6.6.2
- Dams, diversions	
- Flood control	
- mining	
Flood history	Drainage Design Manual, Volume I
Rainfall/Runoff relationships	Drainage Design Manual, Volume I
Possible Components and Strategies	
Channels	6.4.5.5
Bridge components	6.4.1, 6.6.2
River alignment control strategies, mitigation	6.4
Alignment control structures	Chapter 7
Grade control structures	Chapter 7
Non-Structural measures (easement, acquisition)	
Economic and Alternative Analysis	
Designation of significantly different concepts	6.4.5.5
Hydrological & hydraulic detailing of alternatives	6.3.3.2, Drainage Design Manual, Volume I
Least total expect cost evaluation	
Extreme flood evaluation of component	
Environmental considerations	
Documentation and comprehensive evaluation	
Hydraulic Analysis	
Determination of control	
Determination of type of flow profile	Figure 6.3, 6.4.1
Normal depth calculations	6.3.3.2
Water surface profile calculations	6.4.1.2
Bridge hydraulics	Chapter 5
Sand bed formation determination	
Sand bed roughness	
Cobble, boulder, or riprap roughness determinations	6.5.3
Vegetation or combination lining roughness	6.6.3.1
Dune and antidune height	
Supercritical channel hydraulics	6.3.3.3
Drop hydraulics	6.5.6.3
Average characteristics	
Physical hydraulic models	6.4
Sediment Transport Analysis	Table 6.10, Table 5.5
Additional Considerations	
Permanent record	
Post construction data	
Normal inspection (references)	

SPECIAL FEATURES OF

FLOOD CONTROL PROJECTS

Lecture 6

Special Features of Flood Control Channels

Adapted for Flood Control District for short course be D. Williams, Ref. 8

By Tasso Schmidgall

1. INTRODUCTION.

The three most common structures used to provide flood protection are:

channels

levees

and dams.

The most widely used is the channel.

Channels appear to be the simplest type of flood protection, but from a hydraulic standpoint, they are the most complex.

When channels are used to reduce flooding tendencies, the hydraulic characteristics of natural streams are frequently altered enough that special channel features are required to ensure stable performance.

This presentation identifies five types of these special channel features (or structures):

1) drop 2) inlet 3) diversion 4) lining 5) transition

The function and common types of each structure are described. The information presented should help planners and engineers to understand why special channel structures are required:

- that these structures can be costly; and that they need to be planned for early in the project development process.

2. FLOOD CHANNEL PERFORMANCE.

a. Flood Reduction Methods.

Several methods are available for increasing the flood carrying capacity of natural streams. One or more methods may be used to develop a project. A description of the four most common methods follows:

1) Reduce the natural stream roughness by removing vegetation and debris from the channel and overbanks; by providing a uniformly shaped channel cross-section; by smoothening the stream alignment; and/or by adding a smooth concrete lining.

2) Reduce the stream length by cutting straight channels through stream reaches which have natural bendways.

3) Enlarge the stream cross-section by widening, deepening, and/or adding levees.

4) Divert portions of the flood flows away from the project stream into temporary detention basins, through cutoff channels to adjacent watersheds, or through diversion channels around the protected area.

b. Hydraulic Consequences.

Particularly for streams which flow through alluvial materials, physical alterations designed to reduce the natural flood stages will result in changes to other hydraulic characteristics of the stream. Common consequences of the above four flood reduction methods are described.

(1) Reducing either the channel roughness or the stream length will cause an increase in the stream flow velocities.

The potential consequences of this change are stream bed degradation, stream bed headcutting, bankline sloughing, bankline erosion, and an increase in sediment carrying capacity of the stream. These changes will be found to affect not only the project stream but also the tributary streams which flow into it.

Adverse effects of these stream reactions include:

(a) Loss of usable land through streambank erosion and the development of headcutting gullies.

(b) Destruction of facilities which are located near the stream or which cross under or over it.

(2) When flood stages are reduced by enlarging the channel cross-section, the hydraulic response is a reduction in flow velocities.

Diversion channels reduce peak discharges in project reaches and usually also result in reduced velocities. Lowered velocities cause a reduction in the sediment carrying capacity of the stream.

Sediment loads which previously were transported through the project reach are now deposited within the stream aggradation. This consequence burdens the project sponsor with the chronic and expensive task of removing sediment deposits from the stream to maintain the degree of flood protection for which the project was designed.

(a) Solutions.

The purpose of special features for channels is to minimize these adverse consequences resulting from the construction of flood control projects.

3. GENERAL PLANNING AND DESIGN FACTORS FOR CHANNEL FEATURES

a. Construction Materials.

A wide variety of materials have been used to construct special channel features. Appropriate material selection is dependent structure needed, the foundation conditions, the hydraulic conditions, and local availability of materials. In many instances, structures are constructed of more than one type of material.

Some advantages and disadvantages of the more common types of construction materials are presented. Examples of structures constructed of each type are presented in the subsequent paragraphs which describe channel features.

(1) Stone.

In most project locations, natural stone is generally available and fairly economical. It has been used both in a quarry run (randomly graded) form and a riprap (specifically graded) form. It is flexible and easy to repair. However, stone will withstand only limited flow velocities. It frequently requires a gravel or cloth filter bed to avoid leaching of the underlying ground materials through the stone. Riprap is commonly used around the periphery of more rigid structures.

(2) Wired stone.

Riprap can be made more resistant to flow and still retail its flexibility by anchoring it with a wire mesh or containing it in inter-connected wire baskets - commonly called gabions. The construction process for wired stone is labor intensive. Durability can also be a problem in projects which have corrosive water or where high velocities carry large sediment loads which are abrasive to the wire.

(3) Grouted Riprap.

Riprap can also be made more resistant to flow velocities by grouting the voids between the stones and concrete.

However, once grouted, riprap looses its flexibility and porosity.

Consideration needs to be given to potential hydrostatic pressure buildup beneath the material. Structural displacement can occur if adequate relief is not provided.

Another disadvantage is the difficulty of obtaining a consistently high-quality product in the construction process. This makes it difficult for designers to determine the actual degree of protection provided by the structure.

(4) Concrete.

Concrete is a good material for construction of channel structures. It is durable, has a low flow resistance, and is readily adaptable to the streamlined shapes needed to control high velocity flows.

Concrete is inflexible and requires special foundation preparation to ensure against cracking and destruction under shifting ground conditions.

The impervious nature of concrete also requires that adequate pressure relief facilities be provided to prevent structure uplift and possible displacement from hydrostatic forces. Several types of concrete structures are in common use.

(a) Formed Concrete.

Most concrete channel structures are cast in place on the project site.

They include construction forms to hold and embedded reinforcing steel bars to provide the needed strength of forms and steel for poured-in-place structures requires that the site be dry. Therefore, cofferdams and dewatering measures are frequently necessary. These requirements make formed concrete structures labor intensive, time consuming, and expensive. However, the resulting structures usually are the best performing and most durable solutions available.

(b) Flexible-form Concrete.

Fabric containers injected with concrete is a channel slope protection method which has experienced a much wider use in recent years. The fabric containers usually are manufactured of nylon in the shape of bags or mattresses.

These containers are laid out on the graded channel slope and are filled with a pumped concrete to provide a continuous protective layer. The structures are frequently constructed without site de-watering. They ultimately become rigid structures with are subject to cracking from shifting ground conditions.

Provisions for seepage and pressure relief are also necessary. Special provisions are usually vent undermining at the structure edges. Long-term durability has not yet been evaluated.

(c) Precast Concrete.

A number of concrete fabricators now offer precast concrete blocks of various sizes to be used as a channel slope scour protection. They are generally small enough to avoid pressure buildup on the bottom.

Many are shaped to interlock with adjacent blocks to resist displacement. Others are tied together with wires to form a large mat for faster placement on the bankline. Most are porous enough to encourage vegetation growth for extra resistance to flows.

(5) Steel Sheet Piles.

Driven sheet piles are commonly used to construct channel drop structures. They are more flexible, cost less initially, and require less care of water diversion during construction than concrete structures.

However, they are more susceptible to damage from corrosive environments and could be expected to require more periodic maintenance.

b. Capacity and Stability

(1) General.

Early in the project development process, planners and designers need to evaluate and determine the most appropriate hydraulic and structural conditions on which to base the design of each channel structure.

For what discharge should the structure capacity be designed?

Also, for what discharge should the structure stability be designed?

A general rule has been to use the same flow on which the flood channel design is based. In other words, if a project provides 100-year flood protection, size the structure to pass the 100-year frequency discharge. However, this general rule is not always logical and additional factors need to be considered.

(2) Capacity.

The above general rule works fine when project designs provide protection against the 100-year or standard project floods.

However, many flood control projects in are now recommending channel designs which provide protection for only five or ten year frequency floods. In these cases, structure capacities should normally be greater than that needed for the project design flow.

The capacity must be adequate to ensure that flows exceeding the design flood will not induce load damages in excess of pre-project conditions.

Also, using a design capacity greater than the flood protection capacity may prove to be more economical than having to make scour damage repairs each time the design storm is exceeded.

(3) Stability.

When project design flows are slightly exceeded, scour damage is common and expected. When the design flow is significantly exceeded, structural failure is likely.

For example: If a structure is designed to pass only the flows resulting from a 5-year frequency flood, then a 10-year flood could seriously damage the structure and a 25-year flood could completely wash it out. Here the general rule loses its logic.

For this situation, the structures would likely not last as long as the economic life of the project - usually 50 years for flood control projects.

If this rule were followed, project maintenance costs would have to allow for one, or possible even two replacements of the structures over the economic project life. To local sponsors, such project maintenance requirements would indicate a basic design deficiency.

(4) Guidance.

The following rule is practiced by some Flood Control Districts and appears to provide a logical balance of inlet structure cost and protection:

(a) For major channel control structures, provide adequate capacity to safely pass at least the 100-year frequency storm discharge. Allow for some erosion damage (but not structure washout) at the SPF discharge.

(b) For very minor channel control structures, provide adequate capacity to safely pass at least the 25-year storm discharge, and allow for limited scour damages at the 100-year storm discharge. Washout is likely at the SPF discharge.

4. DETAILS OF SPECIAL CHANNEL FEATURES.

The following five sections present the purposes, design factors, and performance Characteristics for the five most common types of special channel structures - drops, inlets, diversions, liners and transitions.

5. DROP STRUCTURES.

a. General.

Drop structures are designed to reduce the adverse effects of high velocities in streams or channels. They accomplish this task by fixing an upper and lower bed control point within a short length of channel and by controlling the turbulence as flows pass from the higher to the lower level.

Drop structures check the upstream migration tendencies of headcuts. When provided over a significant length of channel, they form a series of steps which control the channel grade and reduce velocities.

A series of drop structure designs which cover the wide range of types and sizes are described.

b. Vertical Drop Structures.

The drop consists of an upstream control weir, a downstream basin, vertical training walls, and wing walls to tie the structure into the streambank.

c. Sloped Drop Structures.

Channel drops are frequently constructed of structures with a sloped bottom rather than a vertical drop. The total length of sloped structures is usually more than vertical drop structures. Stability problems are frequently less critical with sloped structures.

6. INLET STRUCTURES.

a. Purpose.

Common consequences to tributary streams resulting from main stream flood improvements include bed degradation, headcutting, bankline caving, erosion, and standing wave turbulence.

Inlet structures are designed to stabilize the hydraulic performance of tributary streams (usually to pre-project conditions) and also to blend tributary flows smoothly into main channel flows.

b. Design Factors.

A wide variety of hydraulic conditions can exist between a stream and its tributaries. Consequently, a wide variety of inlet structures have been developed. Care must be taken to select the proper type.

Several basic factors should be considered in this selection process.

(1) Type of tributary.

Both open channels and enclosed conduits enter flood channels. Specific inlets are designed for each type.

(2) Velocities.

Tributary flows will blend most smoothly with main channel flows if their directions at the point of entry are roughly parallel and if their velocities are roughly equal. Smooth blending is particularly important when flow velocities are high - over 8 to 10 feet/sec. For inlets with velocities under 5 or 6 ft./sec, simple straight inlet junctions have been satisfactory.

Stilling basins are frequently needed within inlet structures to reduce tributary velocities as they enter the main channel.

(3) Relative discharges.

Smoothly blending tributary with main channel flows is particularly important when the tributary flows are a significant portion (over 15%) of the combined flows.

For lesser tributary flows, the preponderance of main channel flows will usually redirect tributary low smoothly into main channel flows.

(4) Flow Coincidence.

Inlet designs must consider the relative times at which flood discharge peaks occur on the main channel and on the tributary. If peaks occur simultaneously, designs can be based on the peak flows. However, if tributary flows peak well ahead of main channel flows, then main channel flow rather than the peak should be considered to ensure proper flow blending at all discharge conditions.

(5) Invert Control.

Flow blending is normally smoothest when the tributary invert is at nearly the same level as the main channel invert. In most flood channel projects, this doesn't occur. If tributary flows are contained in a closed conduit, the conduit invert can be adjusted to match the main channel invert.

However, if tributary flows are in open channels, an inlet structure with an invert control section is usually required to maintain stable flow conditions in the tributary.

The inlet structure may also require a drop section or chute to lower flows to the main channel elevation and a stilling section to dissipate tributary flow energy.

7. DIVERSION STRUCTURES

a. General.

Flood protection for a particular project area can be provided by diverting all stream discharges which exceed the stream's natural capacity away from the project area.

The point of diversion is located upstream from the project reach. Excess stream flows can be diverted into a diversion channel which by-passes the project reach, into an adjacent watershed, or into temporary flood detention basins.

Each type of diversion facility requires special structures to properly control the needed diversions.

b. Diversion channel inlet structure.

The inlet structure is normally a riprapped, sheet pile, or concrete weir section which establishes the elevation at which flood flows begin entering the diversion channel.

Discharge structures are normally simple channel junctions with adequate scour protection to ensure the stability of both the diverting and the receiving channels.

c. Natural channel inlet structure.

This structure ensures that low flows continue to enter the natural stream and flow through the project reach.

The structure also ensures that all discharges entering the project reach are limited to the capacity of the channel regardless of the magnitude of the upstream flood flows.

Adequate flood controls for the natural channel can sometimes be provided with a fixed weir or multiple culvert structure.

However, in some projects, gated discharge controls are required to adequately account for variations in either interior flood runoff or in the natural channel capacity.

c. Detention Basins.

The basic concept of a detention basin is to reduce the peak flood discharges through the project reach by diverting a portion of the peak flows from the flood channel over a control weir into a temporary storage basin or detention site. A low-capacity outlet pipe eventually drains the detention site water well after the flood peak has passed.

8. CHANNEL PROTECTIVE LININGS.

a. General.

The drop, inlet and diversion structures described above all tend to stabilize flood control channels by keeping velocities low enough to prevent flows from displacing the natural materials which comprise the bed and banks of the channels.

An alternative means of stabilizing flood channels is to increase the resistance of these natural materials so that they can resist higher flow velocities. This can be accomplished by adding any of a wide variety resistive materials to the channel bed and side slopes.

b. Protection coverage.

Various degrees of surface protection can be provided for flood channels.

Some projects use a continuous protective lining from beginning to end over both the bed and side slopes.

Others have protection only on the side slopes. Others are protected only at those locations where severe erosion attach is likely to occur.

Still other projects provide scour protection only for the more frequent floods rather than for the full range of channel discharges.

c. Example protection methods.

(1) Concrete linings.

When properly designed, concrete lined channels have been among the most durable and efficient methods of providing flood protection.

The rigid and impervious nature of concrete often subjects channel linings to serious damage and even complete failure from ground water hydraulic uplift forces. Pressure relief drainage systems are often needed to alleviate these forces.

Concrete linings frequently have special construction joints to accommodate minor movements in the foundation materials. Because of the smooth surface of concrete, channel alignments are frequently streamlined to better handle the increased flow velocities.

(2) Riprapped channel.

Probably the most common type of streambank protection is riprap. A number of standard type riprap sections have been developed for streambank protection.

(3) Gabions.

The use of gabions for flood channel protection has become very popular throughout the United States. Both box gabions and gabion mattresses are used.

9. CHANNEL TRANSITION STRUCTURES.

a. General.

Special channel transition structures are frequently needed in reaches which contain fixed obstructions.

Typical obstructions include roadway embankments, culverts, bridges, underground utility crossings, overhead utility supports, and retaining walls for overbank developments.

The design of channels through urban areas can be particularly troublesome when multiple obstructions occur in the same reach. Minor obstructions frequently can be relocated outside the proposed channel limits, but the channel shape must often be adjusted with special transition structures to accommodate major obstructions.

b. Transitions.

Directing channel flows smoothly past obstructions can require transitions in channel grades, bottom widths, and side slopes. The length of transition structures depends on the degree of change required and also on the flow velocity.

Bottom width transitions can vary from an abrupt offset to a reverse curve offset that is long enough to prevent the development of flow separation, standing waves or eddy formations within the channel.

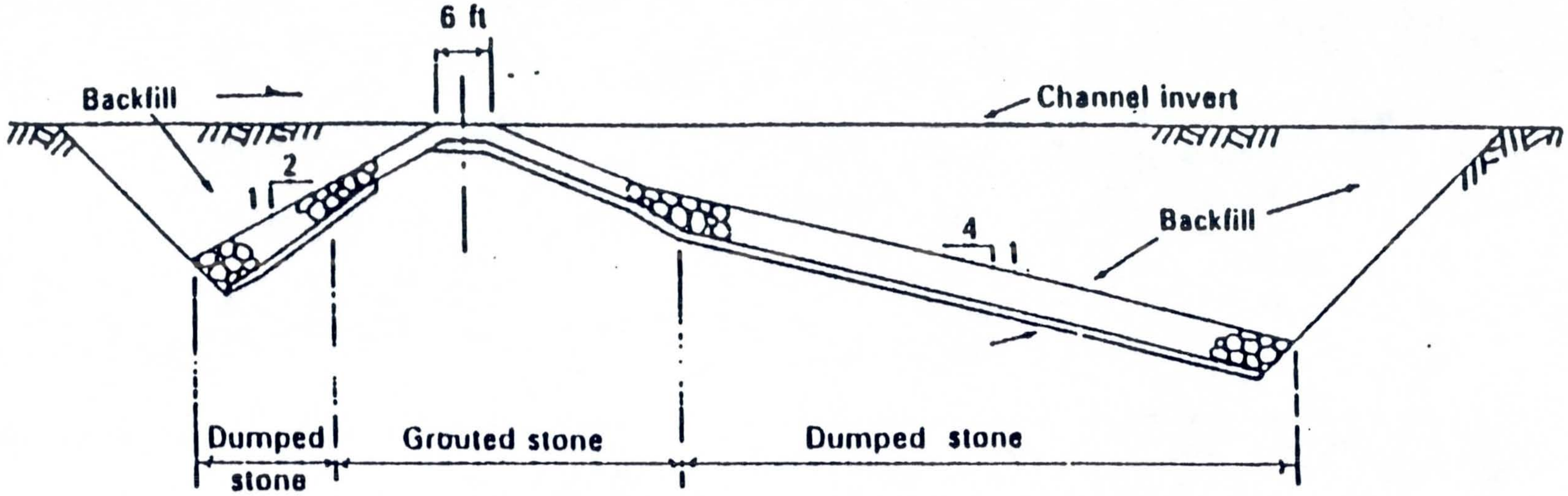
c. Pier Protection Measures.

(1) Bridges, multiple-barrel culverts, and other structures frequently have supporting piers located within flood channels.

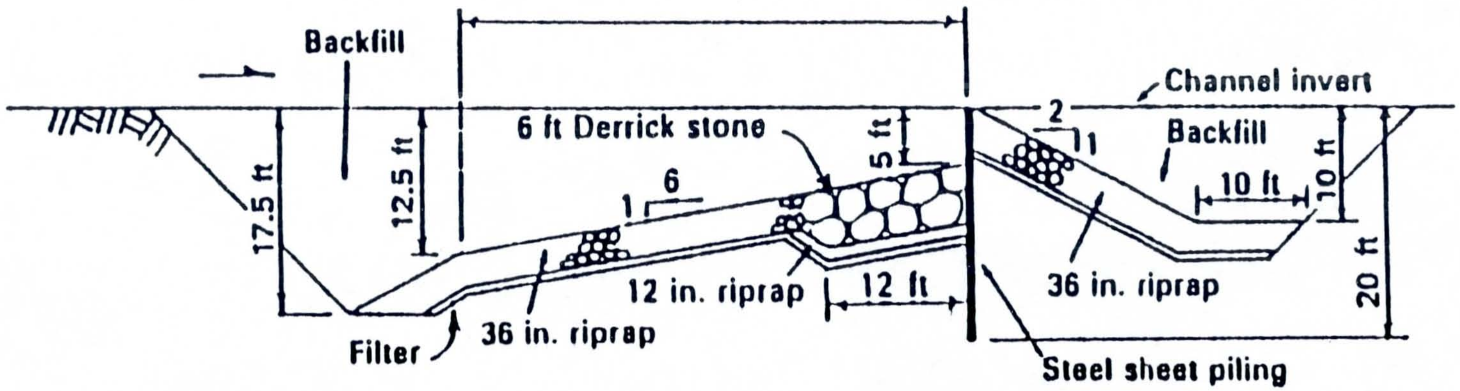
(2) Hydraulically, piers within channels are undesirable because they reduce channel capacity, collect trash and induce bed scour.

(3) If multiple pile piers are required, the gaps between them are often connected by a baffle wall to reduce the trash build-up.

(4) Bride abutments can also scour and wing or spur dikes are used to help flow transition. Also, if scour occurs, it usually occurs at the tips of the dikes, which is away from the bridge structure itself.



(a) Grouted stone stabilizer
(after U.S. Army, Corps of Engineers, 1970)



(b) Sheet piling stablizer (after Linder, 1963)

Figure 7–54 Typical channel stabilizers.

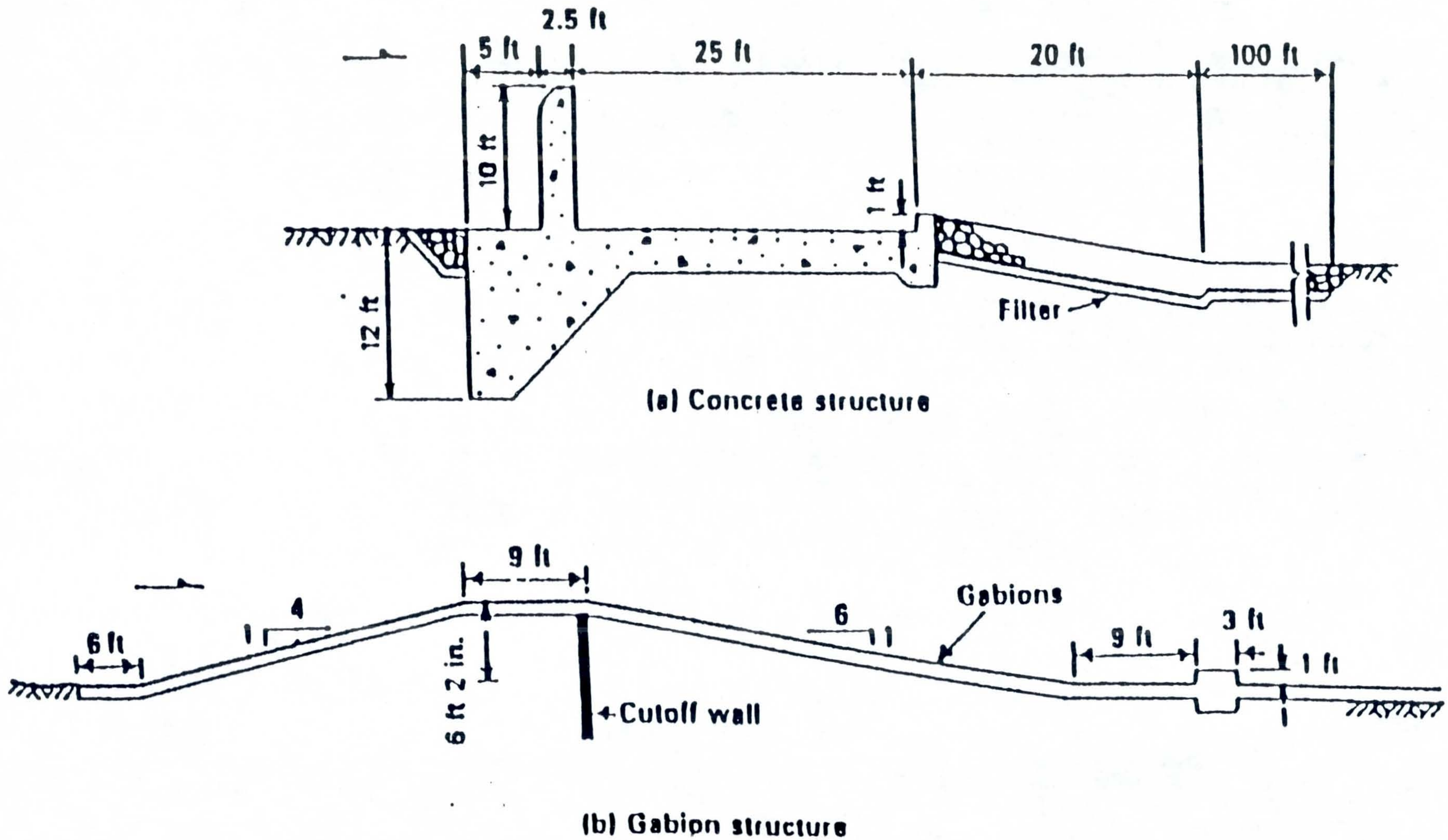


Figure 7–56 Souris River channel control structures. (After Saunders and Grace, 1981.)

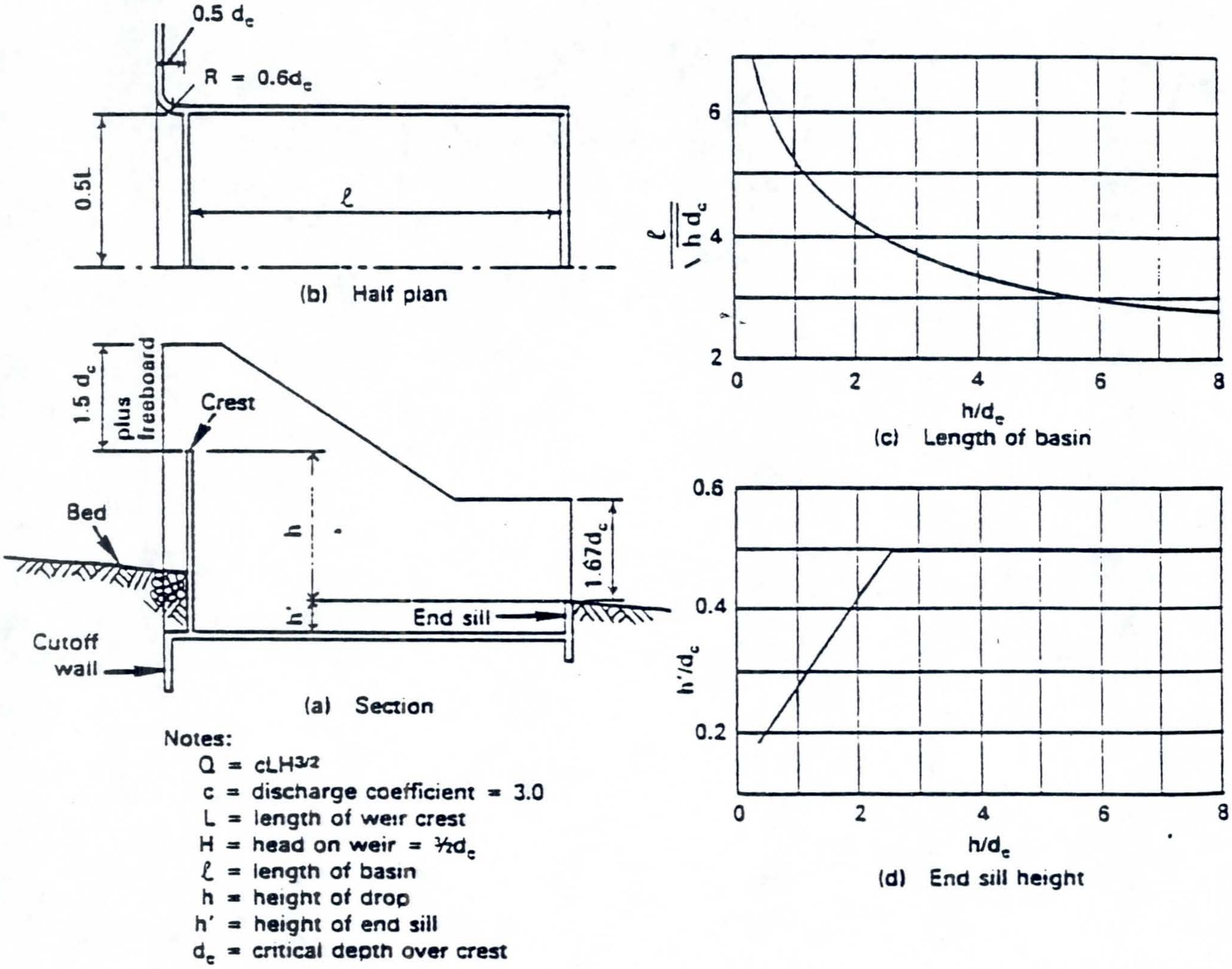


Figure 7–58 Typical drop structure. (After U.S. Army, Corps of Engineers, 1970.)

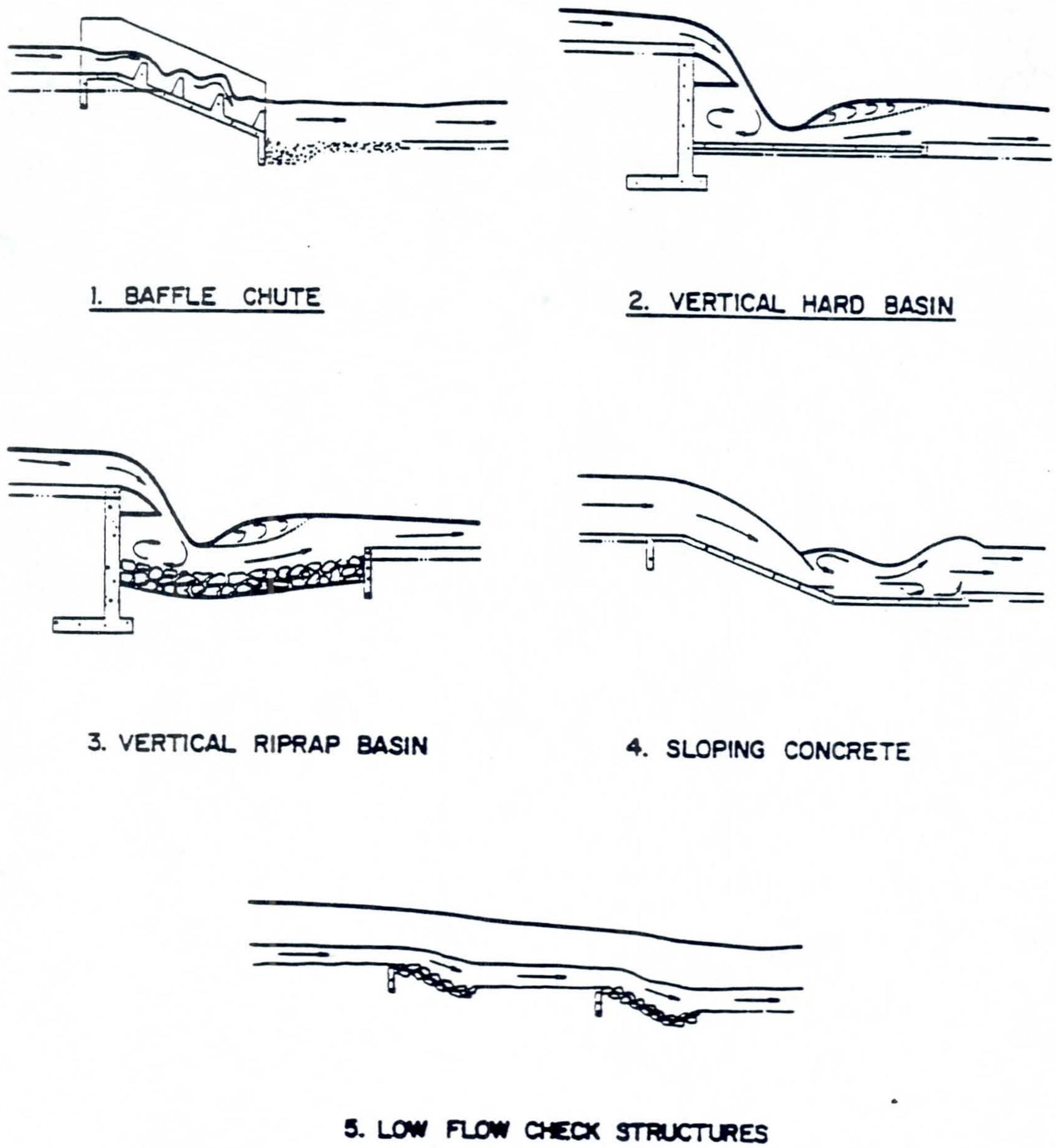
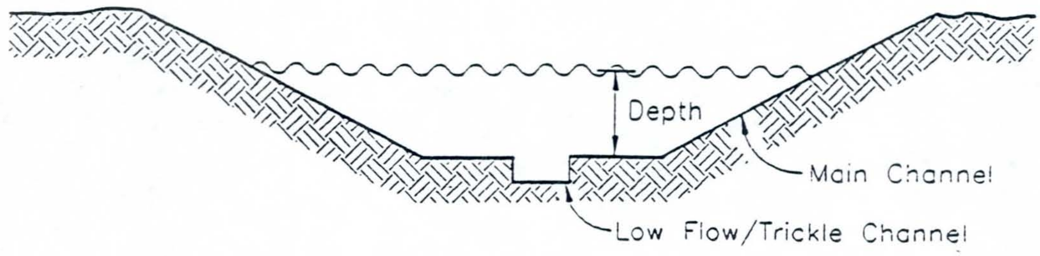
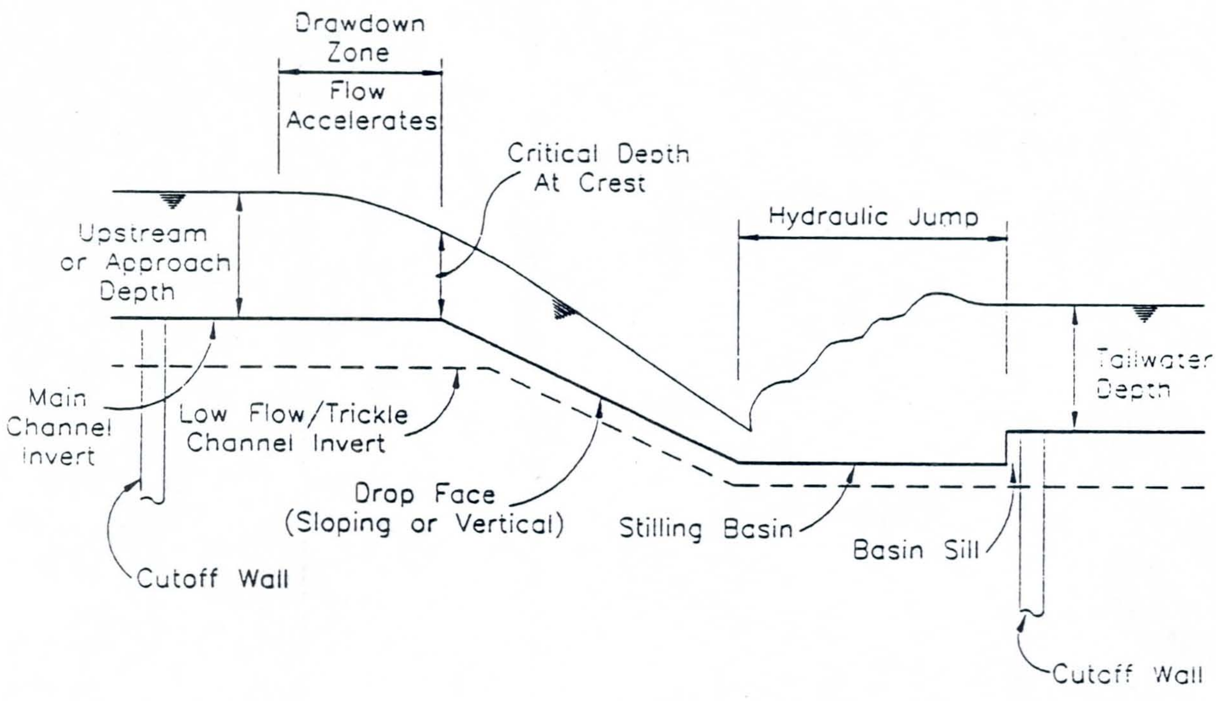


Figure 7.2
Drop Structure Types
(McLaughlin Water Engineers, Ltd. 1986)



CHANNEL SECTION



CHANNEL PROFILE

Figure 7.1
Typical Drop Structure Components
(Adapted from McLaughlin Water Engineers, Ltd. 1986)

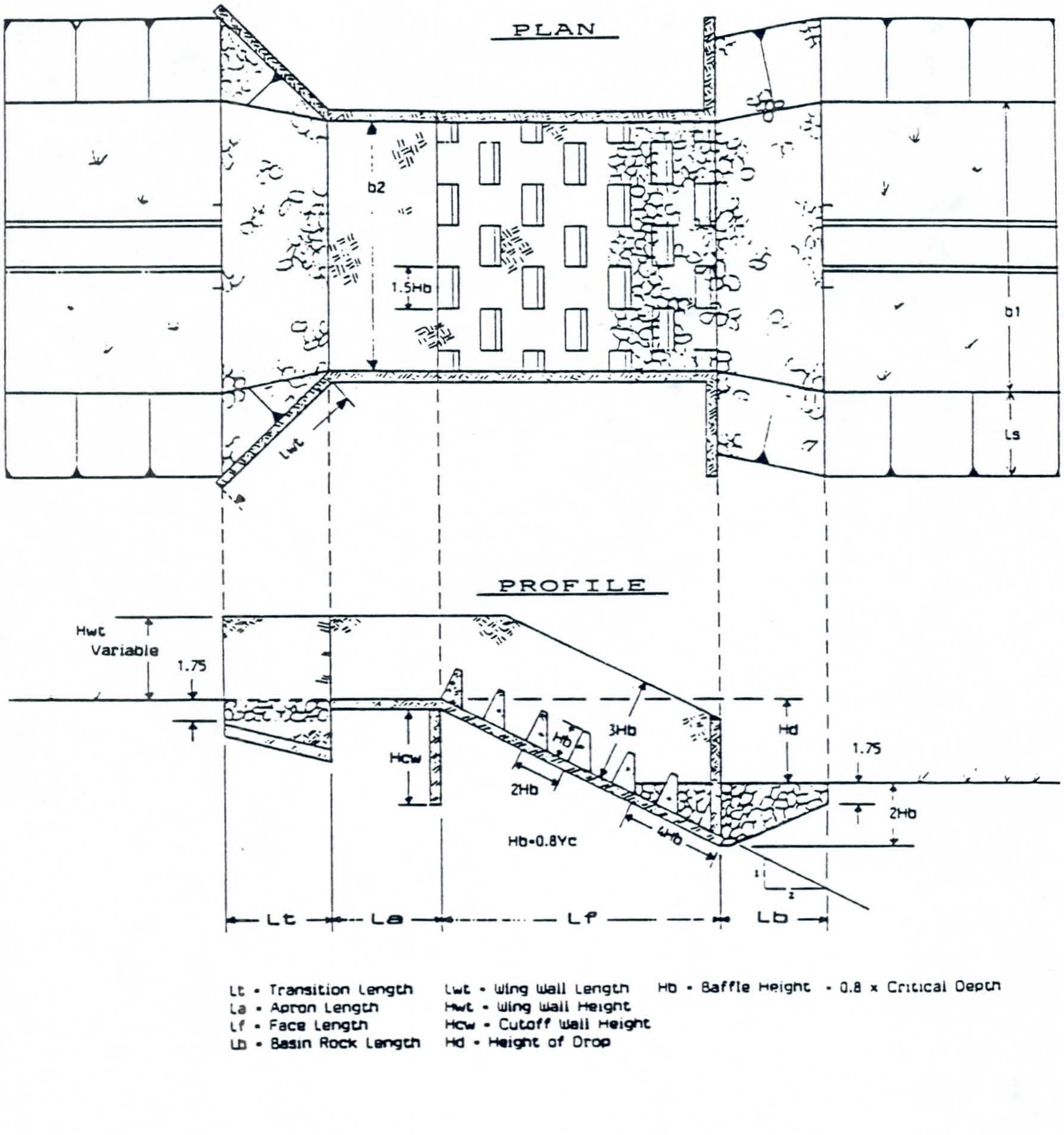


Figure 7.11
Baffle Chute Drop
(McLaughlin Water Engineers, Ltd., 1986)

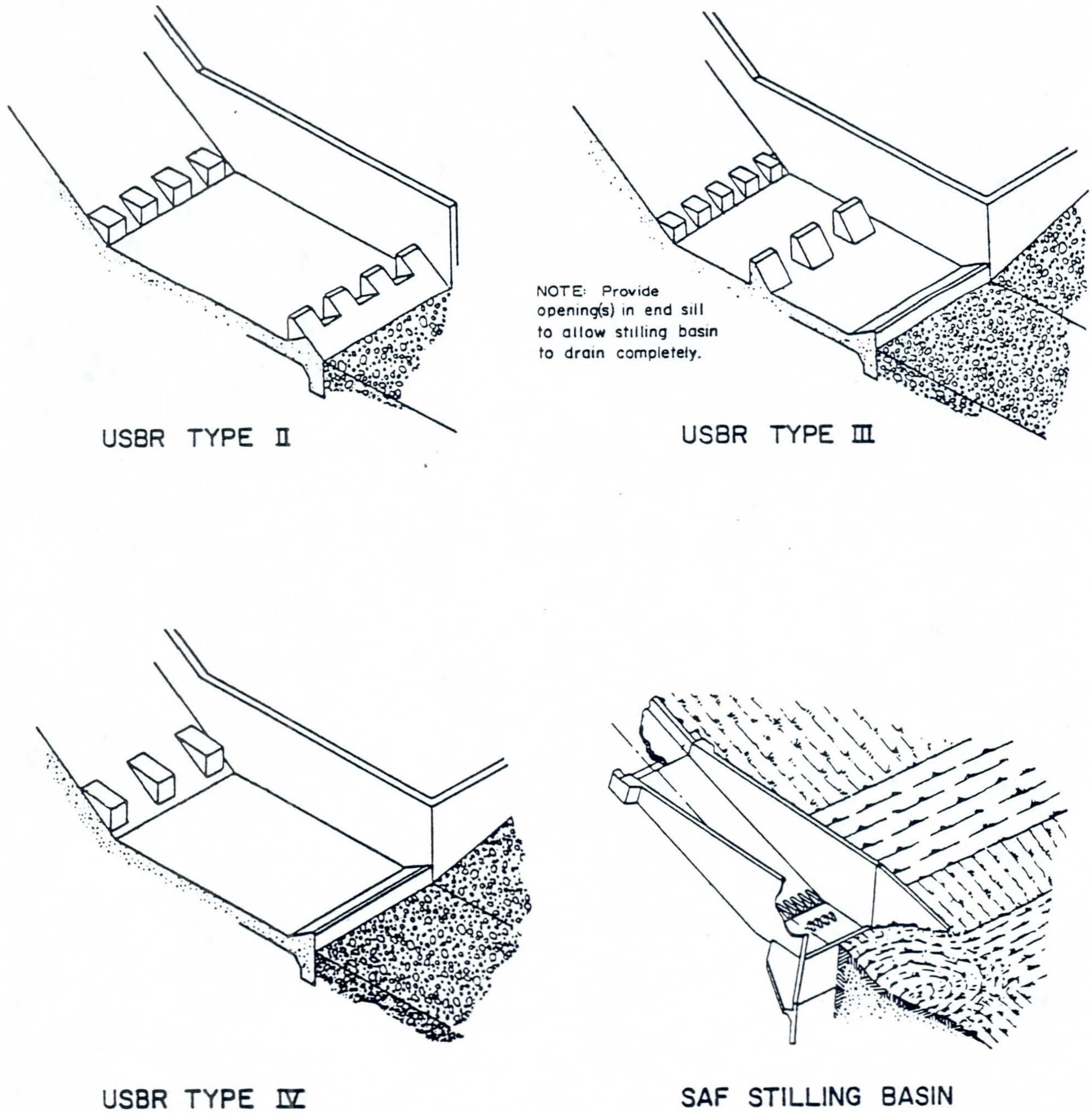
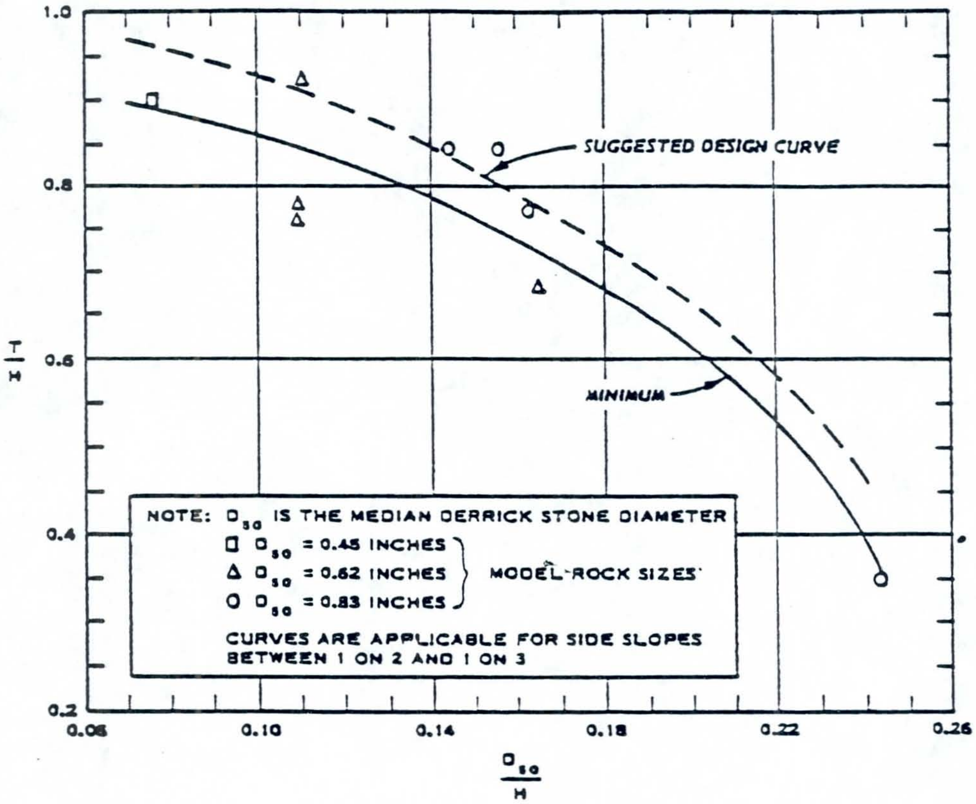
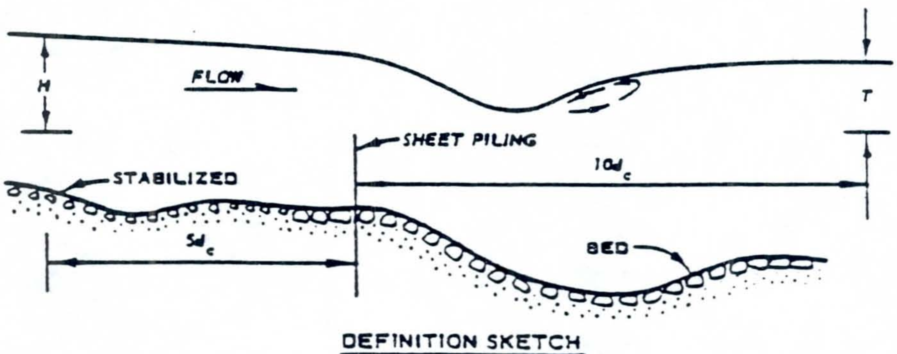


Figure 7.18
Stilling Basins for Sloping Concrete Drops
(Adapted from: FHWA, HEC-14, 1983)





DEFINITION SKETCH

GRAPH LIMITED TO

$\frac{T}{d_c} \geq 0.8$

d_c = CRITICAL DEPTH

REPRODUCED FROM FIG. 17, REF 60.

SHEET PILING STABILIZER

DERRICK STONE SIZE

SEE TEXT PAGE 48

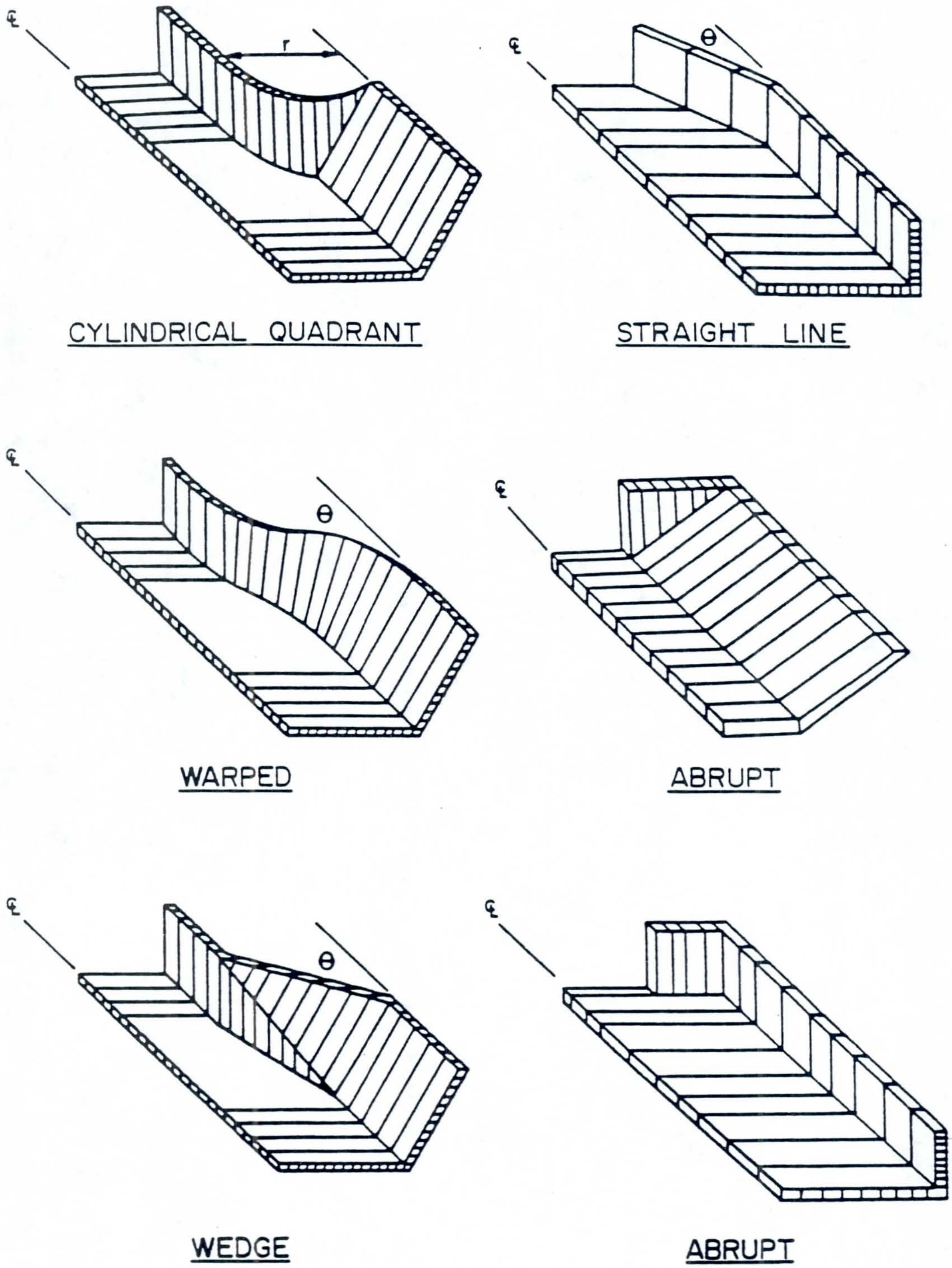


Figure 7.30
Channel Transition Types
(Adapted from: FHWA, HEC-14, 1983)

CHANNEL MODIFICATIONS

AND POTENTIAL RESPONSE

Lecture 7

Channel Modifications and Potential Response

Adapted for Flood Control District short course by D. Williams, Ref. 8

by John J. Ingram

1. General.

Measures of flood control include clearing and snagging, channel excavation, channel realignment, levees, and high flow channels (Council on Environmental Quality Report on Channel Modifications, 1973).

Use of these methods may be singular or in combination. These methods and their potential responses are briefly described in the following paragraphs.

a. Clearing and Snagging.

In some instances, clearing and snagging may be all that is necessary to provide the desired level of flood protection.

Through this method, flow conveyance can be improved by selective removal of log jams, large trees, rocks, sediment blockages, and other debris.

The removal of debris may be beneficial to flood control for two reasons:

(1) the debris will increase flow resistance and generate turbulence which will lead to higher flow depths and increased bank erosion, and

(2) the debris will cause additional debris carried by flood waters to hang up and further impede the flow of water.

The potential responses of local, upstream, and downstream reaches are identified in Table 1.

Table 1

Potential Response of Local, Upstream, and Downstream Reaches Due to a Clearing and Snagging Project

Local Effects	Upstream Effects	Downstream Effects
1. Greater inbank carrying capacity	1. Possible headcutting	1. Deposition
2. Velocity may increase or decrease	2. Increased velocity	2. Possible bed and bank instability
3. Change in sediment load	3. Increase bed material transport	3. Loss of channel capacity
4. Decreased stage	4. Possible river form change	
5. Bank instability		
6. Thalweg meandering		
7. Channel slope change		

b. Channel Excavation.

The purpose of channel excavation is to increase conveyance such that flood flows remain within the channel. This procedure usually entails establishing a new width, depth, and side slope for the channel.

Post construction concerns for channel excavations are: channel bed aggradation and degradation, channel bank stability, and the tendency of the thalweg and channel to meander.

Potential responses of local, upstream and downstream reaches for channel excavations are presented in Table 2.

Table 2

Potential Responses of Local, Upstream, and Downstream Reaches Due to a Channel Excavation Project

Local Effects	Upstream Effects	Downstream Effects
1. Greater inbank carrying capacity	1. Increased velocity	1. Deposition
2. Lower velocities at average discharge	2. Increased bed material transport	2. Possible bed and bank instability
3. Decreased sediment load at average discharge	3. Headcutting	3. Increased flood stage
4. Decreased stage	4. Bank instability	4. Loss of channel capacity
5. Bank instability	5. Possible change of river form	
6. Thalweg meandering tendencies		
7. Channel degradation		

c. Channel Realignment.

Many times a channel is straightened when the natural stream is widely meandering. The purpose of straightening is to reduce bend resistance, increase energy slope, and increase flow velocity which reduces flow depth.

However, any time a channel's geometric configuration is disturbed, the potential exists for upstream, local, and downstream response.

Potential responses to channel straightening are presented on Table 3.

Table 3

Potential Response of Local, Upstream, and Downstream Reaches Due to a Channel Straightening Project
(from Simons and Sen̆turk, 1977)

Local Effects	Upstream Effects	Downstream Effects
1. Steeper slope	1. See local	1. Deposition
2. Higher velocity		2. Increased flood stage
3. Increased sediment transport		3. Loss of channel capacity
4. Degradation and possible headcutting		
5. Banks unstable		
6. Tendency for thalweg and channel to meander		
7. River may braid		
8. Degradation in tributary		

d. <u>High Flow Channels</u>

High flow channels allow the natural channel to remain untouched while providing flood protection to the adjacent land. This protection is provided by excavating a bypass channel across a channel meander or by the construction of berms.

The bypass channels are designed to convey only high discharge events. Ackers (1972) discusses the benefit of bypass channels as follows:

<u>Increase in Channel Flood Capacity</u>.

The engineer planning to modify a section of river to improve its flood capacity should remember that the present width and gradient of the river, if it is indeed stable, are the regime values. Consequently, any attempt to make a significant alteration to the cross-section may be thwarted by a redistribution of sediment.

It is preferable, therefore, to retain the regime width and slope, up to the level of the dominant discharge, and to provide the increased flood capacity by berms, and flood banks perhaps, only come into effect at discharges with a frequency less than the dominant condition.

If channel regrading and/or realignment is planned, remember that a mobile bed river will not take kindly to disturbance to its regime, and special works may be necessary to curb the readjustment process, for example bank protection and a check weir at the entrance to the lowered reach.

When designing a channel for flood control as Ackers describes, modifications made for larger than dominant discharge events (that is, discharge events greater than the constant discharge that would form the channel regime) still need to be evaluated for stability of the excavated high flow channels.

For example, when a bypass channel is designed, the use of an outlet control structure for the bypass may be necessary for the prevention of headcutting in the bypass channel during high discharge events.

Responses that are to be evaluated for high flow channels are shown in Tables 4 and 5.

Table 4

<u>Potential Responses of Local, Upstream and Downstream Reaches During Flood Events Due to a Bypass Channel</u>

Local Effects	Upstream Effects	Downstream Effects
1. At flood flow, overbank discharge is carried by the bypass channel	1. Increased velocity	1. Channel bed scour due to combining of flows and the power relation between sediment transport and discharge.
2. Suspended sediment transport from upstream will be divided between the main channel and the bypass channel. Due to the power relation between sediment transport and discharge, some of this sediment may be deposited in either the main channel or the bypass channel	2. Increased bed material	2. Increased flood stage due to increased passage of flood flows
3. Possible headcutting in the lower reaches of the bypass channel due to an increased gradient of flow within the bypass channel.	3. Possible headcutting	3. Possible bed and bank instability
4. Bank instability	4. Bank instability	
5. Possible change of river form.	5. Possible change of river form	
6. During dominant discharge	6. During dominate discharge events and less, the channel responds in its natural state.	

Table 5

Potential Responses of Local, Upstream, and Downstream Reaches During Flood Events Due to the Excavation of Berms

Local Effects	Upstream Effects	Downstream Effects
1. Greater inbank carrying capacity	1. Increased velocity	1. Increased flood stage
2. Possible instability of the berms	2. Increased bed material	2. Sediment deposition
	3. Headcutting	3. Possible bed and bank instability
	4. Bank instability	
	5. Possible channel form change.	

e. Levees.

Levees provide flood protection without modification to the stream. However, levees may modify the stream.

When levees are not infringing on the natural channel alignment, the response of the system for high and low discharges still need to be evaluated.

Potential responses which should be evaluated for a levee system are identified in Table 6.

Table 6

Potential Responses of Local, Upstream, and Downstream Reaches Due to the Construction of Levees

Local Effects	Upstream Effects	Downstream Effects
1. Normal response at average flows unless the levee system infringes upon the stream's natural alignment. For this latter case, see local effects, Table 3.	1. Normal response at average flows unless the levee system infringes upon the stream's natural alignment Under this latter case, see Upstream effects, Table 3.	1. Increased flood stage
2. Increased flood stage	2. Velocity variations at flood flow.	
3. Higher velocities at flood flow.	3. Unstable bed and banks during high discharges.	
4. Unstable bed during flood flow.		
5. Possible levee instability due to secondary flow during both average and flood flow.		

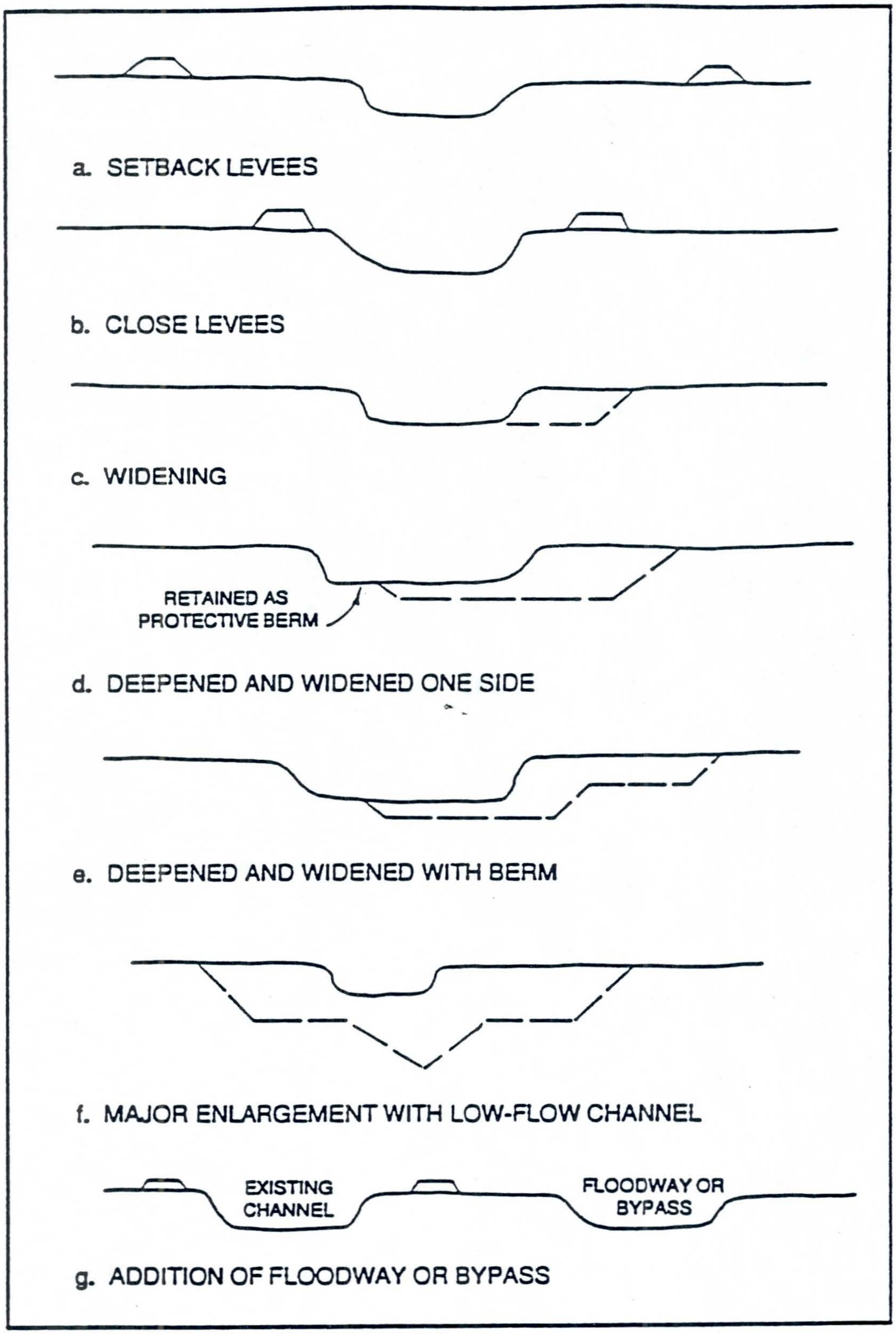


Figure 6-17. Various types of modified cross sections

Quantitative Response to Alluvial Channels to River Works (From Ref. 10)

Variable		Change in Magnitude of Variable	Regime of Flow	River Form	Resistance to Flow	Energy Slope	Stability of Channel	Area	Stage
Discharge	(a)	+	+	M→B	±	-	-	+	+
	(b)	-	-	B→M	-	+	+	-	-
Bed-Material	(a)	+	-	M→B	+	+	±	+	+
Size	(b)	-	+	B→M	-	-	±	-	-
Bed-Material Load	(a)	+	+	B→M	-	-	+	-	-
	(b)	-	-	M→B	+	+	-	+	+
Wash Load	(a)	+	+		-	-	±	-	-
	(b)	-	-		+	+	±	+	+
Viscosity	(a)	+	+		-	-	±	-	-
	(b)	-	-		+	+	±	+	+
Seepage Force	(a)	Outlfow	-	B→M	+	-	+	+	+
	(b)	Inflow	+	M→B	-	+	-	-	-
Vegetation	(a)	+	-	B→M	+	-	+	+	+
	(b)	-	+	M→B	-	+	-	-	-
Wind	(a)	D/S	+	M→B	-	+	-	-	-
	(b)	U/S	-	B→M	+	-	-	+	+

M = Meandering B = Braided

Potential Stability Problems from Flood Control Modifications (from Ref. 7.)

	Potential Stability Problems		
Forms of Channel Modification	Within Reach Directly Affected	Upstream	Downstream
Clearing and snagging	Bank erosion and bed scour	Headcutting	Sedimentation
Cleanout or enlargement	Bank erosion; sedimentation	Headcutting	--
Realignment	Bank erosion and bed scour; meandering	Headcutting	Sedimentation
Levees	Meander encroachment on channel	--	Increased flood peaks
Floodways and bypasses	Sedimentaion of original channel	--	--
Diversions out	--	--	Sedimentation
Diversions in bed	--	--	Bank erosion and scour
Base level lowering (parent stream)	--	Bed scour, widening, tributary degradation	--
Storage reservoir or sediment basin	--	Delta formation; aggradation	Bed degradation

PROCESS OF BANK FAILURE

Lecture 8

PROCESS OF BANK FAILURE

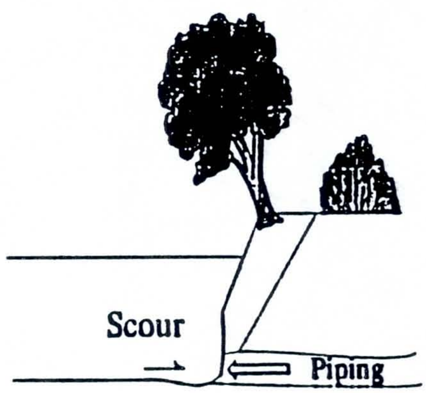
Joseph Haggerty, Ph.D.

Adapted for Flood Control short course by D. Williams, from Ref. 9.

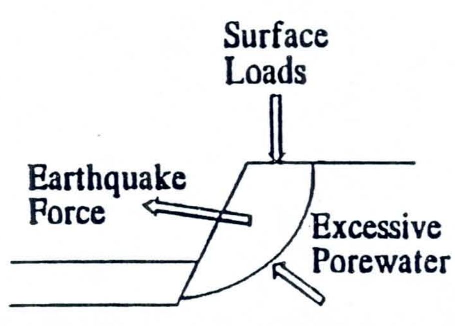
A. Mass Movements and Flows

1. Mass movement of streambanks is caused by two major mechanisms: Erosion-Induced and Load-Induced as illustrated in the following.

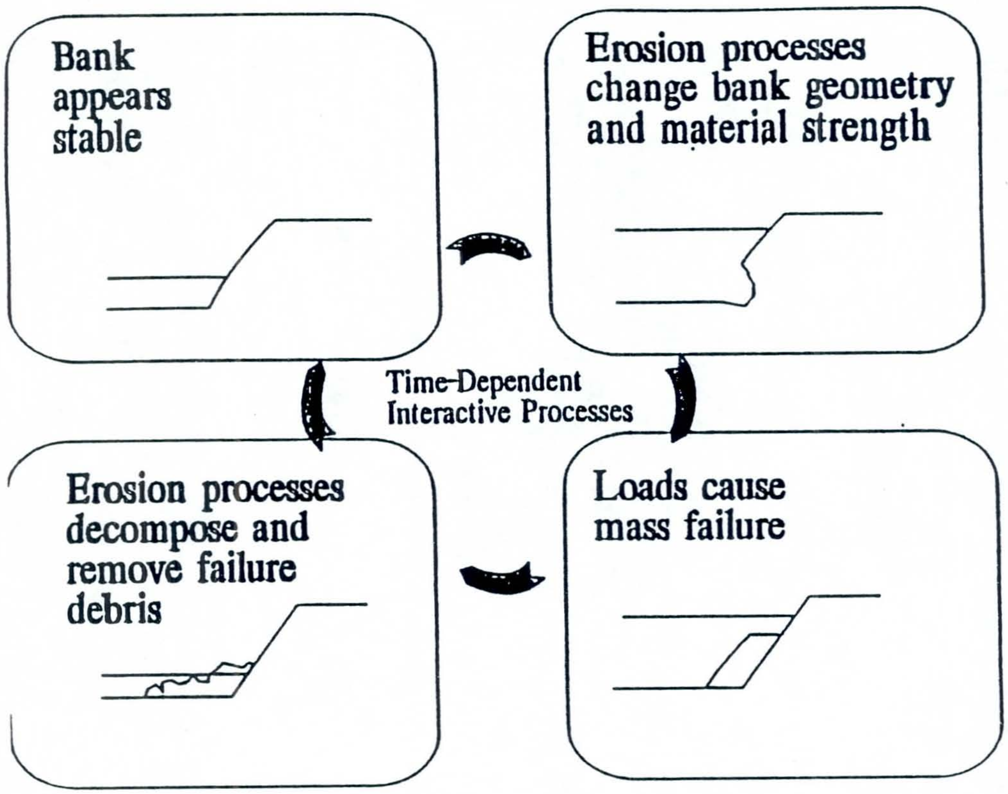
Erosion-Induced Causes



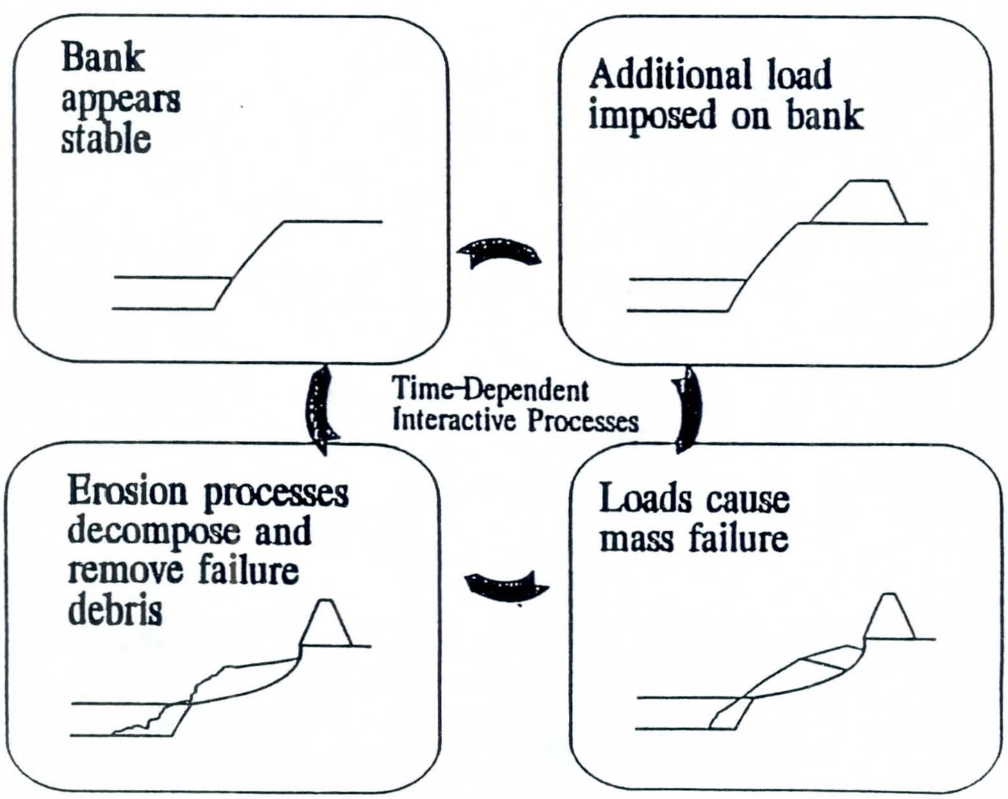
Load-Induced Causes



Erosion-Induced Mass Failure Sequence



Load-Induced Mass Failure Sequence

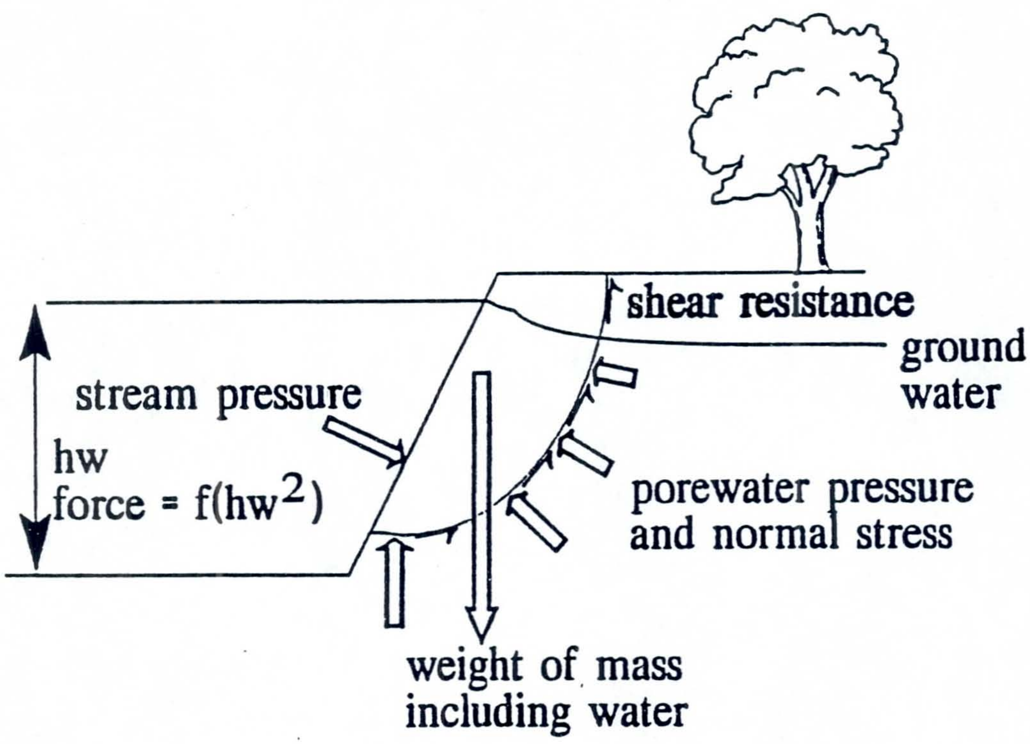


2. Mass Stability Factors

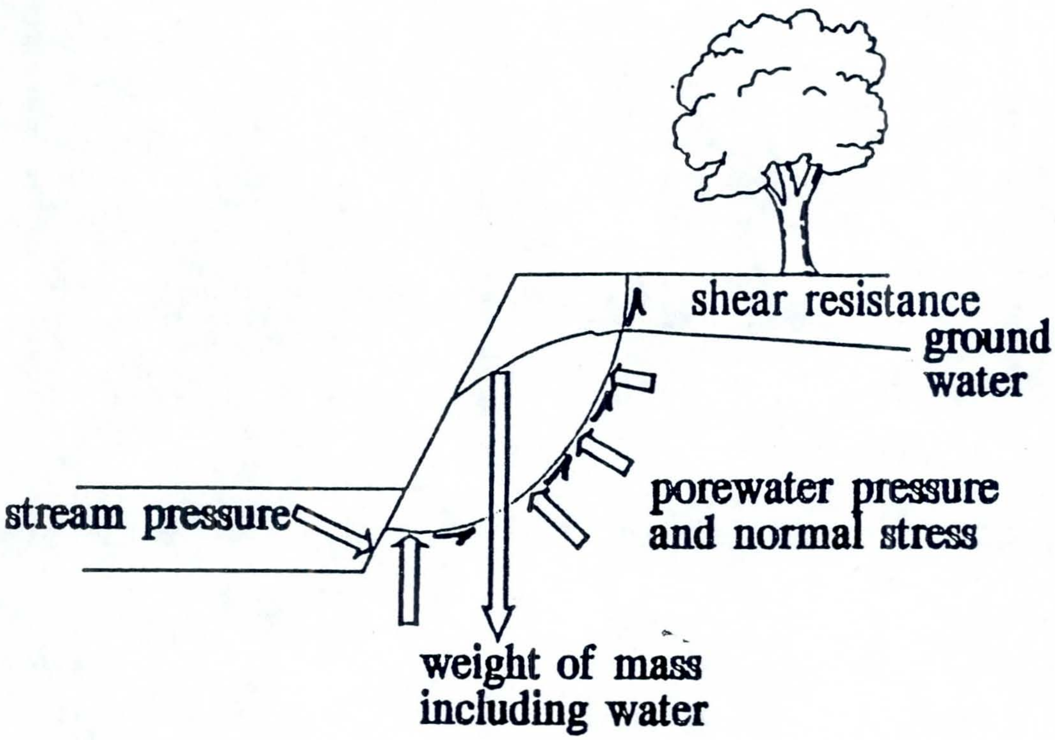
The main mass stability factors are: Forces, Material Strength, and Bank Geometry.

Mass Failure occurs when the stress in the bank material exceeds the material strength.

Forces on Mass of Streambank During High Water



Forces on Mass of Streambank After High Water



B. Water-Related Forces

The following are forces on a streambank with + indicating forces that help stabilize the streambank and - indicating forces that destabilizes the streambank.

+ capillary suction

+ water flow into bank retains bank

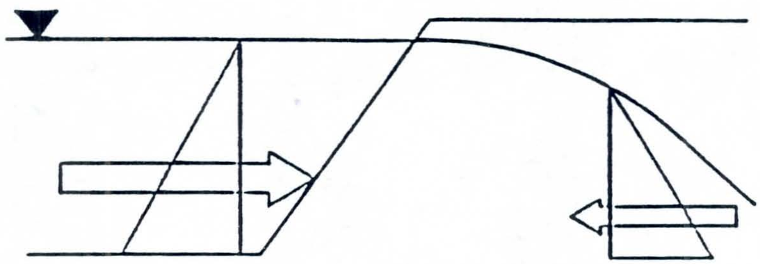
- water flow out of bank tends to destabilize bank

- increased porewater reduces effective stress

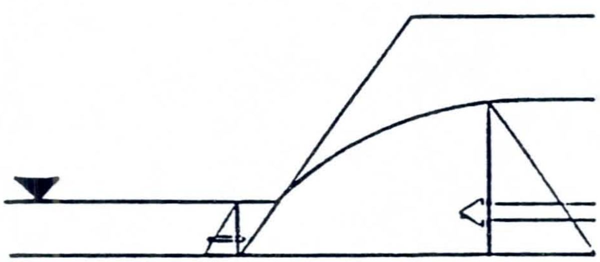
- water increases unit weight of mass

- porewater pressure tends to spread discontinuities

Forces of Stream on Bank

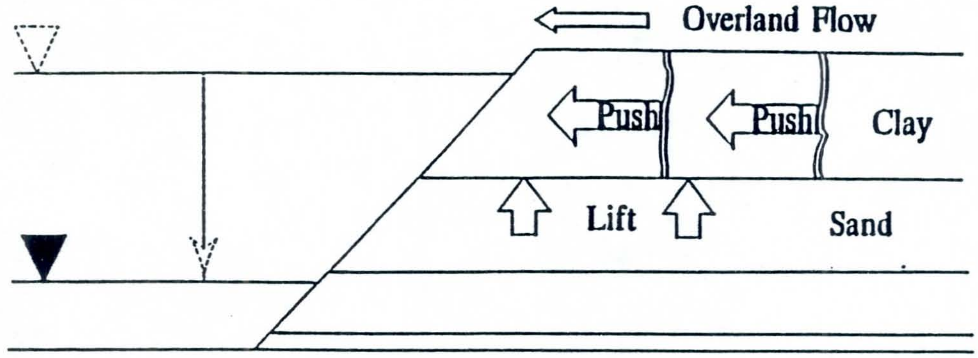


Water Retains Bank During
Increases in Stream Stage

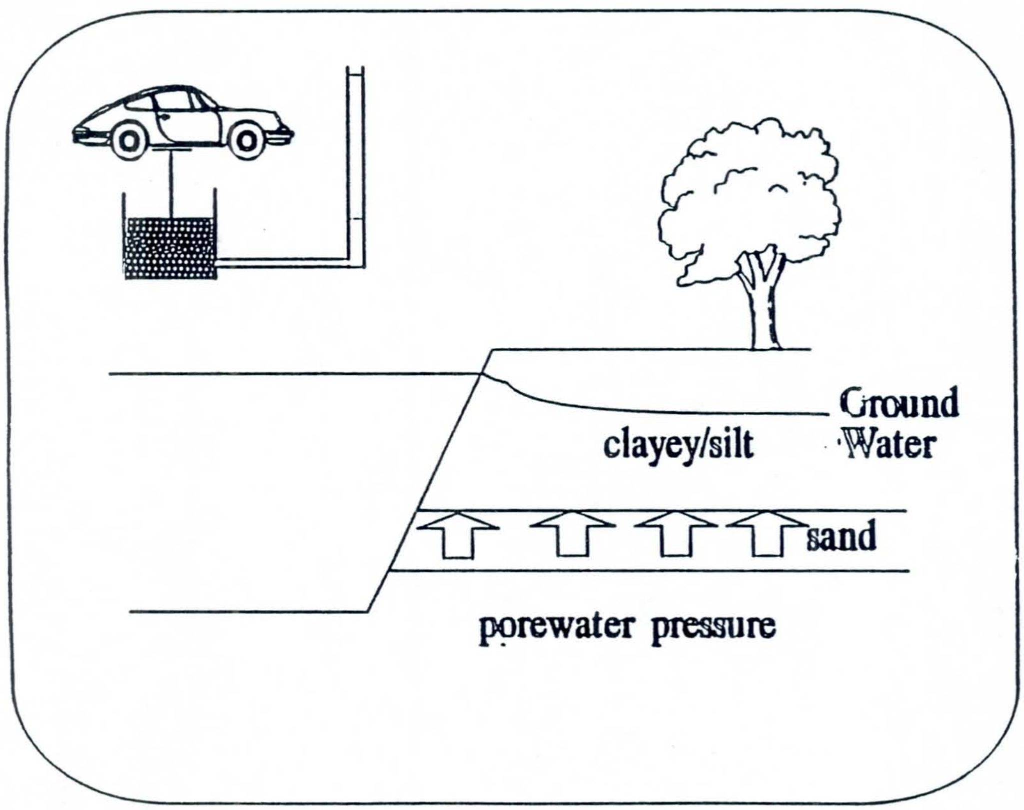


Water Tends to Push Bank Over
During Decreases in Stream Stage

Water Pressures in Cracks



Porewater Pressure



C. Material Strength - Resistant to Forces

The material strengths can be divided into two types.

1. Material Characteristics

    - Cohesive: clay mineral, silt (clay minerals)

    - Cohesion: silt, sand, gravel, cobble

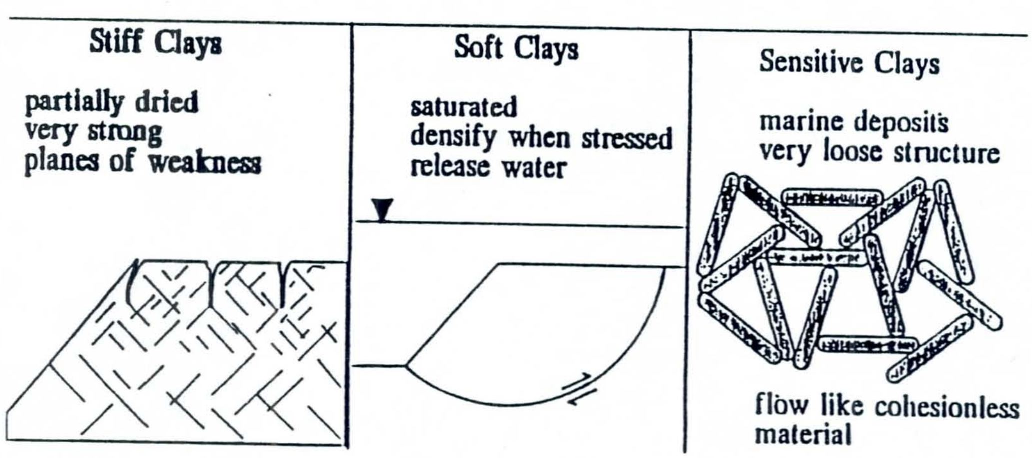
2. Mass Structure

    -stratificationn and scale

    - discontinuities

    - vegetation

Cohesive Materials



D. Vegetation

+ Provides Tensile Reinforcement

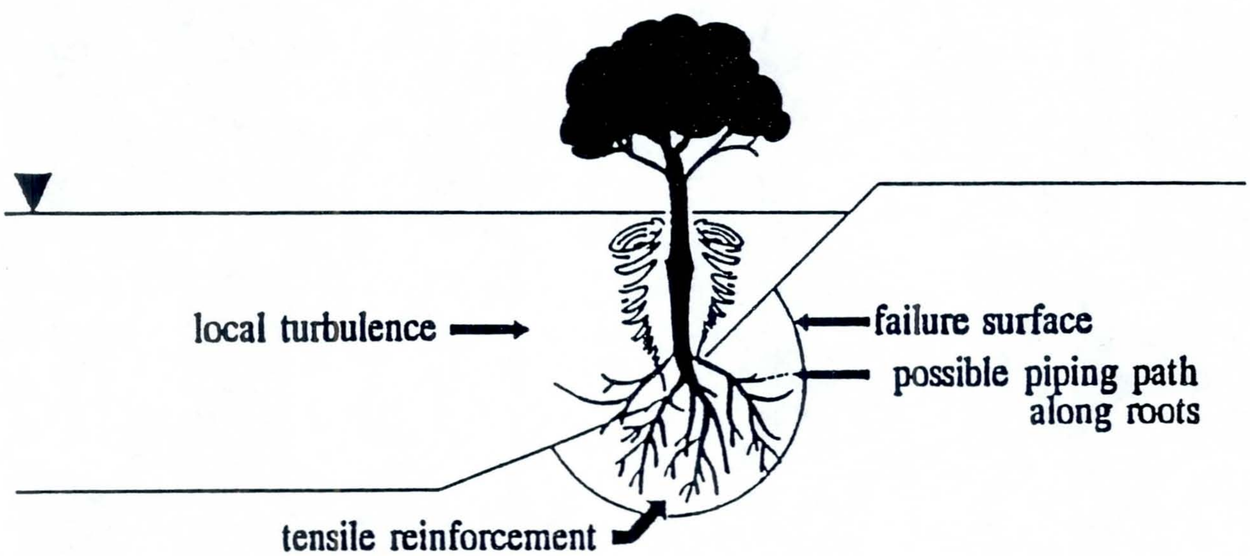
+ Buffers Waves

+ Reduces Local Boundary Stress

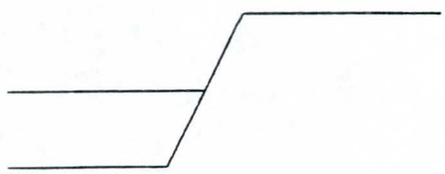
- Disrupts Integrity of Mass and Bank Geometry

- Causes Local Turbulence

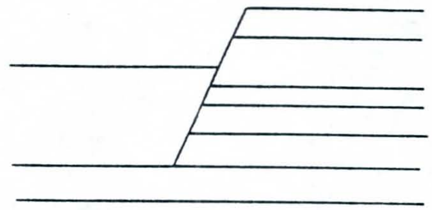
- Toppled Trees Disrupt Flow



Mass Structure: Stratification and Scale



Homogeneous banks

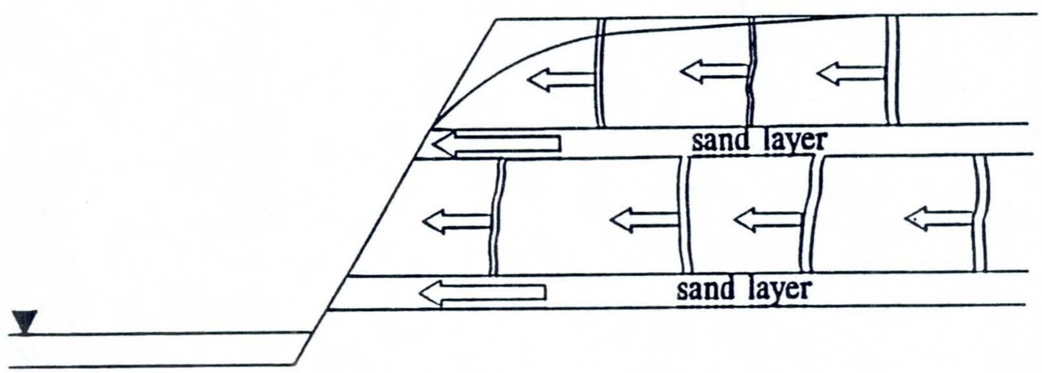


Layered bank

Important because of:

- variance in strength
- variance in hydraulic conductivity

Discontinuities and Layers



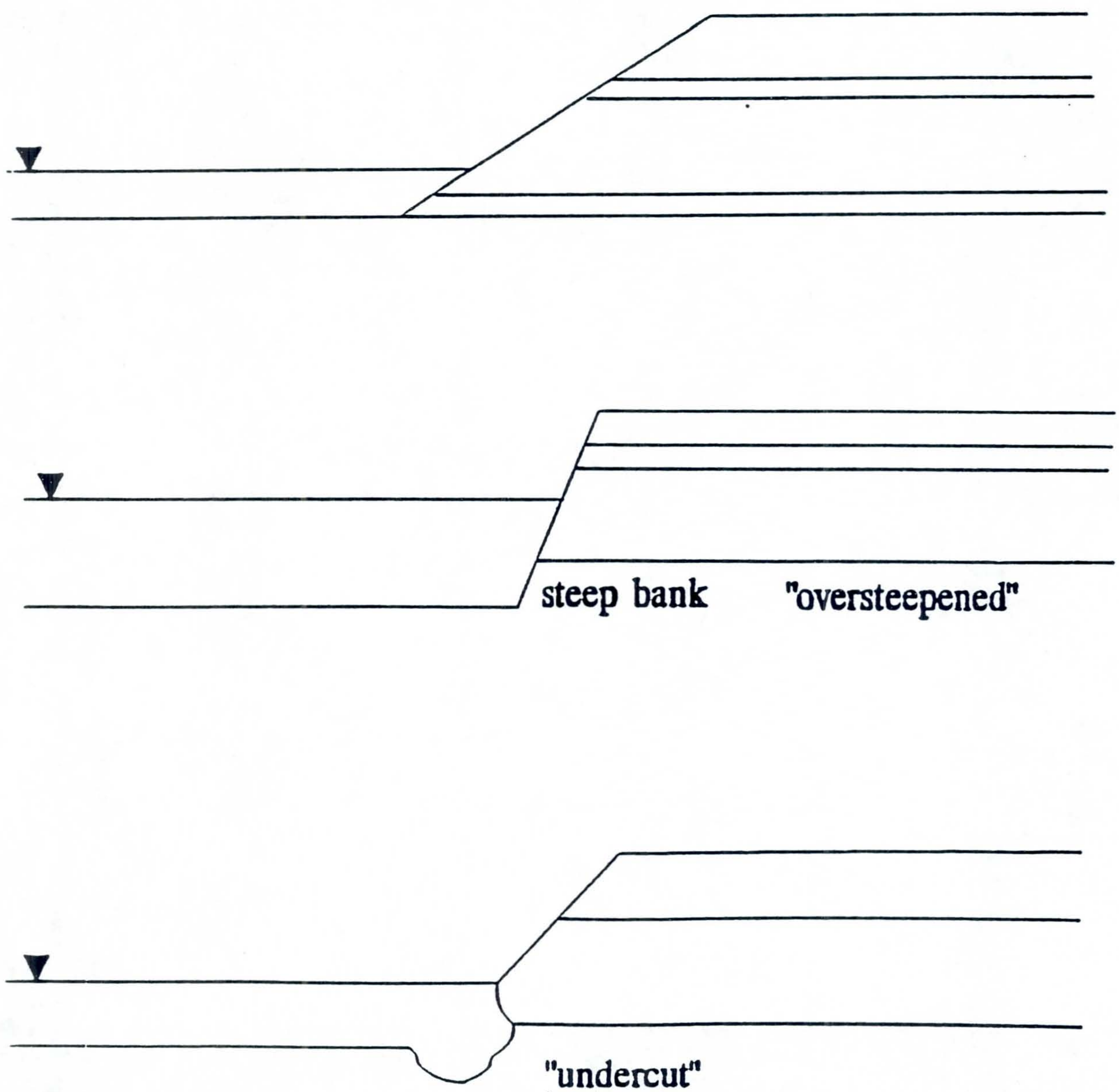
Important because of:

- flowpaths of high conductivity
- planes of weakness
- high porewater pressures
- allows flow through impervious layers

E. Geometry of Bank

The present geometry is a product of past processes and material characteristics. For future conditions, you must first evaluate present conditions, assume processes such as erosion and mass movement. Also look at bank conditions both upstream and downstream.

Geometry of Bank



Earth Movement Classification
(D.J. Varnes)

<table>
<tr><td colspan="3" rowspan="3">TYPE OF MOVEMENT</td><td colspan="3">TYPE OF MATERIAL</td></tr>
<tr><td rowspan="2">BEDROCK</td><td colspan="2">ENGINEERING SOILS</td></tr>
<tr><td>Predominantly coarse</td><td>Predominantly fine</td></tr>
<tr><td colspan="3">FALLS</td><td>Rock fall</td><td>Debris fall</td><td>Earth fall</td></tr>
<tr><td colspan="3">TOPPLES</td><td>Rock topple</td><td>Debris topple</td><td>Earth topple</td></tr>
<tr><td rowspan="2">Slides</td><td>Rotational</td><td>Few Units</td><td>Rock slump</td><td>Debris slump</td><td>Earth slump</td></tr>
<tr><td>Trans-
lational</td><td>Many Units</td><td>Rock block slide
Rock slide</td><td>Debris block slide
Debris slide</td><td>Earth block slide
Earth slide</td></tr>
<tr><td colspan="3">LATERAL SPREADS</td><td>Rock spread</td><td>Debris spread</td><td>Earth spread</td></tr>
<tr><td colspan="3">FLOWS</td><td>Rock flow
(deep creep)</td><td colspan="2">Debris flow Earth flow
(soil creep)</td></tr>
<tr><td colspan="3">COMPLEX</td><td colspan="3">Combination of two or more principal types of movement</td></tr>
</table>

STREAMBANK PROTECTION
AND DESIGN METHODS

Lecture 9

GENERAL DESIGN CONSIDERATIONS

Ref. 9

This Section discusses design considerations which are applicable to all methods of bank stabilization work, and will help in understanding the guidance in Section V.E. on selecting a method. Information given here will also assist in preparation of the preliminary estimates of cost required in the selection process.

The great variation in site conditions and project constraints, and the infinite possible variations in materials and design details of the methods themselves, make a "cookbook" approach impractical. As Simons and Li (1982) state , ". . . handbook-type analyses and designs [for river training and bank stabilization] usually lead to poor solutions of specific problems." Hemphill and Bramley (1989) agree by stating ". . . good design practice necessarily involves **judgement and experience**, and [we] can only draw attention to the various aspects which need to be taken into consideration or on which **expert advice** should be sought" (emphasis added).

This Section can assist the engineer's **judgement** and can provide "surrogate" **experience. Expert advice** on site-specific problems and additional information on specialized topics can be obtained from the points-of-contact and references which are cited at the appropriate point in this section, and elsewhere in the student notebook.

These factors apply to the design of all bank stabilization work:

- Geomorphology
- Hydraulics
- Geotechnical
- Environmental
- Toe Protection
- Surface Drainage
- Manufacturer's Recommendations
- Safety Factor

This list can be used as a "checklist" to insure that your design has not overlooked any major factors. For some stabilization methods, one or more factors can be quickly dismissed. For example, geotechnical analysis is not usually required for "indirect" protection methods, and manufacturer's recommendations apply only to commercial products.

1. GEOMORPHOLOGY

The first design decision is the location of the work. We can use "applied geomorphology" to answer 3 basic questions:

> Where do we begin the work?
> Where do we end the work?
> What alignment do we follow from beginning to end?

1.1 Beginning and Ending Points

The key element in making this determination is **predicting channel migration.** The basic parameters of channel migration, or meandering, are shown on Figure 1. Note that regardless of whether the stream is sinuous or straight, a definite pattern and spacing of bars, pools, and crossings exists. One characteristic of this pattern will be invaluable to us in siting stabilization work:

> Movement of bars, pools, and crossings is both perpendicular to the axis of the meander belt and downvalley, with the greatest movement usually downvalley.

Unfortunately, variation in bed and bank material distorts the classic pattern of movement to some degree, as shown on Figures 2 and 3. Therefore, we must obtain some verification of migration trends for a specific location.

There are 4 potential tools to use for this verification, listed in approximate descending order of reliability:

> Hydrographic surveys, topographic surveys, or scaled aerial photography over time.
>
> > With the position of the stream channel documented at two or more points in time, the length of bank that is subject to erosion can be identified.
>
> Interpretation of existing planform.
>
> > With only one point in time (the present), the principle of down and outward movement, along with experience derived from similar streams, can be used to predict where erosion will occur if the bank is not stabilized.

Interviews with locals.

While local people may not be scientists, and may not be completely unbiased in their reports, they can provide useful information.

Numerical or physical modeling.

Numerical modeling of meandering is a developing tool that shows promise, but unfortunately, reliable prediction of future migration requires that the model be verified using past migration trends as documented by one or more of the first 3 tools listed above, therefore to some extent you must have the answer to get the answer. The same is true of physical modeling, with the additional disadvantages of long testing time and high cost. Numerical and physical models are much more useful for studying hydraulics, changes in the bed, and "generic" meandering than for predicting long-term channel migration for a specific location. The fatal flaws are that meandering is a partially random process, that bed and bank material are seldom uniform, and that it is impossible at present to reliably model the erosion of cohesive material.

Geomorphic experience is required to apply any of these tools. There are no cookbook solutions. If you don't feel comfortable with your own experience, help is available either from your own contacts or from the points-of-contact and references listed elsewhere. In particular, River Meandering (1984) documents field experience and research on channel migration. No one is omnipotent, but an experienced river scientist can narrow the range of possibilities significantly. River scientists seldom consider being asked for advice an imposition. Even if they are "experts," they also learn from every contact.

The **minimum requirement** for length of bank to be stabilized will be one of the following, in ascending order of required length:

The streambank immediately adjacent to a threatened structure.

The length of streambank which is caving actively enough that providing only localized protection would not guarantee adequate protection for the required project life.

For comprehensive projects which require fixing a great length of stream in a stable position for navigation, flood control, irrigation, or other long-term project purposes, the minimum requirement is to stabilize all the bank which must remain in its present position or some other pre-determined position so that navigation channel alignment, flood control works, irrigation structures, or other project features are not threatened.

Figures 4 and 5 illustrate these situations.

An important and less-obvious corollary to the rule for minimum length is:

Even for the most limited bank stabilization need, future migration of the stream channel must be forecast. Even if the minimum length of bank to be stabilized is short, or the required life of the work is short, long-term channel migration still must be forecast. Then one can determine the length of protection required to meet the specific need.

Therefore, even if the minimum requirement is "spot" stabilization, your analysis should determine if problems will result from migration, and if so, provide a conceptual potential solution even if it cannot be implemented immediately.

Application of the preceding discussion to your selection of beginning and ending points should consider the following concepts:

Downstream is more critical than upstream, since scour pools tend to move downvalley, and bank failure usually occurs in scour pools. Although beginning the upstream end of work at the precise point where erosion is presently occurring carries some risk of erosion upstream of that point, cost savings may make the risk acceptable, because a bar will often move down and change the erosion at that point into deposition. In contrast, placing the downstream end of the work at the precise point where erosion stops carries a high degree of risk that the work will later be flanked by erosion.

Model tests under the Section 32 Demonstration Erosion Control Program (U.S. Army Corps of Engineers, 1981) indicated that protection should be extended downstream a distance of at least 1.5 times the approach channel width. This can be used for general guidance if data on channel behavior at a specific location is not reliable. Studies by Parsons (1960) provide similar insight.

The transition into the existing bank can be made rather simply at the upstream end. The details will depend on the type of protection being used, but in general a slight increase in the strength and/or a shallow "key-in" will be sufficient for armor-type protection. Indirect protection can simply be turned or "feathered" into the bank, with a slight recess being good insurance.

At the downstream end, more elaborate precautions are advisable for the transition to the existing bank. The most important precaution is to insure that the work is not stopped prematurely, as discussed above. Beyond that, it is advisable to key in and/or increase the strength of armor revetment, and to provide a pronounced "tuck-in" at the end of indirect protection, perhaps with a liberal application of stone if conditions are severe. The alternative is to be prepared to return to the site, perhaps years later, to armor the scour pocket that will likely form, which may be a sound approach if project authority, rate of erosion, and potential consequences of miscalculation allow it.

Your analysis of present conditions and prediction of future migration should take into account the magnitude of flows that occurred prior to your inspection, or between the points in time that surveys or aerial photos were made. High flows tend to attack further downstream, and low flows farther upstream, because meander wave length is directly proportional to discharge. The stream integrates the total hydrograph over the long-term, but short term observations may be distorted by abnormal flows. In other words, if the period of observation is weighted toward low flows, the long-term attack may be farther downstream and more severe than casual observation would indicate. High flows during the period of observation may have the opposite effect.

On a **braided stream**, stabilization of the bank a considerable distance upstream and downstream from the active erosion may be necessary. Bars and pools frequently move downvalley more rapidly than in a meandering stream. The most efficient approach on a braided stream with low-flow erosion at the upstream end from flow crossing the riffle and impinging on the bank at a sharp angle may be one of the following:

"Fight fires" by constructing spot stabilization as the stream attacks first one spot, then another.

Temporarily divert impinging flows, either by excavating, building temporary dikes of streambed material, or using floating "breakwaters" to absorb the brunt of the impinging flow. Be aware that construction activity in the streambed may not be environmentally acceptable.

If long-term stabilization is required, and project constraints prevent intermittent construction as the channel changes, it may be necessary to construct continuous protection on both sides of the "meander belt" of the braided stream.

1.2 Channel Alignment Considerations

The **preferred alignment** in most cases is to accept the existing general channel alignment. Significantly changing the alignment makes it more difficult to predict the ultimate equilibrium planform and channel geometry. This carries risk not only for the success of the work, but also for assessing the potential for detrimental effects caused by the work.

Relocating a bank to be armored or vegetated would require costly and time-consuming excavating and filling. Environmental effects of moving large amounts of material may be unacceptable. The work would be highly vulnerable to damage from high flows during construction, causing contractual difficulties as well as engineering difficulties. Banks constructed totally of fill material would be more susceptible to settlement and scour even after being armored, unless expensive compaction and subgrade filters are used. Also, vegetation is unlikely to be adequate protection for banks newly constructed of fill material.

Indirect protection methods can more easily be used to modify the existing alignment, but the same basic principle applies - the stream has integrated all the pertinent variables and has developed a corresponding alignment, so be cautious about changing it.

Now three **exceptions** to the preferred choice:

(A) In a sharp bend, it may be desirable to flatten the radius by using indirect protection, or by making a cutoff.

(B) If the existing bank alignment is highly irregular, with protruding points, it may be desirable to "smooth" the alignment by placing the erosion protection landward of those points and allowing the stream to erode them away. The same objective can be achieved by placing indirect protection in front of the bank as in (A).

(C) In a straight reach, or on a braided stream, it may be desirable to realign for more sinuosity to stabilize the location of scour pools and bars. This can also provide better channel alignment and a deeper channel for navigation.

These exceptions, especially (C), are more likely to be attractive on projects with navigation aspects than on projects with only bank stability aspects. The shortest acceptable radius will be dictated by the navigation design criteria, as presented in U.S. Army Corps of Engineers EM 1110-2-1611, "Layout and Design of Shallow Draft Waterways."

Every stream has an envelope of stable values of sinuosity, or in straight reaches, pool and bar spacing. Research has also shown that the ratio of bend radius to channel width has an envelope within which the channel is more stable. Any channel realignment should conform to these envelopes. This will be a particular challenge if the stream has recently aggraded or degraded, and the planform is still adjusting, especially if the threshold between meandering and braided is crossed.

Accomplishing **exception (A)** by flattening the radius by constructing indirect protection in front of the existing bank has several advantages:

It reduces disturbance to the existing bank in the bend.

It moves the deepest scour away from the bank.

It eliminates protrusions, thus satisfying exception (B) as well.

The flatter radius may reduce the maximum depth of scour.

Disadvantages are that this approach places more of the work in the deepest part of the existing channel, thus increasing the cost and difficulty of construction, and that it might be necessary to excavate the opposite point bar to relieve the initial constriction caused by the work. Otherwise, local velocities, and even backwater effects, may be unacceptably high before the channel adapts to the work.

Constructing a cutoff across the neck of the bend will certainly eliminate erosion in the bend. However, erosion will continue elsewhere after the cutoff. Therefore even a cutoff may need to be accompanied by bank stabilization, and the placement and design of stabilization, and predicting its long-term effects, is more uncertain in the presence of a cutoff.

Exception (B), removing anomalies in alignment by placing the stabilization work on a more uniform alignment behind the existing bank, can be accomplished by constructing stone "trenchfill" or "windrow" revetment. This approach allows the channel to develop more uniform velocities and cross-section by removing local constrictions of streamlines. It is most useful on navigation projects where alignment and radius of the "sailing line" is critical. It offers simple design and construction, being removed from the active channel. Unfortunately, the erosion which occurs until the stabilization line is reached may not be greeted enthusiastically by property owners, and if erosion is slow, a navigation project with a schedule to meet may require dredging of the uneroded foreshore.

Exception (C), major realignment of straight or braided reaches, should not be attempted without a confident knowledge of the stream's behavior, especially the stable envelopes of pool and bar spacing and radius/width.

2. HYDRAULICS

Now that we've used "applied geomorphology" to site bank stabilization work, our next step, stated rather crudely, is to use "applied hydraulics" to decide how deep, how high, and how strong to make the work. You had some discussion of hydrologic and hydraulic principles on Tuesday. Here we'll look at the following seven factors:

Design discharge
Variations in discharge and stage
Tractive force
Secondary currents
Prediction of toe scour
Top elevation of protection
Waves and vessel forces

2.1 Design Discharge

First, an important distinction - the term "design discharge" usually refers to an extreme event, usually greater than the greatest event of record, and is used most often in connection with flood control channel analyses. "Design discharge" is also used to compute stone size for riprap armor, and a similar approach can be used to design many commercially available armor materials. "Design discharge" can be defined rather precisely using hydrologic analyses.

In contrast, the discharge which governs the geomorphology of a stream, thus the siting of bank stabilization work, is exceeded rather frequently, but there is no consensus on what to call it, much less how to define it. It is variously called the "dominant" or "channel-forming" or "effective" discharge, but is in fact a fictitious value because no single steady discharge will produce the same channel that is formed by the varying discharges of natural streams, although it is considered by many to be about equal to bankfull discharge on streams that are neither aggrading or degrading.

Stated another way - "design discharge" is the flow which stresses bank stabilization work most severely over a short period of time. It is desirable to quantify it and to use it to size the armor layer if we are using an armor technique for which criteria exist. For most other protection methods, determination of a "design discharge" will be academic because no criteria exists to apply it. In contrast, quantification of the "dominant discharge" is always unnecessary, because the geomorphic approach discussed earlier allows the stream to integrate the actual flows into its geomorphology. Even if one did select a dominant discharge, using it to determine meander characteristics and siting of stabilization work would be unreliable.

This brings up a useful concept for evaluating the performance of existing work. That is that a flow greater than the existing record will probably be experienced eventually, which may stress the work to the point of failure, although the general stream characteristics remain unchanged.

Aside from the semantic exercise, this tedious discussion is intended to reduce confusion in communications about discharge-related concepts, and to define what we can and cannot do in applying quantitative analyses of discharge to the design of bank stabilization work.

Figure 6 portrays this concept less painfully than the text does.

In summary - use **design** discharge if you are using a stabilization method for which critical "strength" values have been determined, usually in terms of tractive force. Otherwise, it is irrelevant. Use **dominant** discharge by letting the stream integrate it into its geomorphology, which you in turn will integrate into siting of the stabilization work.

2.2 Variation in Discharge and Stage

Although this is, strictly speaking, a hydraulic variable, its primary application to design is geotechnical, since susceptibility of the bank to mass failure, leaching, and piping, is to some degree a function of the rate of fall of the water level (Figure 7). The magnitude and timing of the variation also influences the "constructability" of different techniques.

2.3 Tractive Force

This is a widely accepted measure of stress on channel boundaries, be they dirt or structure (Figure 8). A similar expression is "stream power." Tractive force is intrinsically related to design discharge, and we will mercifully only summarize, not repeat, that tedious discussion here: Except for riprap and some manufactured products, little precise guidance exists for the limiting tractive force. For other methods, demonstrated performance under comparable conditions is the best guide.

2.4 Secondary Currents

It is generally accepted that there is a component of velocity near the bed which is oriented away from the outside of a bend, thus transporting bed material away from the toe of the outside bank and contributing to bank failure. Figure 9 shows a simplistic view of this process. Although the general concept is accepted, the precise form of the currents and how they change with channel alignment and varying flows are complex and not well-defined.

This process is important to the theory of meandering, but more to the point here, it is basic to the mechanism of bank failure and to the effectiveness of many types of bank and channel stabilization work. It has significant influence on bed material transport at the toe of the bank, and upon the magnitude of toe scour. Unfortunately, we can usually address secondary currents in design only empirically, once again letting the stream integrate it into its behavior, along with all the other geomorphic, hydraulic, and geotechnical processes. However, some techniques, such as "Iowa Vanes" and "bendway weirs, specifically rely on modification of secondary flow for their effectiveness.

2.5 Prediction of Toe Scour

We stated earlier that we are discussing the design of bank stabilization under the assumption that system-wide instability, such as bed degradation, does not exist, or will be corrected by system-wide stabilization measures, such as bed stabilization. Similarly, the various methods for predicting toe scour deal with local scour as a separate process from system-wide bed degradation.

The general approaches to predicting toe scour are:

Analytical, using one or more of the relationships that have been proposed by various researchers

Empirical, using experience from similar conditions

Modeling (numerical and/or physical)

Some **analytical** approaches are presented in U.S. Army Corps of Engineers EM 1110-2-1601. Illustrating the uncertainty of analytical approaches, U.S. Army (1981, Appendix B) cites seven different equations proposed by as many researchers for the specialized case of predicting scour at spur dikes. There is disagreement even as to the significant factors involved, and certainly disagreement in the results.

An **empirical** approach is often the most reliable, but adequately documented experience to apply to a particular situation may not be available.

The limitations to **modeling** that we discussed in channel migration also apply to prediction of toe scour, although not to as great a degree. Two-dimensional numerical models are required, and three-dimensional models would be preferable, if only they were available, and physical models must be large scale.

The degree to which you apply these techniques will always be a matter of judgement, and will depend on time and funds available for your analysis, the importance of the project and the consequences of failure, and upon the personal and institutional experience you can bring to bear on a specific problem.

Precision of prediction with any approach is greater for armor revetments than for indirect protection methods, and the precision decreases with increasing change in channel alignment, cross section, and boundary roughness produced by the work.

Prediction of scour in long contracted reaches and around structures is discussed by E. M. Laursen in Appendix A of River Engineering (by Margaret S. Petersen, 1986).

2.6 Top Elevation of Protection

The most conservative approach for armor revetments is to set the top elevation at design flowline plus a freeboard. This equates to top of levee for leveed channels, or top of riverbank where levees don't exist or are set back well away from the bank or protected by vegetation. For many situations, this criteria is too conservative and would result in excessive costs. Unless erosive velocities will exist at this high elevation, and the consequences of even minor erosion are unacceptable, you should consider a top elevation at a more frequently occurring flowline. Other factors that should be considered in order to decide on the lowest and least costly, yet effective, elevation are listed below, and shown on Figure 10.

Stage duration
Severity of overbank flow during floods
Erodibility of upper bank material
Type of protection
Bank slope
Consequences of failure

These factors also influence the top elevation of indirect protection, although the most conservative elevation for indirect protection is top of riverbank rather than design flood flowline.

By now you've realized, and may justifiably be becoming resentful, that there are many design factors for which precise criteria do not exist. And as you suspected, there is little quantitative guidance for applying the factors listed above, with the exception that if a primary purpose of the work is protection against wave action, fairly rigorous procedures have been developed to compute run-up, thus top elevation. This is usually not the critical condition for streambank protection, but the references in "Waves and Vessel Forces" below provide ample information if you do encounter a case where it is. Otherwise, merely considering the listed factors qualitatively will allow you to make a judicious decision.

Stage duration is important for two reasons:

It determines how long the upper bank will be subject to potentially erosive current.

It is a factor in determining the lowest elevation that vegetative growth will be effective, which defines the minimum top elevation for armor revetment when it is used in combination with planted or volunteer vegetation.

As an example, but not presented as universal guidance, riprap protection on the Lower Mississippi River is routinely terminated at a flowline elevation exceeded approximately 5 to 10 per cent of the time. Unusual floods cause isolated damage above this elevation, but that appears to be influenced more by the severity of overbank flow, bank material, and overbank vegetation density than by the top elevation of protection.

Severity of overbank flow during floods is determined primarily by channel alignment upstream and downstream, as well as by local variations in the elevation of the bank, and the extent of vegetative cover on the upper bank and overbank. Local bankline irregularities which cause a convergence of streamlines, accompanied by higher velocities, also can play a role. The most severe conditions occur at the necks of sharp bends with relatively low bank elevations and little overbank vegetation. Also, the downstream half of sharp bends is where highest velocities against the bank and overbank usually exist.

Erodibility of the bank material is best determined by site observation. The rate of historic bankline recession is a clue, but is not a totally reliable indicator of erodibility of the material in the zone which you are considering for the top elevation. The material in the upper bank may be very erosion resistant, but still fail from toe scour and subsequent mass failure. General guidance on the erodibility of different bank materials is available, such as that given by EM 1110-2-1601, and can be used if experience with particular site conditions is lacking. The most erosive soils are fine sands and silty sands, and the least erosive are clay and coarse gravel, although alluvial stratigraphy is such that upper banks are not likely to be predominantly coarse gravel.

This is also an important factor for indirect protection. If the upper bank is highly erodible, then the structure should be high enough to reduce velocities well up the bank during most flows. If the upper bank is more erosion-resistant, then the structure needs to be only high enough to induce "berming" deposition in front of the bank, especially if the duration of high flows is short, the stream carries a large suspended sediment load, and/or if vegetation can be expected to colonize the induced berm.

Type of protection and the slope of the bank influence top elevation integrally by affecting the velocity distribution at the top of armor protection. The rougher the armor and the flatter the slope, the lower the velocity will be, and the lower the armor can be terminated, if other factors are equal.

Since rigid armor cannot adjust well to local scour at its top, it should be carried to a higher elevation than adjustable or flexible armor. The alternative is to use adjustable or flexible armor above the rigid armor to transition to a non-erosive zone, or to the elevation of dependable vegetation.

The type of protection also influences one's conservatism - be generous in choosing top elevation for an inexpensive method, be more precise when using an expensive method.

Flatter slopes are more conducive to vegetative growth, thus upper bank erosion in the form of "shelving" behind the armor will be more likely to be arrested by subsequent volunteer vegetative growth, if climate and soils are favorable.

Consequences of failure may well be the dominant factor in setting the top elevation of your protection work, especially if the other factors are not well-defined. This is an integral factor in determining the overall "safety factor" of the work, which is discussed later.

2.7 Waves and Vessel Forces

This topic crosses the boundary between "riverbank stabilization" and "coastal engineering." Even though these forces are not usually the dominant, or even a significant, cause of bank failure on inland streams, in some cases it can be a highly visible, and in the case of vessel-induced erosion, even controversial topic.

There are several references that address design of protective works in detail, and countless more specialized papers and publications. Two general references are by U.S. Army Corps of Engineers (1984) and Hemphill and Bramley (1989).

3. GEOTECHNICAL DESIGN

Geotechnical design considerations are:

Slope stabilization
Filters
Subsurface drainage

The last two are interrelated to a great degree, in that their purpose is to control the movement of water and bank material underneath and through the primary protection layer.

Geotechnical considerations are a vital part of armor bank protection, which requires a smooth subgrade and elimination of any instability of the bank material which would disrupt the armor layer, or at worst, fail the entire work (Figure 11). Geotechnical design is seldom required for indirect protection methods, because they accept the risk of minor bank erosion and failure until a stable state is reached naturally.

Other sessions discuss geotechnical analysis and provide references for further information on this highly specialized topic.

4. ENVIRONMENTAL CONSIDERATIONS

Environmental considerations may be grouped as follows:

- Consideration of people
  - Esthetics
  - Recreation, including safety
  - Cultural resources
- Consideration of critters

This topic has been addressed elsewhere in the course. We simply mention it here to validate it being an important factor in design.

5. TOE PROTECTION

Toe protection is essential to the success of stabilization work, although it may not be a massive element of the work if the exposure to scour is relatively mild. Once a prediction of toe scour has been made, as discussed earlier, you may choose from a variety of methods to protect against it.

5.1 Basic Approaches

The two alternatives are to:

"Dig it in" by extending the toe of the protective works into an excavation down to predicted scour depth, or down to non-erodible material, if you are fortunate enough to have such material within practical limits of excavation (Figure 12, methods A and B).

"Let it launch" by designing the work so that, as scour occurs, it can launch or flex downward sufficiently to prevent the scour from moving inshore and causing geotechnical instability of the bank (Figure 12, methods C and D).

The "dig it in" approach is self-explanatory, and is usually considered in connection with an armor approach. Its primary disadvantage is that excavation and precise placement of an armor material in the excavation in a streambed is often difficult and costly, and sometimes impossible. Ten feet is sometimes used as a rule of thumb for the limit of conventional excavation techniques underwater. Beyond that depth, either dredging or dewatering with a cofferdam may be required.

Sheet-pile retaining walls and pile-supported indirect protection structures designed to withstand maximum scour can be considered special cases of this approach.

The "self-launching" approach offers economy and ease of construction by letting the stream do the excavation, since the stream works for free. However, it does require a larger volume of material in the toe section than if the toe is placed in an excavation, since the launching process may be irregular. Therefore, if site conditions permit easy mechanical excavation to the predicted scour depth, the "dig it in" approach may be the least costly.

The self-launching technique also offers the considerable advantage of providing a built-in scour gage, particularly if the top of the launching section is visible above water. If it is underwater even at lower stages, it can be surveyed by accurately located soundings. If it appears that the toe section is launching more than expected, it can be reinforced simply by placing additional material at the riverward edge of the remaining section.

A combination of these two approaches in the form of "trenchfill" revetment can often provide maximum economy and effective performance.

With either approach, stone is often chosen for the toe protection material, even if another technique is selected for the remainder of the armor or structure, because stone can be precisely and confidently designed for almost any application.

5.2 Specific Guidance For Toe Protection

Application of the basic approaches to specific types of work is discussed below.

STONE ARMOR:

Refer to Figure 12.

OTHER SELF-ADJUSTING ARMOR:

Either dig it in, or use the self-launching technique with required toe volume computed the same way as for stone armor. If the self-launching approach is to be used where predicted scour is more than a few feet, stone is recommended for the toe material.

RIGID ARMOR:

Either dig it in, or use a self-launching technique with required toe volume computed the same way as for stone armor, or use flexible mattress at the toe.

FLEXIBLE MATTRESS:

Dig it in, or extend the mattress riverward of the toe of the bank a horizontal distance at least twice the predicted depth of scour, or follow the manufacturer's recommendations if appropriate. If more than a few feet of scour is predicted, consider the use of a self-launching stone toe, particularly if it will not be feasible to frequently monitor toe scour and reinforce the toe if required.

DIKES:

Toe protection for dikes is a more complex topic than for armor revetments. The complexity arises when trying to distinguish between:

(A) General toe scour which immediately endangers the overall stability of the bank and threatens to flank or fail the dike system.

(B) Localized scour which can threaten the integrity of part of a dike, and which ultimately may fail local portions of the bank.

(C) Localized scour which may cause minor damage to a dike or minor bank instability, but which can be accepted.

In practice, it is difficult to separate these three processes. Conceptually, however, case (A) must be prevented, and case (B) must be addressed if the consequences of it occurring are high. Acceptance of case (C) is inherent in the choice of dikes as the method of erosion control.

Conceptually, the alternative treatments, which can be used in combination, are:

Extend the dikes into the channel to move general scour far enough away from the bank to prevent major geotechnical instability.

Provide separate protection at the toe of the bank with an adjustable armor or flexible mattress. With this approach, the dikes will limit the velocity and associated general scour near the bank, theoretically allowing a less substantial toe protection than without dikes. This approach may not be cost effective for preventing general scour, since the effect of dikes on general scour cannot be reliably predicted, requiring a more conservative design for the separate toe protection than is theoretically necessary. However, it is often used to protect against local scour induced by the dike itself.

Provide separate protection riverward of the bank toe, perhaps along a line connecting the ends of the dikes. This is more positive, but usually more costly, thus negating the cost advantage of dikes. In fact, in the extreme case, this approach would more properly be termed "retards," and the dikes would simply serve as "tiebacks" and would be the secondary component of the work.

Some **permeable dike** designs, such as tire-posts and "Palisades" (a commercial product) allow components of the structure itself to displace downward, maintaining contact with the bed as scour occurs. With these designs, the cautions that are stated below for flexible retards are applicable.

If **impermeable dikes** are constructed of material which will launch into a scour hole as it occurs, such as stone, the size of the scour hole will tend to be self-limiting. However, impermeable dikes are often less effective in inducing deposition than permeable dikes, and are likely to produce more concentrated flows and higher velocities locally, which tends to offset this positive effect.

RETARDS:

For **rigid retards**, such as non-adjustable fencing or piling, some designers assume that the impedance to flow provided by the structure and subsequent landward deposition of sediment will prevent toe scour from endangering bank stability, thus they make no specific provision for limiting toe scour. This approach is sometimes successful. However, it is not recommended unless predicted scour is less than a few feet and the following conditions are met:

Pile penetration and size are designed for predicted scour

The structure will be frequently monitored and reinforced if necessary

Bed material transport in the stream is large, thus deposition behind the structure is likely

The distance from the toe of the bank to the structure is at least twice as far as the magnitude of the predicted scour.

More positive methods for providing toe protection to rigid retards are:

Construct in an excavated trench to predicted scour depth

Use a self-launching toe section of stone or other material

Secure a flexible mattress to the riverside of the retard

Flexible retard designs, such as jacks, tire-post, and trees, allow the retard structure itself to displace downward as scour occurs. Use of this technique requires secure connections between retard units and between the retard structure and the landward anchors, or extra penetration of piling, depending on the specific design. Allowance must be made in selecting retard height so that the downward displacement will not leave the upper bank exposed to significant erosion during high flows. The alternative is occasional maintenance by placing additional units on top as the originals units displace downward. If this alternative is chosen, be certain that structural details in the original design can accommodate future additions.

OTHER FLOW DEFLECTORS:

Bendway weirs and Iowa vanes are similar to dikes and retards in that they function by inducing deposition at the bank toe rather than permitting scour to occur. However, since they change the character of secondary currents rather than simply relocating them, they should be less demanding of toe protection than dikes or retard. However, since these are relatively new techniques, long-term field experience is not available.

VEGETATIVE PROTECTION:

The importance of toe protection here cannot be overemphasized. Vegetation alone is not likely to work unless velocities during design flows are so low that little toe scour is predicted, the lower bank is infrequently inundated during the growing season, and climate and soils are conducive to vigorous growth. Selection of a toe protection technique should assume that the vegetated portion of the bank is in effect a rigid armor, which dictates that a self-launching material or flexible mattress be used at the toe. In practice, vegetation is usually used as a cost-saving or environmental feature in conjunction with a structural technique, and appropriate toe protection will be a part of the design of the structural technique. Typical examples are vegetative plantings between dikes, behind retards, and on the upper bank slope above one of the many armor materials.

RETAINING WALL:

If a retaining wall is part of the geotechnical solution, then the approach will be the same as for rigid retards, discussed above. The alternative is to design the wall to be stable under maximum scour, which is likely to be more costly than limiting the scour, and which also is more likely to produce the specter of sudden, dramatic, and perhaps catastrophic mass failure in the event of miscalculation, since underdesign of toe protection is more likely to manifest itself gradually and is more easily detected in time for remedy than is excess scour during high flows in the absence of toe protection. Since retaining walls are often used in situations where consequences of failure are high, increasing the safety factor by using toe protection as well as extra structural strength may be prudent.

A powerful variation on the retaining wall approach is to combine the geotechnical solution with erosion prevention and toe protection by using a stone bulwark longitudinally along the bank to act simultaneously as a retaining wall, lower slope armor, and toe protection (Figure 13).

6. SURFACE DRAINAGE

Inadequate provision for surface drainage seldom results in complete failure of the work, but it should not be neglected. It can be a major concern to adjacent property owners. Design flaws here give the impression of incompetent design, affecting public perception of the success of the work. Careless mistakes occur easily, because our primary focus is usually on channel behavior, and proper design for overbank flow outlets can be a tedious process, especially if rigorous design procedures are followed.

Attention to surface drainage is even more important if the stream is degrading, and flowline lowering is anticipated, since overbank drainage channels will likewise degrade if not adequately protected.

The amount of design effort which is feasible will be determined by the:

project purpose

susceptibility of a site to damage, which depends on topography, rainfall, vegetation, soil characteristics, and the type of bank stabilization which is being used

engineering, environmental, and political consequences of erosion

feasibility of collecting sufficient data to permit a rigorous design

The potential for surface erosion is best determined by observations of existing problems, but construction of bank stabilization work can make the problem worse, as well as more noticeable, since gullies leading into the stream will no longer be periodically destroyed by streambank caving. Freshly graded banks are particularly susceptible to surface erosion, and natural levees and existing drainage pattern and vegetation may be disturbed by construction operations.

The basic steps in preventing erosion from surface drainage are to:

Protect all bare ground
Collect the overland flow
Provide outlets into the stream

In the simplest of situations, surface drainage will be away from the stream in sheet flow, to a natural interior drainage channel. In this case, **protection of bare ground** on unarmored bank slopes and areas disturbed by construction activities is all that is necessary. This is usually done with vegetative treatments. Various types of chemical soil stabilizers are also available and are often effective. Manufacturers can provide recommendations and service records for a particular application. Treatments for more severe conditions, such as soil-cement, cross the boundary from surface erosion protection into bank stabilization.

If topography is such that significant amounts of surface drainage will enter the channel in the vicinity of the work, **collecting the overland flow** is necessary. This can often be accomplished by small unlined ditches if drainage areas are small, slopes are flat, or the soil is erosion resistant. The ditches should usually include vegetative treatment or soil- stabilization. If grading of the bank is part of the stabilization work, the natural levees should be rebuilt using material from bank grading or ditch construction, as shown in Figure 14. Beyond this point, into the design of lined ditches, options become a more complicated specialized topic, comprehensively addressed by Schwab et al (1981). Useful advice can also be obtained from local soil conservation experts.

Providing outlets into the stream is sometimes a simple matter of leaving a natural outlet undisturbed, if the flow carried by the outlet is not increased by alterations to the topography during construction. Otherwise, or if the natural outlet shows signs of instability, a lined outlet or culvert should be provided. Steep drops can be accommodated by a drop culvert, or by providing energy dissipators at the ends of lined outlets or culverts. The design of these is site-specific. Again, Schwab et al (1981) treat the subject thoroughly, and other specific guidance based on site conditions can be obtained locally.

Rigid armor is more susceptible to being undermined at the top from surface drainage, and to buildup of excess hydrostatic pressure from surface drainage being trapped under it than are most other armors. Therefore, special care should be taken in collecting surface water and providing outlets into the stream. "Keying in" the top of the armor, or providing a "collar" of adjustable armor, is a common practice.

When indirect bank protection methods are used, surface drainage is often not a consideration, since the work usually does not alter existing drainage detrimentally. Reduction of erosion from surface drainage may be an incidental benefit of the work if deposition behind the bank protection structure raises the base level of existing outlets. This may in fact present a problem if deposition is so high as to block local drainage outlets. Usually, however, the only drainage treatment necessary with indirect protection is to treat areas disturbed during construction. Treatment for local surface erosion can be designed separately if it is a significant problem to be addressed under the project.

7. MANUFACTURERS RECOMMENDATIONS

This factor was mentioned earlier in discussion of a few specific topics. The general point to be made in listing it separately here is this: Manufacturers and distributors of the various patented or commercially available erosion protection products may not be completely objective, since they want to sell their product. However, they also want their product to perform well, and their experience with it is likely to be extensive. One should never accept their recommendations on blind faith, but if they are supported by a service record under comparable conditions, much of the design work for a particular method will have been done already. However, if you are in government practice, be aware that procurement policies sometimes prevent specifying a particular product by name. Adding the phrase "or equal" to the specification may alleviate that difficulty, but even if a contractor's proposed substitute is considered by engineering personnel to not be "equal," documenting that to the satisfaction of administrative personnel may be tedious.

8. SAFETY FACTOR

Most engineering analyses for designing structures such as buildings and bridges provide for a "safety factor," even when the physical laws governing the behavior of the structure are well-defined and readily quantifiable. Because rigorous design procedures are lacking for many aspects of streambank protection, the need for a safety factor is even more apparent. However, there is a fallacy in the analogy, in that failure of buildings and bridges invariably carries the ultimate risk, that of loss of life, whereas that is often not the case with streambank protection works. Also, the safety factor for buildings and bridges is usually strictly specified by codes of practice, which is likewise not usually the case for us.

EM 1110-2-1601 suggests a safety factor of 1.1 for riprap. Beyond that, it would be impossible as well as presumptuous for us to quantify here the safety factor for specific situations, especially since basic design parameters can't be quantified to begin with for many protection methods. We will have to be satisfied with pointing out that at least a qualitative evaluation should be a specific and tangible step in the design process.

The safety factor is influenced by:

The engineer's experience (of lack thereof)
- with the protection method being used
- with the stream itself or comparable streams

Difficulty of construction of the work

The sponsor's capability
- to perform routine maintenance
- to perform emergency reinforcement

The consequences of failure.

The **engineer's experience** with the protection method and familiarity with the stream is a measure of the confidence that can be placed in a prediction of the performance of the work and in a prediction of the consequences of failure of the work. The reliability of available data on the stream's characteristics also influences the level of confidence provided by experience.

The **difficulty of construction of the work** affects the possibility that undetected construction flaws will leave vulnerable points. The timing of construction affects whether weather and high flows will extend the work period, making incomplete portions of the work more vulnerable, especially if vegetative treatment is an important component. Also, construction delays which result in changes in the channel may make the design itself unsatisfactory in some respects.

A very useful way to reduce problems which occur when site conditions change in the time between design and construction is to provide prospective bidders with sufficient information to bid on the work, then provide the details of the work based on site inspection and surveys immediately prior to construction. This allows you to place the work more confidently, thus reduces the safety factor.

Difficulty of construction is also influenced by the competence of the construction forces and the capability of their equipment. Unfortunately, if the work is to be done by the lowest bidder, this will be unknown during design.

The sponsor's capability and commitment to perform routine maintenance will determine the probability of minor failures becoming catastrophic. The sponsor's ability to perform emergency reinforcement under difficult conditions will affect the safety factor for work which protects important facilities such as levees. The availability of sources of assistance during emergency conditions is also a factor. Project documentation should emphasize the importance of monitoring and maintenance, if appropriate.

Consequences of failure ("COF") will probably be the most important single element of the safety factor. This requires an assessment of the likelihood of loss of life, significant property damage, or severe stream channel instability if the work fails. Particularly when faced with design decisions that cannot be resolved analytically, let the COF be the deciding factor.

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SOIL CEMENT

Lecture 10

Soil Cement

Adapted for Flood Control District short course by D. Williams, Ref. 9

1. Definition.

   A highly compacted mixture of soil, portland cement and water.

   As the cement hydrates, it hardens into a strong, durable, low permeable material.

2. Uses.

   - Pave streets, airports, parking lots

   - Slope protection on dams, levees

   - Line spillways, channels, reservoirs

3. Advantages:

   - Low cost

   - Ease of construction

   - Uses local or in-place soil

   - Environmentally attractive ?

   - May serve as alternative to riprap

4. <u>General Requirements</u>.

- Soil should be easily pulverized
- (>5% but <35% silty clay)
- For velocities >8 fps - should contain at least 20% gravel (4.74 mm)
- Fine textured soils (clays) difficult to pulverize and require more cement
- Cement contents vary between 7-12% by weight.

5. <u>Construction Techniques</u>.

- Placed in horizontal layers 6 to 9 ft. wide.
- Layer thickness 6 to 9 inches
- Can be mixed in a central plant or in-place
- Must be placed in the dry
- Can be broken up in riprap size pieces for underwater placement

6. <u>Soil Cement Characteristics</u>.

- Low permeability
- Shrinkage cracks can sometimes be a concern.
- Mix design determined by Standard ASTM procedures; i.e., cement content, optimum moisture content, and maximum density
- Cement content increased by 2% if in freeze-thaw environment.
- Zone between layers must be kept clean and moist to get proper bond.

7. <u>Basic Steps to Fabricate Soil Cement/Riprap</u>.

a. Obtain gradation analysis of soil to be used.

b. Conduct laboratory tests to determine cement content.

c. Inspect subgrade of area to use to fabricate mixture.

d. Determine curing method - slow curing process is necessary; avoid disturbances or vibration during curing period.

e. Determine method of mixing (in place or central plant)

f. Compaction - compaction process started immediately - and completed within three hours after mixing.

g. Thickness - single layer of at least maximum diameter of desired material.

h. Fracture process - Scarify upper three inches of layer to create fracture lines during breaking process.

I. Breaking - Cured spoil cement will easily break on fracture lines by driving equipment over the material

j. Test Section - Set up test section to evaluate effectiveness of fabrication process.

k. Inspection - Continuous inspection is essential to ensure quality control of proper cement content, compaction, curing, and placement procedures.

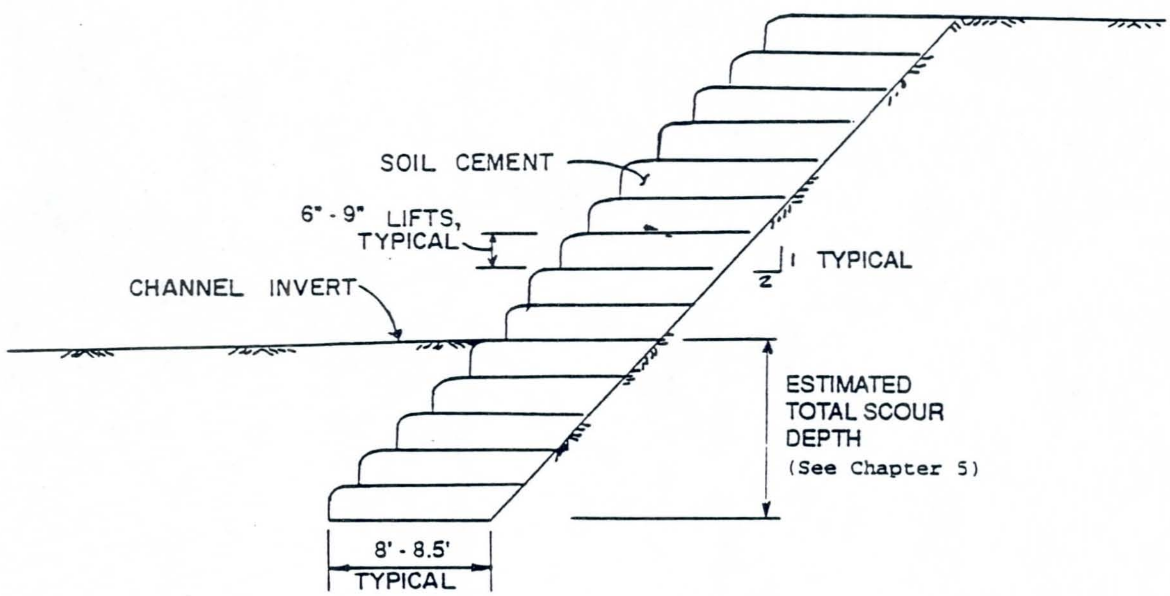

Figure 6.5
Soil Cement Placement Detail
(Not to Scale)

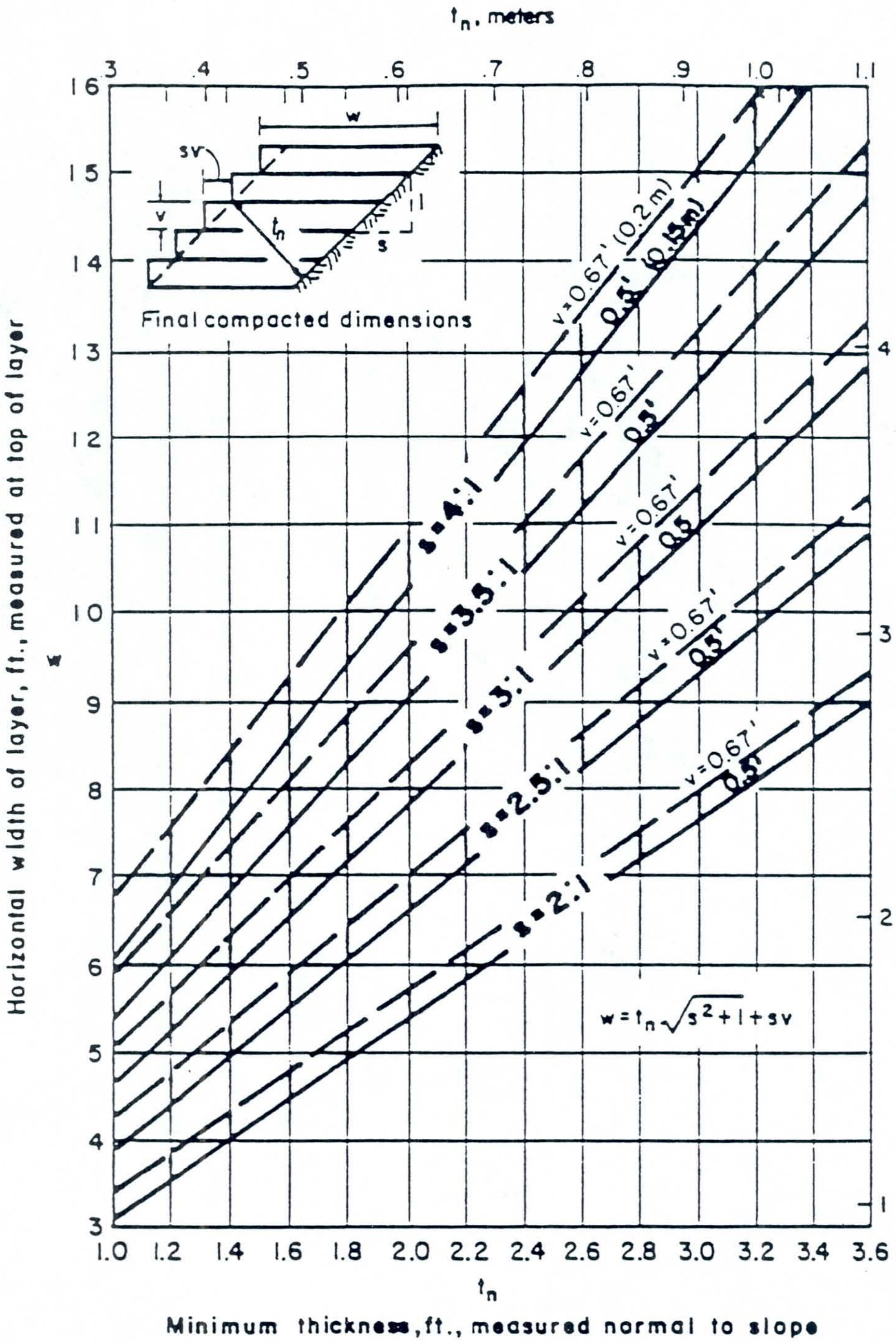

Figure 6.6
Relationship of Slope, Facing Thickness, Layer Thickness, and Horizontal Layer Width for Soil Cement Lining
(Portland Cement Association, Undated)

SEDIMENT TRANSPORT

CONCEPTS

Lecture 11

SEDIMENT TRANSPORT CONCEPTS

By Peter Klingeman, Ph.D., Oregon State University

Adapted for Flood Control District short course by D. Williams, Ref. 10

THE "DOUBLE CONDITION" THAT CONTROLS SEDIMENT TRANSPORT

Not all sediment that passes a cross section of a river obeys the same natural laws. Therefore, a relation to describe the transport of one part of the sediment load with great precision will be unable to describe the transport of the other part of the sediment load.

It is important to know:

a) the kind and amount of material that can be moved by a given discharge in a given channel;

b) what will happen if less material is available for movement than the river has energy to move; (The flow will try to erode the channel)

c) what will happen if more material is available for movement than the river has energy to move; (The flow will only move as much material as it is capable of moving)

d) what will happen if the flow rate increases or decreases , for a given amount of material present; (The transport capability will change)

A "double condition" thus exists.

This offers one way to evaluate the likelihood of sediment transport and the amount of transport that might occur.

As the term implies, two conditions are involved:

(1) sediment availability

sediment must be available for transport by the flow from somewhere upstream in the channel or basin;

(2) flow capability

the flow must be capable of moving the available sediment through the channel past the point or cross section of interest.

Each condition can limit the sediment discharge past a channel cross section:

a) sediment may be available but the flow may be incapable of moving it;

b) the flow may be capable of moving only part of the available sediment;

c) the flow may be capable of moving sediment but sediment may not be available;

d) not as much sediment may be available as the flow is capable to move.

Thus:

$$(\text{flow capability}) \quad \begin{matrix} > \\ = \\ < \end{matrix} \quad (\text{sediment availability})$$

If greater, not as much sediment moves as might be expected, based on the amount of flow.

If less, little or none of the available sediment moves.

If equal, prediction of the sediment load may be possible.

The relative magnitudes of the two limiting conditions (availability and capability) will determine whether availability upstream of the reach or transport capability in the reach will limit the actual amount of sediment being transported.

Sometimes one condition will govern; sometimes the other will govern. This is because the stream discharge is variable with time, as are other watershed processes (for example, soil freezing, landslide likelihood).

Two entirely different sets of conditions are involved, obeying different rules. Each is complicated and difficult to describe, evaluate or check.

In most rivers:

> transport of <u>finer-sized</u> sediment, which can be transported in large quantities, is usually limited by its <u>availability</u> in the basin; and
>
> transport of <u>coarser</u> sediment, which is more difficult for the flow to move, is usually limited by the <u>capability</u> of the flow to move it.

The above discussion of sediment source-transport-deposition zones and the "double condition" of sediment availability and flow capability shows the need to evaluate all conditions governing sediment transport.

TYPES AND CLASSIFICATION OF SEDIMENT TRANSPORT

Sediment discharge: the quantity of sediment per unit time carried past a cross section of a stream.

Definitions based on the mechanisms of movement:

bed load:

(1) that part of the sediment load consisting of coarse material moving on or near the bed;

(2) material collected in or computed from samples collected in a bed-load sampler or trap.

suspended load:

(1) material moving in suspension;

(2) material collected in or computed from samples collected with a suspended-load sampler.

Definitions based on composition of the bed, the source area, or the method of calculation:

bed-material load: that part of the sediment load which is composed of particle sizes found in appreciable quantities in the shifting (moveable) portions of the stream bed.

wash load: that part of the sediment load which is composed of particle sizes smaller than those found in appreciable quantities in the shifting portions of the stream bed.

Additional definitions involving the bed-material load:

contract load: that part of the sediment load consisting of material rolling or sliding along the bed in almost continuous contact with the bed.

saltation load: material bouncing along the bed, or moved directly or indirectly by the impact of the bouncing particles.

Definitions based on method of measurement:

measured load: that part of the total sediment load that is actually measured.

unmeasured load: that part of the sediment load that is not measured.

Comparative classifications of sediment transport that show the various modes of transport of the total sediment load, can be given as follows:

<table>
<tr><td rowspan="3">Total Sediment Load</td><td rowspan="2">Suspended Load</td><td>Wash Load</td><td>Suspension Load</td><td rowspan="2">Measured Load</td></tr>
<tr><td rowspan="2">Bed Material Load</td><td>Saltation Load</td></tr>
<tr><td>Bed Load</td><td>Contact Load</td><td>Unmeasured Load</td></tr>
<tr><td></td><td>by mechanism of movement</td><td>by bed composition, source area or method of calculation</td><td>by manner of movement</td><td>on basis of measurement</td></tr>
</table>

BED LOAD AND BED-MATERIAL LOAD TRANSPORT CHARACTERISTICS

1. Description of development of bed-material sediment motion:

small Q: . . . . . . no sediment motion

larger Q: . . . . . . intermittent rolling/sliding; contact load

still larger Q: . . . contact load and saltation load

still larger Q: . . . contact load and saltation load; suspended load

2. Some terms and their customary uses:

scour . . . . short time scale

erosion . . . . long time scale

deposition. . . . . short/long time scale

sedimentation . . . . long time scale

equilibrium of bed . . . . elevation of bottom remains constant over time

degradation of bed . . . . . . elevation of bottom lowers over time

aggradation of bed . . . . . . elevation of bottom raises over time

3. Equilibrium aspects of the bed for some river reach:

We want to know if:

$$\text{amount of sediment entering reach} \quad \begin{matrix} > \\ = \\ < \end{matrix} \quad \text{amount of sediment leaving the reach}$$

Scour and deposition always occur. But different particles pass the entrance and exit from the reach, since:

$$V_{particle} << V_{flow}$$

For degradation of the bed, the bed elevation drops over time:

(sediment load in) < (sediment load out)

For aggradation of the bed, the bed elevation rises over time:

(sediment load in) > (sediment load out)

For equilibrium of the bed, the bed elevation is constant over time:

(sediment load in) = (sediment load out)

If the sediment inflow to a reach is more than the transport capacity of the flow, the flow will try to deposit material in the bed to adjust for the excess sediment available compared to the transport capacity. Therefore, we have net deposition.

If the sediment inflow to a reach is less than the transport capacity of the flow, the flow will try to remove material from the bed to adjust for the deficient sediment available compared to the transport capacity. Therefore, we have net erosion.

To know if bed equilibrium exists, we must know if:

(rate of erosion) >, =, or < (rate of deposition)

or

(sediment transport rate out reach) >, =, or < (rate of sediment supply to reach from upstream)

4. Some definitions about equilibrium:

   a. A stream is in equilibrium if it has just that bed slope and cross section needed to transport the water and sediment load coming from the watershed.

   b. An equilibrium stream = a graded stream.

   c. "Dominant" discharge of a stream is that equivalent steady discharge which would produce the same stream waterway (bed slope, cross section)

   d. An equilibrium stream is said to be "in-regime".

   e. A "regime" stream is one which is in the regime condition or is adjusting itself to attain this condition.

5. Developing a transport relation:

   Equilibrium of bed load - some sediment is moved downstream from part of the bed but enough sediment comes from upstream to take its place.

   Scour - depends on strength of flow and rate at which particles are moving in and out of part of the bed.

   Deposition - depends on strength of flow and rate at which particles are moving. The number of particles in motion depends on the rate at which particles are moving.

6. Developing a bed load equation:

   Bed load equations are based on an assumed bed equilibrium whereby:

   scour (or local erosion) = deposition

   A stream reach is in equilibrium if it has the needed bed slope and cross section to transport the water and sediment load coming from upstream.

We want to know:

How much sediment is being supplied to a river reach?

How far will the river transport this sediment?

Where will this sediment deposit the river?

How can the river and its sediment be controlled to achieve equilibrium or erosion or deposition?

The bed supplies sediment:

a) in accord with the transport capability' or

b) in accord with the availability of particles.

With respect to time, this happens:

never ----> irregularly ----> periodically ----> continuously.

. . . . typical situations . . . .

7. The bed load function:

If there are enough bed particles (i.e., availability is great), then the river will transport grains at capacity, where:

capacity/capability = f (channel, flow, grain-characteristics).

This can be described in the form of: $Q_{sediment} = f(Q_{water})$

For a given channel and streambed, this is considered to be a unique function. It is called the bed load function.

This can be determined

a) by direct measurement;

b) by theoretical calculations based on field data.

But, for a given river, there are great variations over time and location.

a) due to bars and dunes in the bed;

b) due to passage of floods, etc.

Therefore, we must use averages over time.

The time periods can be of the order of

a) seconds (small bars and bed irregularities)

b) season of year (large bars).

CAUSES AND CONSEQUENCES OF SCOUR AND DEPOSITION

Adapted for Flood Control District short course by D. Williams, Ref. 3

1. Definitions

The process of sedimentation are classified as local scour and deposition or general scour and deposition.

The latter is referred to as degradation (lowering of the stream bed profile by erosion of the bed) and aggradation (the raising of that profile by deposition on the channel bed).

2. Scour

   a. Degradation.

      Degradation is the term describing a general lowering of the stream bed elevations due to erosion of the bed sediments.

      For example, sediment deficient water released to the channel downstream from a dam has the potential to cause generalized scour.

When inflowing water is deficient in sediment of the size classes forming the bed, degradation will start at the point of inflow and move in the downstream direction.

When the tailwater control has shifted downward, degradation will start at the downstream and move upstream.

There can be combinations of these conditions not only at project boundaries but also in the project reach.

The significance of the trend is often masked by the slow rate of growth, but a degrading stream is a potentially severe problem which should be investigated to discover the cause and develop a solution.

Numerical modeling techniques provide the computational framework for such investigations.

b. Head Cuts.

Another common type of general scour is head cutting. Head cutting is a discontinuity, i.e., a rapid drop or waterfall, in the stream bed profile which moves in the upstream direction.

It occurs when the channel bed sediment is weakly cohesive.

This condition is common when the base level of the stream is suddenly lowered.

When runoff from the watershed is the only source of water, the height of the vertical face will decrease as it moves upstream because the water yield decreases.

Head cutting is an important consideration because it promotes bank caving; it causes bridge failures as well as failure of other structures in its path; and it increases the sediment discharge into the receiving stream.

c. Local Scour.

Local scour is the term applied when erosion of the channel bed is limited, in plan view, to a particular location.

It can occur in otherwise stable reaches of a stream as the direct result of a disturbance to the flow field.

(1) Bridges.

Because of their number, bridges are the most frequent cause of local scour. The scour depth can be attributed to two processes.

First is the increase in unit discharge across the channel caused by the bridge abutments and piers. That increase causes a greater bed shear stress, resulting in erosion of the bed sediment.

Secondly, the disrupted flow field contains pressure fluctuations which add lift forces to the bed sediments.

(2) Points of Caution.

(a) Local scour should be regarded as a potentially severe problem in any mobile bed stream.

(b) The maximum depth is difficult to assess since the most severe scour will often occur during the peak flow and deposition will fill in the scour hole as the hydrograph recedes.

d. Drop structures.

Local scour also occurs below drop structures.

It shows up as a deep hole, in erodible beds, flanked by bank caving.

Standard drop structure designs provide bed and bank armoring to limit this type of scour.

e. Miscellaneous.

Local scour also occurs at the downstream junction between riprap and revetment and the natural earth channel.

Channel training dikes cause local scour.

3. Deposition.

a. Aggradation.

General deposition, like general scour, spans long reaches of a stream.

It will occur when the concentration of inflowing sediment exceeds the transport capacity of the stream at that cross section.

The deposition process starts at the upstream end of the reach and moves toward the downstream end; however, there is a feed back loop.

That is, as the deposit moves downstream the backwater effect is reflected in the upstream direction which results in more deposition. The rate and limits are predicted by numerical sediment modeling techniques.

b. Local Deposition.

Local deposition compares to aggradation like local scour compares to degradation.

It refers to a deposition zone that is limited in aerial extent. It implies nothing about the severity of the problem.

For example, when the channel width expands, transport capacity will decrease.

Sand and gravel will deposit as a center bar because the particles are too heavy to move laterally. During the intermediate range of flow depths, this center bar will deflect water toward both banks.

If the banks are unprotected, bank erosion would be expected and that would initiate a new plan-form alignment starting at the center bar and progressing downstream.

On the other hand, streams which are carrying silt and clay would be expected to deposit sediment in the eddies formed on either wide of the expansion until a narrower stream width is produced.

c. Points of Caution.

The following symptoms of general aggradation problems are given to aid in assessing the condition of a stream. These are not an exhaustive list. As other symptoms area recognized, they should be added to it.

(1) When the plan-form changes from straight to meandering with the no actively caving banks or bar building, the inflowing sand and gravel discharges are in balance with the transport capacity of the stream.

However, when the plan-form changes from straight to meandering with associated actively caving banks, the inflowing sand and gravel discharges exceed the transport capacity of the stream and that location is aggrading.

(2) When the plan-form changes from straight or meandering to braided the inflowing sand and gravel discharges exceed the transport capacity of the stream.

(3) When a channel avulsion has occurred and there is no evidence of a downstream, hydraulic control, the inflowing sand and gravel discharge probably exceeds the transport capacity of the stream in that reach and deposition has filled the channel causing the water to seek a lower place on the valley floor.

(4) The significant slope in transport of sands and gravels is the local energy slope not the general slope of the stream.

(5) Bank caving due to geotechnical failure is associated with a degrading reach. Active meanders, those at which there is active bank erosion, are more likely to be associated with an aggrading reach than a degrading reach.

Table 6.10
Design Checklist for Sediment Transport Analysis

Level I Sediment Transport Analysis
Data Requirements
Determination of Plan Form Characteristics
Lane Relation and other Geomorphic Relationships
Aerial Photograph Interpretations
Bed and Bank Material Analysis
Land Use Changes
Flood History
Rainfall/Runoff Relationships
Level II Sediment Transport Analysis
Data Requirements
Watershed Sediment Yield
Detailed Bed and Bank Material Analysis
Profile Analysis
Incipient Motion Analysis
Armoring Potential
Sediment Transport Capacity
Equilibrium Slope Analysis
Sediment Continuity Analysis
Quantification of Vertical and Horizontal Channel Response
Bend Scour
Low Flow Channel Incisement
Gravel Mining Impacts
Contraction Scour
Local Abutment Scour
Local Pier Scour
Cumulative Channel Adjustment
Level III Sediment Transport Analysis
Data Inventory Modeling
Watershed Sediment Modeling
Instream Mining Response
Single Event Stream Bed Modeling
Long Term Bed Modeling

Resources:

1. Laursen and Duffy, 1980
2. FHWA, 1990
3. Sabol, Nordin, and Richardson, 1990
4. Simons, Li and Associates, 1985

SEDIMENTATION OR

DEBRIS BASINS

Lecture 12

SEDIMENTATION OR DEBRIS BASIN DESIGN

Adapted for Flood Control District short course by D. Williams, Ref. 3

1. Debris Basin Design.

Debris basins, sometimes called sediment retention basins, are reservoirs designed to trap sediment and debris.

They are not intended for water storage or peak discharge control.

In this usage, debris refers to the assortment of sand, gravel, cobbles, boulders, logs and other large pieces of material that deposit in a channel, causing flood flows to spill out before design conditions are reached.

2. Need.

Generally, debris basins are used where channel slope becomes flatter, for example, where a stream leaves hills and flows across a flood plain.

The need is easily identified by noting channel meander and braiding patters on aerial photographs.

The amount of reduction in sediment discharge can be estimated by making sediment transport calculations.

3. Design Considerations.

Debris basins are growing in popularity; however, little work has been done to aid in their design and evaluation except in the southern California area, and that work is not portable to other locations.

   a. Design guidelines.

      The Federal Highway Department has published guidelines for sedimentation basin design.

b. Safety.

It is imperative that project safety be a key factor in sizing the basin.

Project safety requires not only design flood considerations, but also the proper consideration of conditions antecedent to a design flood.

Also, the debris basin should function so if a flood should occur which exceeds the design flood, the project will not make conditions worse than would have occurred without the project.

c. Location.

Debris basins are place upstream from flood protection or navigation channels.

Access and shape are important considerations because they affect clean-out and trap efficiency, respectively.

d. Basin size.

They are usually small and designed to be cleaned out from time to time.

However, the size is not arbitrary. It must be justified by project economics and available sites.

Some basins are sized for only one or two major storms. Others may have a 50 or 100 year capacity.

e. Topset slope.

The volume available for sediment storage is the debris basin is considerably different from the horizontal planes used in water storage calculations.

A delta will form in these basins just as it does in a reservoir.

Starting at the crest of the dam, the topset slope of the delta can be estimated to be 50 percent of the original valley slope.

That is adequate for the impact assessment, but numerical modeling should be used to calculate a topset slope for the detailed sedimentation study.

It will often exceed the 50% approximation.

Of course, trap efficiency of the basin decreases as it fills, and that will determine how much material can be stored before removal is required.

f. Sediment yield.

Sediment yield estimates for debris basin design should include two kinds of hydrological events: the normal, long term records and the design flood events.

Long term average sediment concentration records should be used for the long term hydrologic events. The long term average concentration is determined from the best fit line through the log-log plot of water discharge versus sediment discharge.

It assumes flood data are available and low flow data were not extrapolated up to the range of water discharges in the design flood peak.

g. Analysis by particle size class.

Sediment yield studies for debris basin design always require grain size data.

Methods which seem to ignore that data, such as Tatum, actually have it built into the coefficients and procedures. They should be used only in the region for which they were developed.

h. Single event sediment concentrations.

The best fit line on the water discharge-sediment concentration plot should be adjusted upward to develop a concentration for large floods.

For example, in a flood having a chance, or less, 1 or 2%, the sediment concentrations may exceed long term averages by a factor of 2 or 3.

I. Sediment discharge curve extrapolation.

If flood measurements are not available, use the transport capacity approach to extrapolate the water-sediment discharge relationship.

If the concentrations of fines exceeds 10,000 ppm, (10063 mg/1), they will begin to increase transport capacity.

By the time they reach 100,000 ppm (106,640 mg/1) that influence can be as much as a factor of 10 or 20 times the normal transport capacity.

j. Design method.

The deposition characteristics can be checked using numerical sediment models provided the proper skill is used in defining hydraulics.

(1) Defining the hydraulics.

Initially, flow is 3-dimensional; however, the rapid deposition of sediment seems to cause a rapid return to the 1-dimensional channel hydraulics problem. Therefore, a 1-dimensional numerical model can be used provided the following flow field -sediment deposition concepts are modeled.

(2) The inflow will not expand instantaneously.

(3) Deposition will occur for sands and gravels and the location will start near the inlet.

(4) Deposition of sands and gravels will first fill the channel under the expanding jet until the loss in conveyance causes the jet to deflect to one side or the other.

(5) Whereas, the coarse particles settle out under the expanding jet, 1 to 2 fps is enough energy to keep the fines in suspension.

Fines in the slower velocity water adjacent to the jet will be entrained by eddys and deposit toward the sides of the basin if at all.

(6) As the basin fills the fluid jet will tend toward the same width as the natural channel width rather than remaining a uniformly distributed velocity across a wide basin.

k. Embankment height.

The height of the top-of-embankment above the spillway crest should be designed for the condition when the active flow channel has become the width of the inflowing channel and is located adjacent, and parallel to, the embankment.

Calculate the height of embankment using a slope equivalent to the valley slope transporting sediment into the basin and the distance from the spillway to the end of embankment.

Add freeboard and velocity head to that height as appropriate to turn the approaching flow. That will accommodate an energy loss for a flow that is the width of the natural river channel and flowing along the face of the embankment.

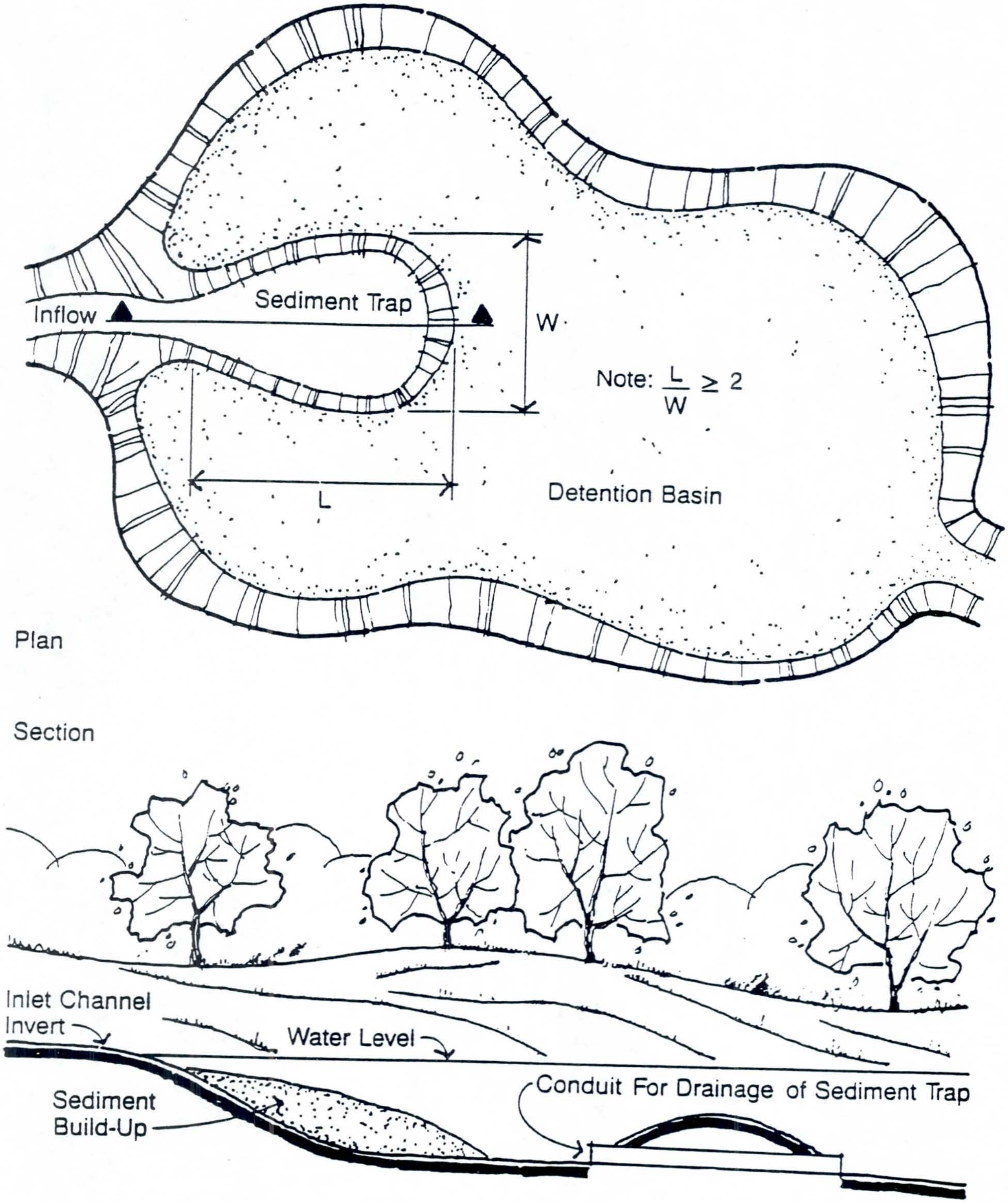

Figure 8.10
Sediment Trap Concept
(Pima County Department of Transportation and Flood Control District)

ENVIRONMENTAL CONSIDERATIONS IN HYDRAULIC DESIGN

Lecture 14

ENVIRONMENTAL CONSIDERATIONS

Adapted for Flood Control District short course by D. Williams, Ref. 8

By Anne MacDonald

1. OBJECTIVES

The purpose of this lecture is to provide guidance on integrating environmental considerations into the design of flood control projects.

Participants should be able to understand general environmental concepts, major types of impacts which a given project will have, and identify design features which will reduce these impacts.

2. REFERENCES

- EM 1110-2-1205 "Environmental Engineering for Flood Control Channels"
- WES TR's E-82-9 "Assessment of Environmental Considerations in the Design and Construction of Waterway Projects"
- E-84-11 "Environmental Features for Streambank Protection Projects"
- E-85-3 "Incorporation of Environmental Features In Flood Control Channel Projects"
- E-85-7 "Environmental Features for Streamside Levee Projects"
- Rivers: Form and Process in Alluvial Channels, by K. Richards, Methuen & Co. 1982
- Impounded Rivers: Perspectives For Ecological Management by G. E. Petts, Wiley 1984

Useful journals: Regulated Rivers (Wiley);
Environmental Management (Springer-Verlag)

3. <u>INTRODUCTION</u>

Environmental (biological and cultural) resources are commonly well developed and significant near river channels. In addition, they will be closely tied to the physical and biological diversity represented by the pre-project condition.

Therefore, environmental features for channel projects are aimed at limiting or preserving these existing conditions within the scope of the project itself.

4. <u>DETAILED LECTURE OUTLINE</u>:

I. Introduction

II. Environmental Policy: Why are we concerned?

A. Laws and Regulations:

1. NEPA [PL 91-190]

a. Responsibility to present and future generations to preserve, conserve resources.

b. Proponent agency shall document:

I. environmental impacts of proposed action

ii. unavoidable adverse environmental effects

iii. alternatives (<u>including</u> "do nothing")

iv. balance between short-term use and long term productivity

c. Spawned numerous SEPA's

B. Increased project benefits over design life:

Give the customer a project to enjoy <u>between</u> floods.

C. Stewardship role: see above (II-A-1a)

III. Environmental Impacts

A. Water quality: transient and permanent

1. Temperature and implications (DO)

2. Sediment concentration

B. Terrestrial ecosystems

1. Elimination

2. Remove from flooding: succession

C. Aquatic ecosystems

1. Increase energy

2. Decrease diversity

D. Cultural resources: High likelihood of disturbance

E. Recreation: Remove river from the local community

F. Aesthetics: Varying degrees of "unnaturalness"

IV. Environmental Features

A. Design principles: "Mother Earth Designs, Inc."

1. Channel stability -- with a little give to accommodate other changes in the system.

2. Physical diversity -- IMPORTANT to throw away the ruler.

3. Minimal disturbance to what is there.

4. An example: Aquatic habitat

    I. Physical diversity: slackwater, organic retention

    ii. Mimic desirable natural conditions: cover, temp.

    iii. Stable substrate (spawning, bugs, etc.)

    iv. Tied to terrestrial inputs

B. Selection procedures for environmentally enhanced design

1. Establish project objectives

    a. Flood damage reduction

    b. Environmental objectives:

        i. Water quality

        ii. Fish and wildlife habitat

        iii. Social: recreation, aesthetics, cultural resources

        iv. MIX?

2. Selection matrix: Fit features to your objective(s).

3. Suitability matrix: Fit features to stream reach, system characteristics.

C. Features for environmentally enhanced design

1. Selective clearing and snagging: "Streams will always need big wood -- too much too fast is just no good"

   a. Environmental effects: selective C & S if possible.

   b. Hydraulic effects: remove sediment too? selective?

   c. Costs

   d. Maintenance

   e. Selection of trees: significant "n" increase?; F&W

   f. Disposal and utilization: firewood? F&W

   g. Construction methods: minimize use of heavy equipment

   h. Revegetation after you're done

2. Designs for excavated channels

   a. High and low flow channels

      i. Environmental benefits

      ii. Design alternatives

      iii. Maintenance: sediment, conveyance

b. Pool and riffle construction in straight channels

c. Meandering alignments

d. Single bank modification

e. Cutoff bendways: lake or stream?

3. Aquatic habitat structures

    a. Sills

    b. Deflectors

    c. Random rocks

    d. Cover

    e. Fishways

4. Bed and bank protection

    a. Grade control

    b. Linings

    c. Vegetation

5. Levees

    a. Borrow pit considerations: irregular, connected to flow

    b. Setback levees

    c. Berm creation

d. Oversized levees

e. Vegetation management

6. Recreation and aesthetics

7. Construction, O&M

   a. Erosion control

   b. Scheduling: ex., avoiding nesting, spawning, migration

   c. Floating plant

   d. Risk-based maintenance efforts where possible

FEMA REGULATION OF RIVERS, STREAMS AND WASHES

Lecture 15

FEMA REGULATION OF RIVERS, STREAMS AND WASHES

Adapted from FEMA 37 and HEC-RAS lecture notes by D. Williams

A. Introduction

1. The National Insurance Program

The National Flood Insurance Program (NFIP) was established by the National Flood Insurance Act of 1968 and further defined by the Flood Disaster Protection Act of 1973.

The 1968 Act provided for the availability of flood insurance within communities that were willing to adopt floodplain management programs to mitigate future flood losses.

The Act also required the identification of all floodplain areas within the United States and the establishment of flood-risk zones within those areas.

2. Flood Insurance Studies

A vital step toward meeting these goals is the conduct of Flood Insurance Studies (FISs), restudies, and Limited Map Maintenance Program (LMMP) FIS projects for flood-prone communities.

An FIS provides a community with sufficient technical information to enable it to adopt and amend the floodplain management measures required for participation in the NFIP.

An FIS also develops the flood risk information necessary to establish and maintain accurate actuarial flood insurance premiums.

The Federal Emergency Management Agency (FEMA) has compiled the Flood Insurance Study Guidelines and Specifications for Study Contractors (referred to herein as the Guidelines) to define technical policy and procedures to be followed in the preparation of FISs, restudies, and LMMP projects.

General guidance is provided for work involving standard professional practice for flood hazard evaluation and revision, whereas specific instructions are provided for work unique to FISs and subsequent updates.

The results of these studies are set forth in a final FIS report, which contains a written section, flood profiles, figures, and tables.

3. Flood Insurance Rate Map (FIRM)

In addition, an essential product of the study is the Flood Insurance Rate Map (FIRM), which is distributed to the private insurance industry, the community, Federal and State agencies and others.

This map provides 100-year flood evaluations and divides the area studied into flood hazard zones that are used to establish actuarial insurance rates.

The FIRM may also depict areas determined to be within the FEMA-designated floodway and 500-year floodplain.

In addition, certain landmark features in the community may be shown on the FIRM to assist in locating individual properties.

4. FEMA and Community Role in the NFIP

The Federal Emergency Management Administration (FEMA) is the administrator of the National Flood Insurance Program (NFIP).

Communities participating in the NFIP must prevent undue development of the floodplain that could cause a significant increase in potential flood damage.

Under certain rules, generally established by local ordinances, the floodplain can be developed but cannot encroach on the "regulatory floodway", which is defined by conducting a detailed (hydraulic) study using computer modeling.

An approximate study can be conducted without modeling using "hand computations" and floodplains delineated; however, a "regulatory floodway" cannot be determined.

B. Concepts of Floodway and Allowable Encroachment

FEMA defines the floodway as "the channel of a river or other watercourse and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water-surface elevation more than a designated height."

The base flood used to determine the floodway is usually the 100 year flood.

FEMA sets the designated height (sometimes called surcharge) as 1.0 foot but local agencies can specify a smaller, but not higher, height.

Floodways are determined by modeling encroachments (usually with HEC-2 or HEC-RAS) at each cross section (from both sides of the channel) such that the subsequent rise in the water surface elevation, when compared to the un-encroached (natural) condition, is not more that the designated height at any cross section.

The area between the floodway and 100 year floodplain is called the floodway fringe. These areas are shown in the Figure 1.

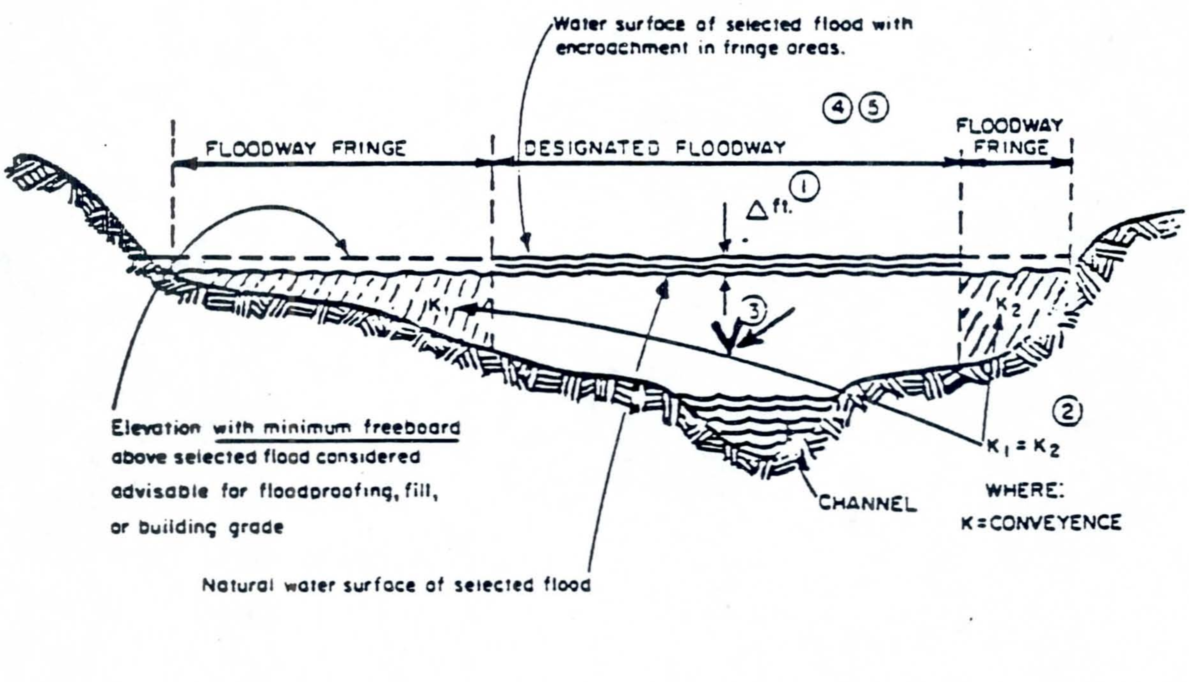

Figure 1, Concept of Floodplains and Floodways

C. Encroachment Methods

HEC-2 and HEC-RAS contain several methods for specifying floodway encroachments. The most commonly used are Methods 1, 4, and 5. In general, contracts for performing detailed studies require that the floodway optimization be performed using Method 4 but the final floodway be defined by using Method 1.

1. Encroachment Method 1

   With encroachment method 1, the user specifies the exact locations of the encroachment stations for each individual cross section.

   An example of encroachment method 1 is shown in Figure 2.

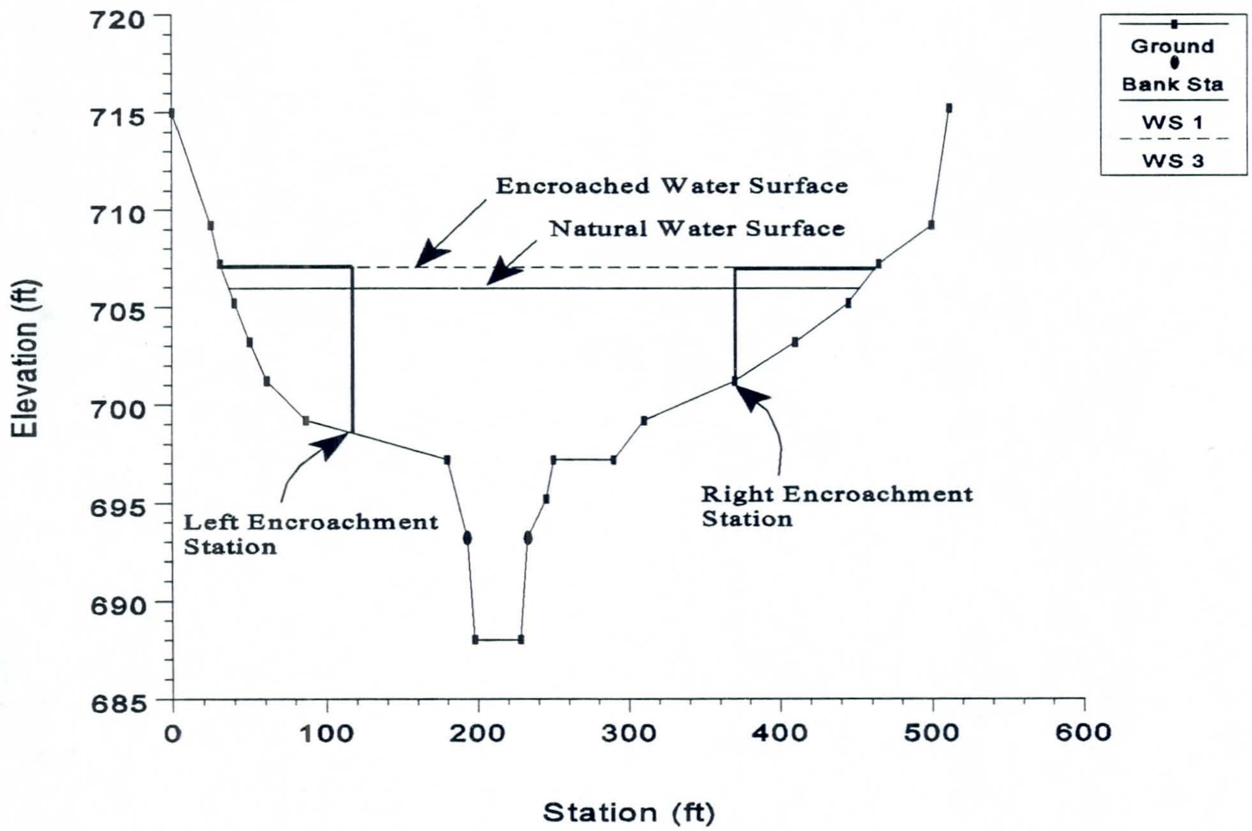

Figure 2, Example of Encroachment Method 1

2. Encroachment Method 4

   Description of Method

   Method 4 computes encroachment stations so that conveyance within the encroached cross section (at some higher elevation) is equal to the conveyance of the natural cross section at the natural water level.

   This higher elevation is specified as a fixed amount (target increase) above the natural (e.g., 100 year) profile.

   The encroachment stations are determined so that an equal loss of conveyance (at the higher elevation) occurs on each overbank, if possible.

   Limitations of Encroachments

   If half of the conveyance loss cannot be obtained in one overbank, the difference will be made up, if possible, in the other overbank, except that encroachments will not be allowed to fall within the main channel.

A target increase of 1.0 indicates that a 1 foot rise will be used to determine the encroachments based on equal conveyance.

This method is illustrated in Figure 3.

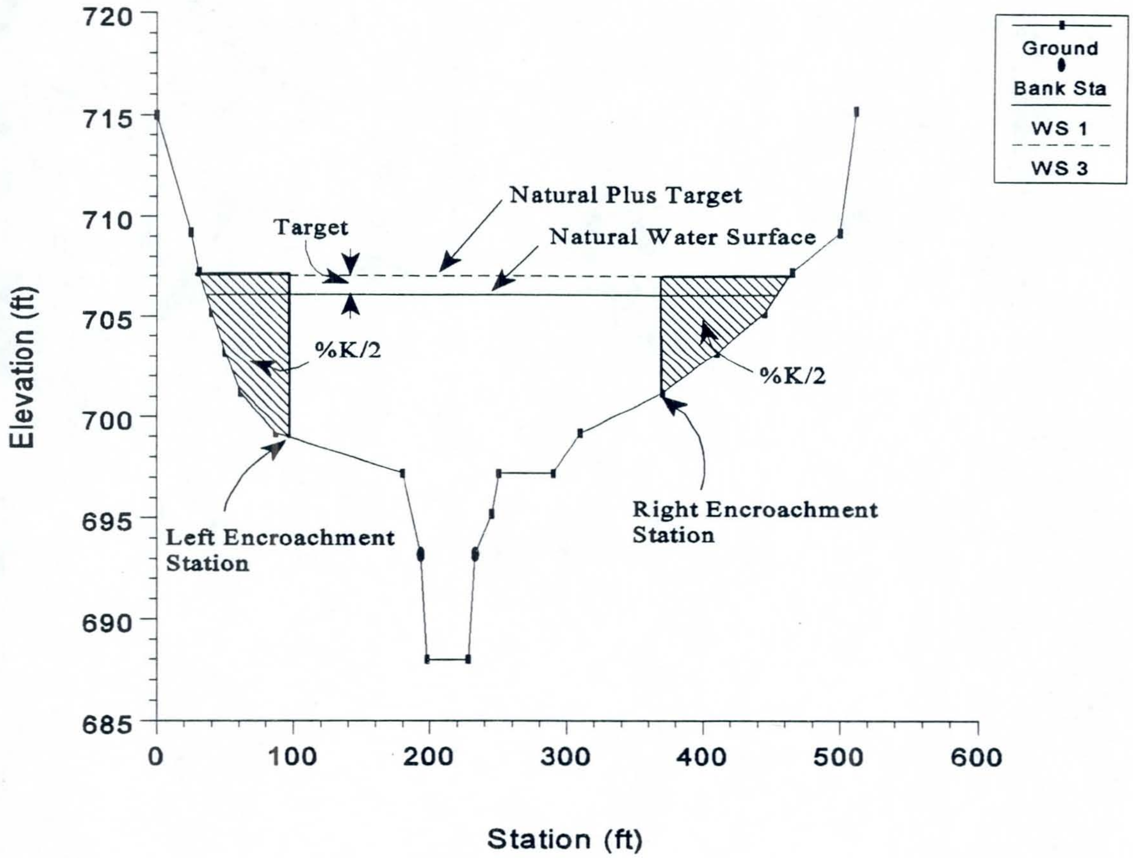

Figure 3. Example of Encroachment Method 4

D. Examination of Initial Floodway Estimates

Recognizing that the initial floodway computations may provide changes in water surface elevations greater, or less, than the "target" increase, initial computer runs are usually made with several "target" values.

The initial computer results should then be analyzed for increases in water surface elevations, changes in velocities, changes in top width, and other parameters.

From these initial results, new estimates can be made and tested.

E. Refinements Using Method 1

After a few initial runs, the encroachment stations should become more defined.

Because portions of several computed profiles may be used, the final computer runs are usually made with encroachment Method 1 defining the specific encroachment stations at each cross section.

Additional runs are often made with Method 1, allowing the user to adjust encroachment stations at specific cross sections to further define the floodway.

F. Obtaining Reasonable Encroachment Alignments

While the floodway analysis generally focuses on the change in water surface elevation, it is important to remember that the floodway must be consistent with local development plans and provide reasonable hydraulic transitions through the study reach.

Sometimes the computed floodway solution, that provides computed water surfaces at or near the target maximum, may be unreasonable when transferred to the map of the actual study reach.

If this occurs, the user may need to change some of the encroachment stations, based on the visual inspection of the topo map and the floodway computations re-run with the new encroachment stations to ensure that the target maximum is not exceeded.

This is illustrated in Figure 4.

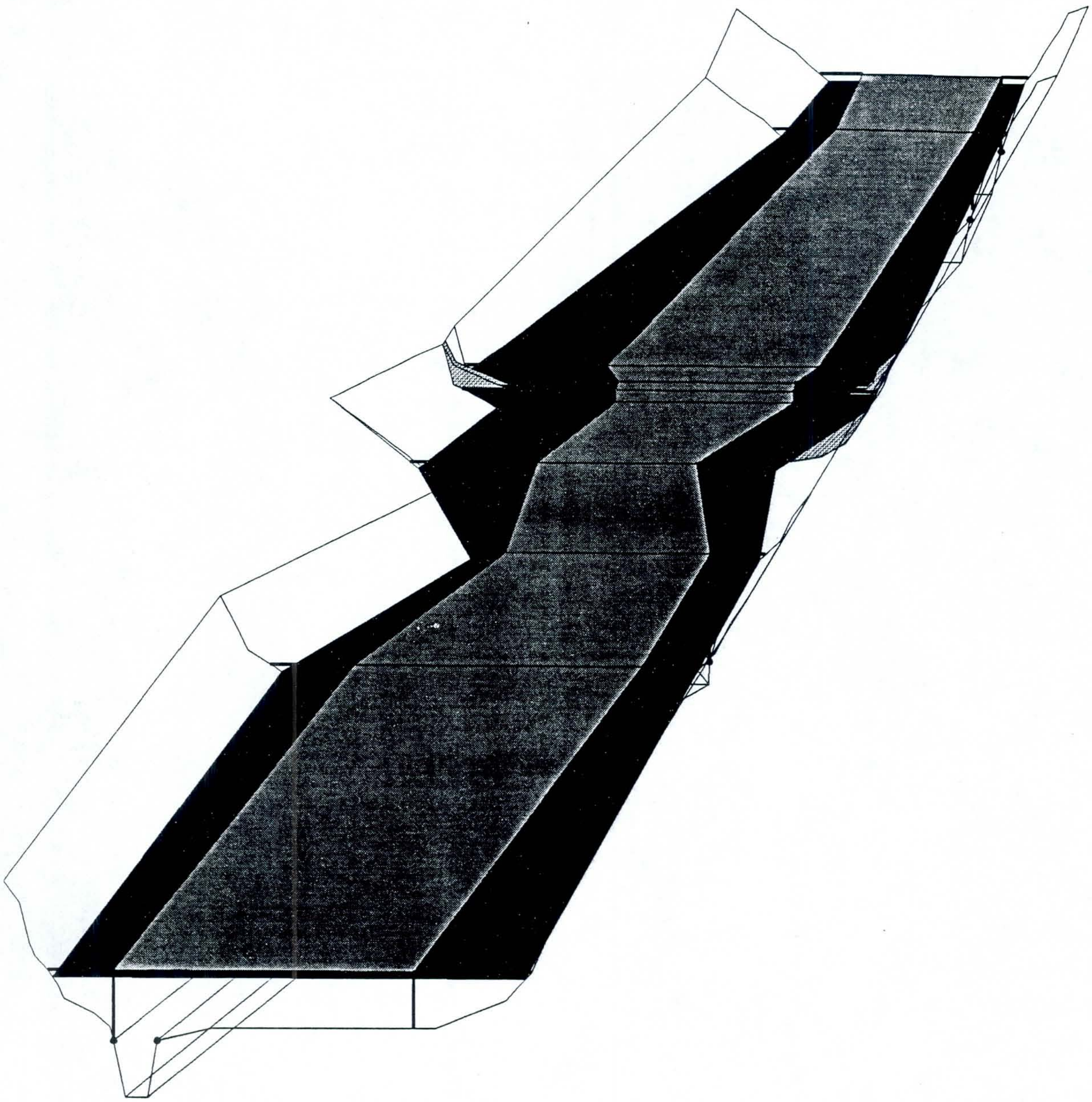

Figure 4, Floodplain and Floodway Example Using HEC-RAS

G. Development of the "Natural Condition" Model

The floodway procedure is based on calculating a natural profile (existing conditions geometry) as the first profile in a multiple profile run.

Other profiles, in a run, are calculated using various encroachment options within a hydraulic model.

Before performing an encroachment analysis, the user should have developed a model of the existing river system.

This model should be calibrated to the fullest extent that is possible.

Verification that the model is adequately modeling the river system is an extremely important step before attempting to perform an encroachment analysis.

H. Suggested Steps for Floodway Design

- Step 1

  Coordinate with all interested agencies, especially the local government, to determine the conditions that dictate floodways that vary from the standard.

- Step 2

  Run the natural flood profiles through the study reach.

- Step 3

  a. By inspection of the natural profiles, consider eliminating from further analysis those reaches where higher order floods would likely result in loss of life or catastrophic damages if fringe areas were developed.

  b. Eliminate from further analysis, those reaches where the local, Federal, or state requirements forbid encroachment. Coordinate this with the appropriate GTM for Flood Insurance Studies.

- Step 4

    a. Using HEC-2 or HEC-RAS (Method 4, 5/6), run the floodway (1.00 foot surcharge and equal conveyance) for the remaining reaches.

    b. Check for excessive velocities and adjust the width accordingly.

    c. Check for excessive surcharge (in excess of 1.00 foot).

    d. Adjust the floodway to obtain a smooth alignment.

    e. Adjust the floodway to meet local requirements.

    f. Adjust the floodway so that it is implementable (uniform width, etc. along short stream reaches.

    g. Adjust the floodway to meet local minimum width requirements.

- Step 5

    a. Rerun the floodway using HEC-2 or HEC-RAS (Method 1) based on adjustments from Step 4.

    b. Repeat Step 4, if necessary.

- Step 6

    Present the floodway to all interested agencies and make additional adjustments, if necessary.

ALLUVIAL FANS

Lecture 16

ALLUVIAL FANS

Lecture 16

Alluvial Fans

FORMATION OF AN "IDEALIZED" ALLUVIAL FAN

Adapted for Flood Control District short course by D. Williams, Ref. 1

1. Streamflow from intense rainstorms emanates from the confined channel of a mountain canyon and proceeds onto the relatively flat valley below.

   The canyon outlet forms the APEX of the fan, which represents the point of highest elevation on the fan.

2. Flow leaving the apex spreads onto the uppermost portion of the alluvial fan surface via a single high-velocity channel.

   This singular channel will either follow a pre-existing path cut from past flood events, possibly deepening the channel in a process called entrenchment, or cut a new path downslope.

   Flood hazards in this CHANNELIZED ZONE of the upper fan region can be severe due to the high velocity of flow, the presence of debris from the watershed, and the unpredictable location of flowpaths.

3. As the single channel flow encounters the flatter slope of the mid-fan area, it widens and becomes shallower, losing velocity and depositing sediment and debris.

   Materials that become deposited into previously-cut channels can backfill the old streambeds, leading to the abrupt development of new channels in a process called avulsion.

   The erosion/deposition processes include channel braiding, where singular flows split and rejoin as channels are alternately cut and filled with sediment.

These BRAIDED ZONE processes occur erratically, creating random, unpredictable flow patterns.

4. Toward the base of the fan, called the TOE, water velocities are further reduced as the fan surface becomes more uniform, its slope flattens and water infiltrates the soil surface.

   In this portion of the fan, SHEET FLOW (shallow, overland flow) is common, though flow velocities may remain high.

   Adjacent fans which have formed along mountain fronts tend to converge near their bases, producing alluvial APRONS, or zones of coalescence.

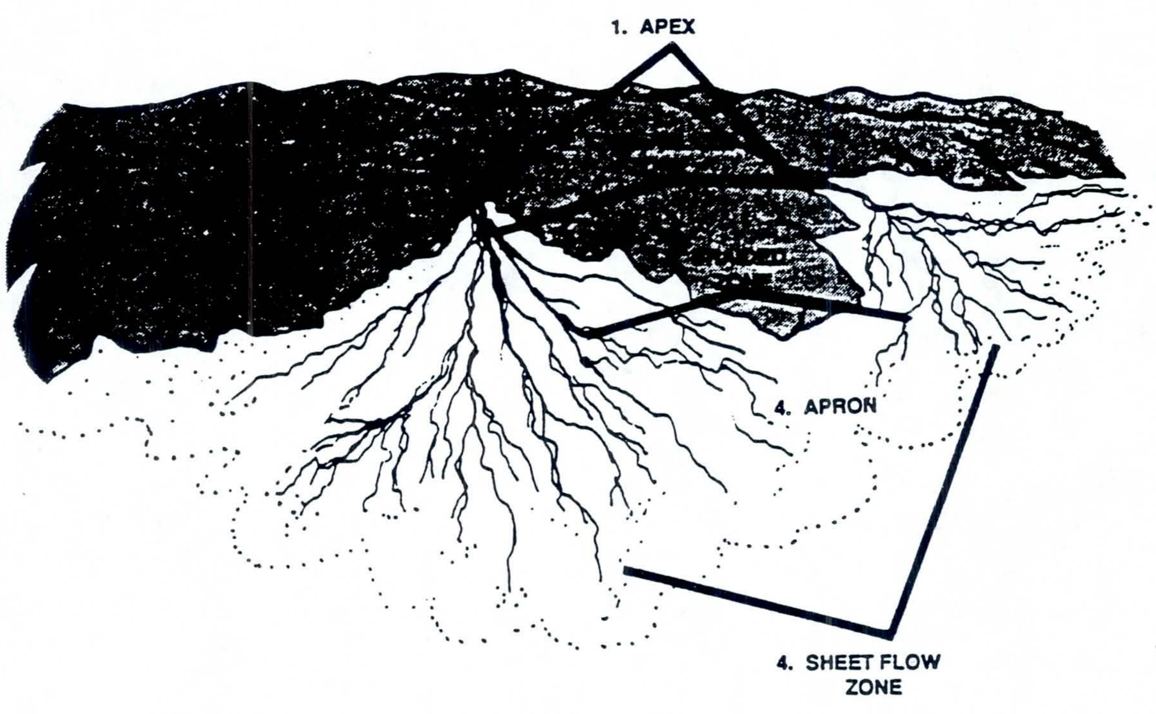

Figure 3

PROBLEMS ASSOCIATED WITH DESIGN OF CHANNELS ON ALLUVIAL FANS

Adapted for Flood Control District short course by D. Williams, Ref. 7

1. Assessing the Stability of the Alluvial Fan

   In considering the location and alignment of flood control channels, it is important to determine whether the fan is actively aggrading or whether it is in a stable or degrading state geomorphologically.

   If the fan surface is generally unvegetated and the principal channel spills easily and is "perched" in relation to ground at equal distances from the apex (Figure 6-5), the fan is likely to be actively aggrading.

   On the other hand, if the surface is generally well vegetated between channels and the main channel is well incised, the fan may be stable or even degrading.

2. Locating Flood Control Features on Aggrading Alluvial Fans

   On aggrading fans, developments requiring flood protection should often be discouraged because expensive flood control structures and ever-increasing maintenance may be required to keep the flow in the existing main channel or channels as their bed levels build up with deposited bed material.

   If the existing main channel is perched, it may be preferable to select a lower initial route or fall line for the flood control channel.

   It should be recognized that selected routes may not be maintainable indefinitely because of constraints on maintenance, especially during flood events, and because on some fans, the risk of catastrophic flood-debris events can be much more severe than previously observed floods.

   If development proceeds with recognition of risks, consideration may be given to sediment control features including debris basins and concrete linings.

On an alluvial fan, a debris basin would normally be located at the head of the fan, unless the main sediment supply is located farther downstream (Figure 6-6).

3. Locating Flood Control Features on Stable or Degrading Alluvial Fans

   On stable or degrading fans, problems of alignment and planform are essentially those of multichannel streams.

   In some cases it may be desirable to construct levees along the route of the main channel, closing off secondary channels or retaining them as escape routes for spills at designated low points in the levee system.

4. Alluvial Fans Adjacent to Each Other

   In some places where development has occurred on closely adjacent alluvial fans (piedmonts or bajadas) all issuing from the same mountain range, cross-slope interceptor channels have been used to pick up flows from a series of fans and lead them to the main channels.

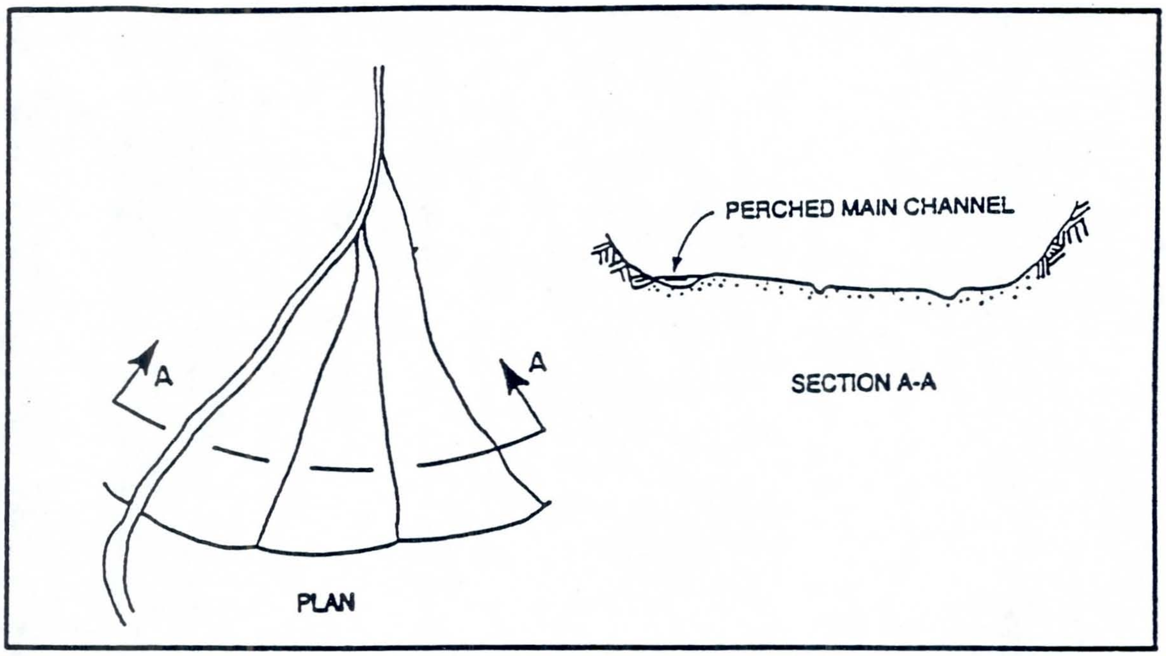

Figure 6-5. Perched channel on aggrading alluvial fan

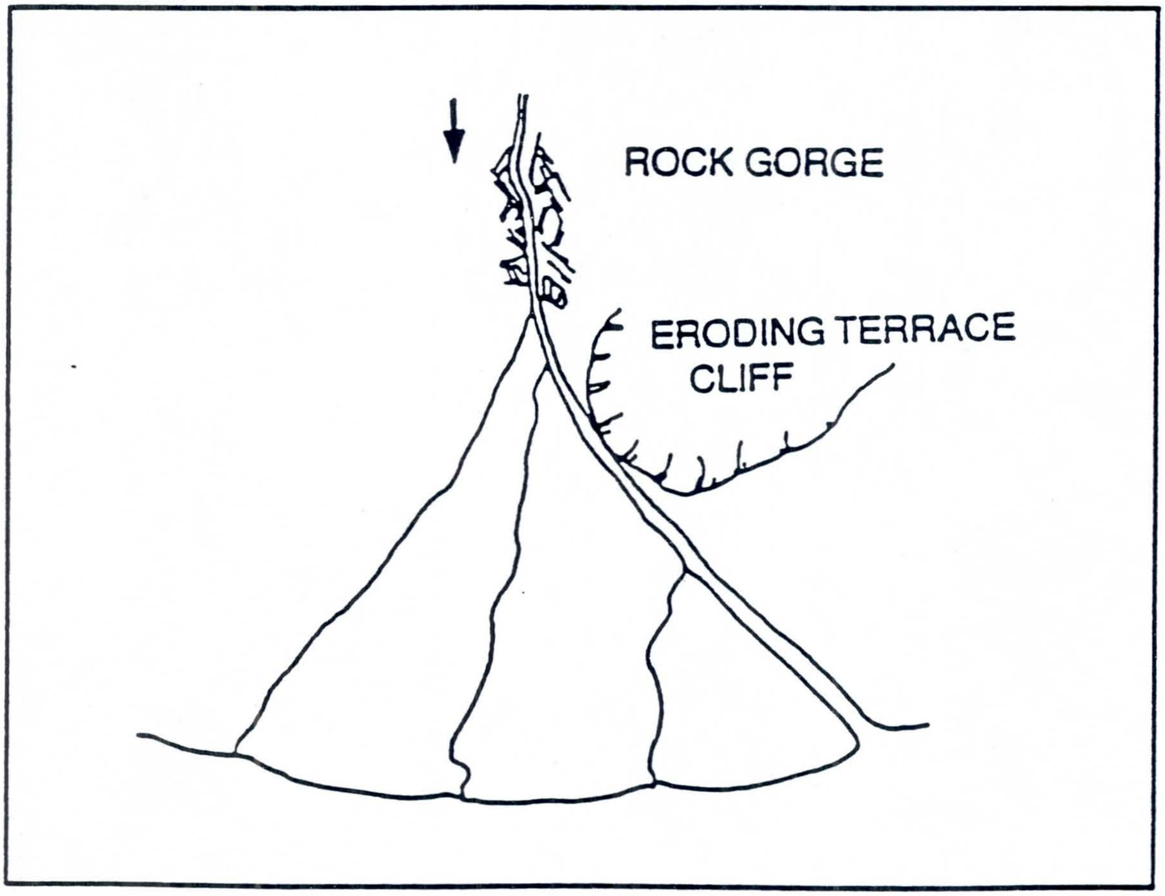

Figure 6-6. Principal active source of fan bed load may be downstream of apex

How Does the NFIP Address Development on Alluvial Fans

Adapted for Flood Control District short course by D. Williams, Ref. 1

1. FEMA Zones on FIRM Maps

The NFIP identifies alluvial fan hazards on FIRMs as Zone AO and provides information on flood depths and velocities.

AO zones are Special Flood Hazards Areas (SFHA) subject to inundation by 100-year sheet-type flow, which are sometimes associated with high velocities.

If the community's FIRM identifies AO zones with depths and velocities, construction within those alluvial fan areas are subject to certain regulations (in addition to those which apply to all SFHA's) found in Chapter 44 of the Code of Federal Regulations, Part 60.3:

A. Elevate lowest floor (including basement) above the highest adjacent grade to at least as high as the depth number specified on the FIRM.

B. It is recommended, however, that the depth of flow assumed for a particular sire should take into consideration local topographic anomalies when determining the elevation of any flood protection measure.

C. Mechanical and utility equipment must also be placed above the depth of flooding.

D. Provide adequate drainage paths around structures on slopes, to guide floodwater around and away from proposed structures.

E. Do no deflect floodflow onto adjacent properties.

2. FEMA Map Revision Process on Alluvial Fans

As part of the FIRM revision process, FEMA will review the development plans submitted by owners of projects who request the removal of their property from the SFHA.

To ensure that these projects are in fact protected from alluvial fan flood hazards, FEMA's review criteria require that the construction include elements which:

A. do not cause the disturbance of natural flood processes on the fan

B. allow for the safe collection, passage, and disposal of flood-related water, sediment and debris without negative impact to adjacent property

C. address erosion, scour, deposition, impact and hydrostatic forces

D. provide that the design and maintenance of project elements be coordinated with the local jurisdiction and/or agency responsible for flood control within the community

3. Powers of Local Jurisdictions

With knowledge of local conditions and in the interest of safety, state and community officials may set higher standards for construction in floodplain areas.

As with all flooding situations, FEMA encourages local jurisdictions to adopt floodplain management measures which are more tailored to the community's particular flood problems.

This is especially important for communities prone to alluvial fan flood hazards, where each fan presents a unique set of flood conditions.

4. Minimum Requirements

It should be noted that the provisions of Section 60.3 are minimum requirements; buildings constructed according to these rules alone will not provide adequate protection against high velocities or debris loads unless additional measures are undertaken.

FIELD RECONNAISANCE

OF A LOCAL STREAM

Lecture 17

FIELD RECONNAISSANCE
OF A LOCAL STREAM

Adapted for Flood Control District for short course be D. Williams, Ref. 9

OBJECTIVE: To make the student aware of the steps that are necessary to prepare for a field trip and the tasks and observations that should be made during the field trip.

I. Preparation for Field Reconnaissance

A. Program funds, etc. for at least one follow-up reconnaissance of the stream.

B. Prior to the actual field trip, an investigation of data readily available in the office should be conducted.

Knowledge of various historical, hydraulic and sediment parameters will make the field investigation easier and more efficient.

II. Suggested Field Reconnaissance Tasks and Observations

A. Photographs (bed, banks, existing projects, lateral inflows/ outflows, significant failures).

Note where you took photograph and take follow-up photographs at or near that spot.

B. Verify topographic maps, aerial photographs, surveys (cross-sections of channel and overbanks and bed profile to find evidence of head-cuttings).

C. Note boundary conditions

D. Note whether channel is straight, meandering, or braided.

E. Note low flow channel and crossings between bars.

F. Note slope of stream in general and any break points.

G. Sample bed material.

Check underlying material (e.g., shallow layer of erodible material over unerodible material or a channel incised in clay valley with sand just underneath).

H. Note condition, slope, and height of banks, whether stable, caving, cropping along top presence of erodible lenses, vegetation, trees, fences.

Might drive stakes in overbank near channel to measure rate of erosion follow-up visit by you or others.

I. Estimate the percent of the bed that is naturally armored.

J. Note problem areas and attempt to ascertain the cause(s).

K. Note changes in bed gradation.

L. Note channel mining activities.

M. Note traffic on waterway and any eroding mooring spots.

N. Note tributary entry points, the amount of flow, turbidity of flow, condition of the tributary.

O. Note diversion points (e.g., irrigation pumps or channels, levee flanking).

P. Note natural grade controls such as rock outcrops.

Q. Note presence of protection measures (e.g., riprap, grade control structures, dikes, etc.), their site, why and when they were placed, condition, tie-in upstream downstream project limits to natural channel.

R. Note location of gage(s) and types.

S. Note structural feature and obstacle locations (e.g., bridges, docks, fallen trees) and observe bank and bed conditions in the vicinity of the structures.

T. Note existing similar projects on same of adjacent streams.

Note how they are performing.

U. Note overbank conditions - areas of scour or deposition - if deposition exists, obtain samples and measure depth and note extent on map.

V. Take velocity measurements at several locations if none are available from other sources.

Surface velocity can be estimated by timing the movement of a floating object over a known distance.

W. Talk with locals to identify problem areas and get an estimate of the time of origin of any problem areas.

Also, inquire as to local land use history - when urbanized, cleared, etc.

III. Post Reconnaissance Activities

A. Based on the data available in the office and observations/data/information collected during the reconnaissance, the engineer should be able to ascertain the following:

1. Present stability of stream

2. Adequacy of present structural features (e.g., bridges, bank and bed protection).

3. Adequacy of past channel improvements and/or alignment charges.

B. Depending on the availability of relatively long-term, historic data, the engineer may be able to ascertain the following:

1. Long-term stability trends

2. Stream response to land use changes

3. Stream response to previous improvements

C. Depending on the availability of historic and contemporary hydraulic, hydrologic, topographic and sediment data, the engineer should be able either qualitatively or quantitatively, to evaluate:

1. Future long term stability with and without the proposed improvement.

2. Future maintenance requirements with and without the project.

3. Design alternatives that address the interaction of all project considerations in order to arrive at the "best" design.

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Lecture 18

BIBLIOGRAPHY

Lecture 18

BIBLIOGRAPHY

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GLOSSARY

Lecture 19

Glossary

1/2 PMF: The flood hydrograph with ordinates equal to one-half the corresponding ordinates of the Probable Maximum Flood Hydrograph.

100-Year Flood: A flood stage or height that, statistically, has one percent chance of being equaled or exceeded in any given year. The 100-year flood is often referred to as the base flood.

Abutments: Walls supporting the end of a bridge or span, and sustaining the pressure of the abutting earth. In a drop structure, the walls which form the sides of the crest of the drop. In some structures, wingwalls (transition walls) extend upstream of the abutment walls to create a smooth transition from the upstream channel.

Aggradation: A progressive buildup or raising of the channel bed due to sediment deposition. Permanent or continuous aggradation is an indicator that a change in the stream's discharge and sediment load characteristics is taking place, see Degradation.

Alluvium: Unconsolidated material deposited by a stream in a channel, floodplain, alluvial fan, or delta.

Armor: Surfacing of channel bed, banks, or embankment slope to resist erosion.

Armoring: (a) Natural process whereby an erosion-resistant layer of relatively large particles is formed on a streambank due to the removal of finer particles by streamflow. (b) Placement of a covering on a streambank to prevent erosion.

Arterial Street System: The arterial system should carry a major portion of trips entering and leaving the urban area, as well as the majority of movements through the central city. Frequently, the arterial system will carry important intra-urban as well as intercity bus routes. Arterials are typically located on one-mile intervals on section lines.

Baffle Chute: A type of drop structure or outlet structure that incorporates baffles for energy dissipation.

Baffles: Deflector vanes, blocks, guides, grids, gratings or similar devices constructed to: 1) check or effect a more uniform distribution of velocities; 2) dissipate energy; 3) divert, guide, or agitate flow; and 4) check eddy currents.

Basin Area: The area which contributes stormwater to a concentration point such as a lake, stream, or drainage system. See Watershed.

Basin Floor: The bottom of a stormwater retention facility which has been specifically designed for the purpose of disposing stored runoff following a storm event by the process of infiltration into the subsurface.

Basin Sediment Yield: The total sediment outflow from a watershed or a drainage area at a point or reference and in a specified time period. This outflow is equal to the sediment discharge from the drainage area.

Bed Material: Material found on the bed of a stream (may be transported as bed load or in suspension).

Bed Sediment Discharge: The part of the total sediment discharge that is composed of grain sizes found in the bed and is equal to the transport capability of the flow.

Braided Stream: A stream whose flow is divided at normal stage by small mid-channel bars or small islands; the individual width of bars and islands is less than about three times the water width; a braided stream has the aspect of a single large channel within which are subordinate channels.

Bridge Low-chord: The elevation of the lowest portion of the bridge deck structure used in determining the area of the bridge opening available for flow conveyance.

Catch Basin: A chamber or well, usually built at the curb line of a street, for the admission of surface water to a storm sewer or sub-drain.

Channel Failure: Sudden collapse of a channel due to an unstable condition, such as the removal of a bank by scour.

Channel Reach: A segment of stream length that is arbitrarily bounded for purposes of study.

Channel Stabilization: Methods of achieving slope and cross-section which allow a channel to transport the water and sediment delivered from the upstream watershed without aggradation or streambank erosion.

Check Dam: A low dam or weir across a channel,for the diversion of irrigation. Also used herein for a low dam to control stream gradient, typically associated with small streams or the low flow channel of a floodplain or other channel.

Check Structure: A small drop structure constructed in the low flow portion of a channel for the purpose of controlling stream gradient.

Clear Zone: The roadside border area, starting at the edge of the traveled way, available for safe use by errant vehicles.

Clear-water Scour: Scour which occurs when there is no movement of the bed material of the stream upstream of the crossing, but occurs as a result of acceleration of the flow and vortices created by piers or abutments causing material at their base to move.

Collector Street System: Collector streets may penetrate neighborhoods and may carry a minor amount of through traffic.

Contraction Scour: General scour resulting from the acceleration of flow due to a natural channel constriction or bridge contraction.

Crest: That portion of the drop structure which controls the gradient of the upstream channel. In a vertical drop structure the crest is a wall typically constructed of reinforced concrete or sheet pile. In a sloping drop structure, the crest is the portion of the drop at the top of the slope and usually incorporates a buried cutoff wall for seepage control.

Critical Depth: The depth at which a given discharge flows in a given channel with a minimum specific energy. For depths greater and lower than critical, the flow is said to be subcritical and supercritical, respectively.

Critical Flow: Flow at critical depth.

Culvert: A hydraulically short conduit which conveys surface water runoff through a roadway embankment or through some other type of flow obstruction. Culverts are constructed from a variety of materials and are available in many different shapes and configurations. Culvert selection factors include roadway profiles, channel characteristics, flood damage evaluations, construction and maintenance costs, and estimates of service life.

Degradation: A progressive lowering of the channel bed due to scour. Permanent or continuing degradation is an indicator that a change in the stream's discharge and sediment load characteristics is taking place, see Aggradation.

Design Discharge: Maximum flow a structure or channel is expected to accommodate without contradicting the adopted design constraints.

Detention Basin: A basin or reservoir where water is stored for regulating a flood. It has gravity-flow outlets for outflows during floods.

Design Frequency: The nth-year storm for which it is expected that the structure or facility designed for that storm would experience an actual hydrological event of a given or greater magnitude, once, on average, in n years. For example, a 50-year storm has a 2 percent chance of occurring in any given year. Also called the return period, excedence interval, or recurrence interval.

Discharge: Volume of water passing through a channel during a given time.

Drainage Basin: A geographical area which contributes surface runoff to a particular concentration point. The terms "drainage basin", "tributary area" and "watershed" are used interchangeably.

Drainageway: A route or watercourse along which storm runoff moves, or may move, to drain a catchment area.

Drop Structure: A structure constructed in a conduit, canal, or open channel for the purpose of gradient (bottom slope) control.

Dry Well: An engineered subsurface chamber designed to accept surface runoff and allow it to drain into the subsurface strata.

Embankment: A man-made earth fill structure constructed for the purpose of impounding water.

Emergency Spillway: An outflow spillway from a stormwater detention/retention facility that provides for the safe overflow of floodwaters for storm events in excess of the design capacity of the Primary Outlet Structure, or in the event of malfunction or debris blockage of the Primary Outlet Structure.

Energy Grade Line (EGL): An inclined line representing the total energy of the flowing water. For an open channel, the EGL is above the **water surface** by a value of the velocity head. In a closed pressure conduit, the EGL is above the **pressure head line** by a value of the velocity head. See Hydraulic Grade Line and Figure 4.3.

Equilibrium: The state of balance of natural channels between hydraulic forces or actions. Equilibrium occurs when the streambed has achieved a graded condition when the slope and energy of the stream are just sufficient to transport material delivered to it. Natural channels which have small changes resulting from periods of low and high flows are considered in equilibrium.

Erosion: Displacement of soil particles on the land surface due to water or wind action.

Filter: Layer of fabric, sand, gravel, or graded rock placed (or developed naturally where suitable in-place materials exist), between the bank revetment and soil for one or more of three purposes: 1) to prevent the soil from moving through the revetment by piping, extrusion, or erosion; 2) to prevent the revetment from sinking into the soil; and 3) to permit natural seepage from the streambank, thus preventing buildup of excessive hydrostatic pressure.

Filter Blanket: A layer of graded, intermediate-size gravel placed between fine-grained material and riprap, to prevent wash-out of the finer material.

Filter Fabric: Fabric of synthetic strands that serves the same purpose as granular filter blanket.

Fine Sediment Load (or Washload): That part of the total sediment load that is composed of particle sizes finer than those represented in the bed. Normally, the fine-sediment load is finer than 0.062 mm for a sand-bed channel. Silt, clay, and sand could be considered fine sediment load in a coarse gravel and cobble bed channel. The washload generally comes from the watershed.

Flood Fringe: A regulatory district within the floodplain but outside the floodway district.

Flood Peak: The largest value of the runoff flow which occurs during a flood event, as observed at a particular point in the drainage basin.

Flood Routing: The mathematical simulation of a flood wave as it moves downstream along a watercourse or through a detention/retention facility.

Floodplain: A flood-prone area of land adjoining or near the channel of a watercourse which have been, or may be, covered by floodwaters. A floodplain functions as a temporary channel or reservoir for overbank flows.

Floodway: A specific regulatory district within the floodplain as identified on FEMA flood hazard boundary maps; or the channel of a river or other watercourse and the adjacent land area necessary to discharge the 100-year flood without cumulatively increasing the water surface by more than one foot and without creating hazardous velocities of floodwaters.

Freeboard: The vertical distance above a design water surface elevation that is provided as a contingency or allowance for waves, surges, water-borne debris or other factors.

Froude Number: A dimensionless number (expressed as $V/(gy)^{0.5}$) that represents the ratio of inertial to gravitational forces. High Froude numbers (values greater than 1) indicate supercritical flow with associated high velocity and scour potential.

Gabion or Wire-Enclosed Basket: A basket or compartmented rectangular container made of steel wire mesh. When filled with cobbles or rock of suitable size, the gabion becomes a flexible and permeable block with which flow-control structures can be built.

General Scour: Scour in a channel or on a floodplain that is not localized at a pier, abutment, or other obstruction to flow. In a channel, general scour usually affects all or most of the channel width.

Geomorphology: That branch of both physiography and geology that deals with the form of the earth, the general configuration of its surface, and the changes that take place due to erosion of the primary elements and in the buildup of erosional debris.

Grade Control Structure (sill, check dam): A structure across a stream channel placed bank to bank (usually with its central axis perpendicular to flow) to control bed slope and prevent scour or headcutting.

Gradient: The rate of change of a characteristic per unit of length. The term is usually applied to such things as channel/stream bed slope elevation, conduit invert elevation, velocity, pressure, etc.

Guide Bank: A dike extending upstream from the approach embankment at either or both sides of the bridge opening to direct the flow through the opening. Some guide banks extend downstream from the bridge.

Gunite: Term formerly used for dry-mix mortar shotcrete.

Headcutting: Channel bottom erosion moving upstream along a waterway indicating that a readjustment of the channel's slope and its discharge and sediment load characteristics is taking place. Headcutting is evidenced by the presence of abrupt vertical drops in the stream bottom or rapidly moving water through an otherwise placid stream. Headcutting often leaves stream banks in an unstable condition as it progresses along the channel.

Hydraulic Grade Line (HGL): For an open channel, it is coincident with the water surface. In a closed pressure conduit, it is the line representing the pressure head of the conduit. HGL will always be EGL minus the velocity head. See Energy Grade Line and Figure 4.4.

Hydraulic Jump: The hydraulic jump is an abrupt rise in the water surface which occurs in an open channel when water flowing at supercritical velocity is retarded by water flowing at subcritical velocity or a stationary pool. The transition through the jump results in a marked change in energy, evidenced by turbulence of the flow within the area of the jump. The hydraulic jump is often used as a means of energy dissipation.

Hydraulic Structures: The facilities used to impound, accommodate, convey or control the flow of water, such as dams, weirs, intakes, culverts, channels, and bridges.

Hydrograph: The functional relationship between time and flow discharge, as observed at a particular point within a drainage basin. In the case of a detention/retention facility, an Inflow Hydrograph depicts the relationship of time and runoff inflow to the facility, and an Outflow Hydrograph is a graph of flow discharge from the facility versus time.

Impervious: A term applied to a material through which water cannot pass, or through which water passes with great difficulty.

Incised Stream: A stream that flows in an incised channel with high banks. Stream banks that stand more than 15 feet above the water surface at normal stage are regarded as high banks.

Infiltration: The movement of water into and through the soil.

Invert: The lowest point in the channel cross section or at flow control devices such as drop structures, dams, or outlet structures, see Thalweg.

Jurisdiction or Jurisdictional Agency: Maricopa County, the Flood Control District of Maricopa County, and the incorporated municipalities within Maricopa County.

Lateral Stream Migration: Change in position of a channel by lateral erosion of one bank and simultaneous accretion of the opposite bank. Movement in which the material has a dominate lateral component.

Launching: Release of undercut material (stone riprap, rubble, slag, etc.) downslope; if sufficient material accumulates on the streambank face, the slope can become effectively armored.

Live-bed Scour: Scour which occurs when the bed material upstream of the crossing is also moving.

Local Aggradation: Aggradation in a channel or on a floodplain that is localized at a pier, abutment, or other obstruction to flow.

Local Scour: Scour in a channel or on a floodplain that is localized at a pier, abutment, or other obstruction to flow.

Local Street System: The local street system comprises all facilities not on one of the higher systems. It offers the lowest level of mobility and usually contains no bus routes. Service to through traffic movement usually is deliberately discouraged.

Low Flow Channel: A channel within a larger channel which typically carries low and/or normal flows.

Major drains: Include natural and man-made channels and conduits that serve watershed areas from 160 acres to about 10 square miles.

Master Planning: A "systems" approach to the planning of facilities, programs and management organizations for comprehensive control and use of stormwater within a defined geographical area or drainage basin.

Meandering Channel: A channel exhibiting a characteristic process of bank erosion and point bar deposition associated with systematically shifting meanders.

Median Diameter: The particle diameter at the 50 percentile point on a size distribution curve such that half of the particles (by weight for samples of sand, silt or clay and by actual measurement for samples of gravel and riprap) are larger and half are smaller. The median diameter is denoted D_{50}.

Minor drains: Serve watershed areas up to 160 acres and are normally the drains associated with subdivision development.

Multi-purpose Facility: A detention or retention facility that provides benefits in addition to the primary function of flood control. Such benefits may include recreation, parking, visual buffers or water harvesting.

Nappe: The sheet or curtain of water overflowing a weir or dam. When freely overflowing the crest of a structure, it usually has a well-defined upper and lower surface.

Off-stream Detention/Retention Facility: A facility that is located near or adjacent to a watercourse (i.e., the stream does not flow directly into the facility). Inflow to the facility is typically accomplished by means of side weirs. It is also referred to as an Off-line Detention/Retention Facility.

On-site Detention/Retention: The temporary storage of excess storm runoff in the upper area of a drainage basin. This type of facility is typically within a subdivision, primarily by an individual development and generally irrespective of watershed features.

On-stream Detention/Retention Facility: A facility that is located within the path of a stream or watercourse, and thereby intercepts the entire flow from the upstream drainage basin. It is also referred to as an On-line Detention/Retention Facility.

Orifice: A hole in the outlet structure of a stormwater storage facility sized to drain the facility at a specific rate of flow.

Outlet Structure: A hydraulic structure placed at the outlet of a conduit, open channel, spillway, etc., for the purpose of dissipating energy and providing a transition to the channel or conduit downstream. Outlet structures may consist of culverts, weirs, orifices (gated or un-gated), dry wells, or any combination thereof.

Plunge Pool: An energy dissipation device placed downstream of a conduit, channel or vertical wall drop structure. The plunge pool basin is typically lined with rock riprap, concrete or other protective covering and dissipates the energy of free falling water through impact and turbulence.

Pressure Head: In a closed pressure conduit, it represents the energy per unit weight stored in the fluid by virtue of the fluid being under pressure expressed as P/γ. Generally having the units of feet. In an open channel, the pressure head is zero.

Primary Outlet Structure: Also known as the Primary Spillway or Principal Spillway, it is the main outlet structure by which stormwater is discharged from the detention/retention facility.

Probable Maximum Flood (PMF): The flood runoff that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region.

Pump Station: A facility housing stormwater pumps, controls, power plants and their appurtenances.

Regional Detention/Retention: The temporary storage of excess runoff by means of large storage facilities located at strategic sites within a drainage basin. Sites are generally planned to provide control of excess runoff from an entire drainage basin with an optimum (presumably a minimum) number of storage facilities to achieve the most cost-effective drainage system. Regional detention/retention sites are normally maintained by a public or quasi-public agency.

Regional drains: The main outfalls for drainage. They serve watershed areas generally greater than 10 square miles, and include rivers and washes.

Residual Freeboard: For an embankment dam, the vertical distance between the maximum water surface elevation and the minimum dam crest elevation.

Retention Basin: A basin or reservoir wherein water is stored for regulating a flood, however, it does not have gravity-flow outlets for outflows during floods as detention basins do. The stored water must be disposed by some other means such as by infiltration into soil, evaporation, injection (or dry) wells, or pumping systems.

Reverse Filter Drain: An engineered granular filter placed at weep hole locations on hydraulic structures to collect and direct groundwater to the weep holes to relieve uplift pressures and other adverse effects of uncontrolled seepage water.

Riprap Toe Protection: In the restricted sense, layer or facing or broken rock or concrete dumped or placed at the toe of a channel to protect a structure or embankment from erosion; also the broken rock or concrete suitable for such use. Riprap has also been applied to almost all kinds of armor, including wire-enclosed riprap, grouted riprap, sacked concrete, and concrete slabs.

Runoff: The portion of precipitation on land that ultimately reaches streams; especially water from rain or melted snow that flows over the ground surface.

Scour: Erosion due to flowing water, usually considered as being localized as opposed to general bed degradation.

Sediment (or Fluvial Sediment): Fragmental material transported, suspended, or deposited by water.

Sediment Discharge: The quantity of sediment that is carried past any cross section of a stream in a unit of time. Discharge may be limited to certain sizes of sediment or to a specific part of the cross section.

Sediment Trap: An area within a stormwater detention/retention facility which is designed to trap the majority of incoming sediments for the purpose of facilitating maintenance.

Seepage: The movement of water through pores and voids of pervious material such as soil, gravel, synthetic filter media, etc.

Seepage Cutoff Wall: An impervious subsurface barrier constructed of clay, concrete or synthetic material for the purpose of increasing the length of travel of subsurface water flow and thereby reducing and/or controlling the action of such flows (for example, uplift forces) at hydraulic structures.

Shotcrete: Mortar or concrete pneumatically projected at high velocities onto a surface.

Sill: A raised edge at the downstream end of a stilling basin. The sill typically has a notch or opening to allow normal stream flows to pass through and/or to allow the basin to drain completely following a storm.

Slope Paving: Covering of a channel bank or bed with stones or concrete.

Soil-Cement: A designed mixture of soil and portland cement compacted at a proper water content to form a veneer or structure which when placed on a streambed or bank can prevent erosion. Also referred to as Cement Stabilized Alluvium.

Spillthrough Abutment: A bridge abutment having a fill slope on the streamward side.

Spillway: (a) A low-level passage serving a dam or reservoir through which surplus water may be discharged; usually an open ditch around the end of a dam, or a gateway or a pipe in a dam. (b) An outlet pipe, flume, or channel serving to discharge water from a ditch, ditch check, gutter or embankment protector.

Stage: The depth of water within a stormwater storage facility, as measured above an established datum.

Storage Reservoir of Pump Station: A reservoir wherein peak flows from storm drains are stored for reducing capacity requirements of the pump station to pump runoff to an appropriate outlet.

Storm Drainage System: A drainage system for collecting runoff of stormwater on highways and removing it to appropriate outlets. The system includes inlets, catch basins, storm sewers, main drains, storage reservoirs, detention basins and pump stations.

Stormwater Detention Facility: A stormwater storage facility which temporarily stores surface runoff and releases it at a controlled rate through a positive outlet.

Stormwater Retention Facility: A stormwater storage facility which stores surface runoff. Stored water is infiltrated into the subsurface or released to the downstream drainage system or watercourse (via a gravity outlet or pump) after the storm event.

Streambank Erosion: Removal of soil particles from a bank surface due primarily to water action. Other factors such as weathering, ice and debris abrasion, chemical reactions, and land use changes may also directly or indirectly lead to streambank erosion.

Streambank Protection: Any technique used to prevent erosion or failure of a streambank.

Subdrain: An underground conduit, usually perforated and surrounded by an engineered granular filter material that is designed to permit infiltration for the purpose of collecting and conveying groundwater.

Subgrade Erosion: Erosion of the material underlying that portion of the stream bed which is subject to direct action of the flow.

Subsurface Disposal: Drainage of stormwater runoff into the subsurface by the process of infiltration. This is typically accomplished through the use of dry wells, engineered basin floors, etc.

Tailwater: The water surface elevation in the channel downstream of a hydraulic structure.

Thalweg: The line extending down a channel that follows the lowest elevation of the bed, see Invert. Not to be confused with the channels's centerline.

Total Freeboard: For an embankment dam, the vertical distance between the emergency spillway crest and the minimum crest elevation of the dam.

Total Sediment Discharge: The sum of suspended sediment discharge and bedload discharge or the sum of bed material discharge and washload discharge of a stream.

Transport Rate: Rate at which sediment particles are carried when hydraulic conditions exceed the critical condition for motion. Transport rates are calculated analytically by the use of transport functions.

Trash Rack: A metal bar or grate structure designed to prevent blockage of the structure by water-borne debris.

Trickle Channel: Also called the low flow channel, the trickle channel is that portion of a major channel which is sized to carry the normal low flow.

Underdrain: See Subdrain.

Uniform Flow: Flow of constant water area, depth, discharge, and average velocity through a reach of a channel.

Uplift Pressure: Pressure caused by uncontrolled seepage or groundwater flow beneath a structural slab which can lead to cracking and displacement of the structure.

Velocity Head: Represents the kinetic energy of the flowing fluid generally expressed as $V^2/2g$ in feet, but actually is the energy per pound of flowing fluid.

Vortex: Local current accelerations which cause a whirling or circular motion that tends to form a cavity or vacuum at its center, thus moving sediment toward the cavity.

Waters of the U.S.: All waters which are currently used, were used in the past, or may be susceptible to use in interstate or foreign commerce, including all waters which are subject to the ebb and flow of the tide.

Watershed: An area confined by drainage divides, often having only one outlet for discharge. See Basin Area.

Weep Hole: Openings in an impermeable wall or revetment to relieve the neutral stress or pore water pressure. Weep holes are typically combined with reverse filter drains to form a total system for seepage control.

Weir: A notch of regular form through which water flows. A weir may be a depression or notch in the side of an outlet structure or a depression of specific shape in the embankment of a stormwater storage facility. Classified in accordance with the shape of the notch, there are rectangular weirs, V-notch weirs, trapezoidal weirs and parabolic weirs.