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Factors Affecting the Stability of Structures Erected Along Water Courses : Applied Fluvial Geomorphology

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by

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FOREWORD

During the past several years, the author of this report has been earning an advanced degree in environmental engineering with the undersigned serving as his major advisor. The original study of fluvial morphology came to my attention about two years ago when James MacBroom presented me with a copy. I was intrigued by the scholarly yet practical approach to this topic and encouraged him to edit his paper into a report form in the belief that this would provide a reference work of great value to the engineering designer who must deal with river systems.

In preparing this for publication, Mr. MacBroom went far beyond the usual rewriting and editing which was expected, and in addition, submitted his text to his professional colleagues for use in practical engineering situations. The revised version thus retains the basic elements of the original study at the same time that it serves as a carefully reworked report which has met the test of technology transfer in water resources.

The Institute of Water Resources is very pleased to be able to publish this report which we believe will be a valuable addition to our series. The author is to be commended for this very timely and extremely useful volume.

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1.0 APPLIED FLUVIAL GEOMORPHOLOGY

1.1 INTRODUCTION

The planning and design of river stabilization and flood control projects is a difficult task because rivers are among the most dynamic elements of the earth's surface. They are constantly changing in size, form, and location, and can only be considered to be "stable" when such changes in river morphology reach a condition of dynamic equilibrium over a period of time. Rapid "unstable" changes in river morphology can be disasterous, causing flooding, economic losses, and the destruction of terrestrial and aquatic habitats.

It is important that the alteration of rivers be planned to increase the stability of the river, encourage ecological diversity, provide recreation and open space and be visually attractive in addition to having adequate hydraulic capacity. With these goals in mind, it is necessary for man to understand natural river patterns and processes and apply this information to projects involving the alteration of rivers and floodplains.

The first portion of this report discusses the geologic properties and characteristics of natural rivers and floodplains. The second part outlines the influence of man on fluvial geomorphology, ecological considerations, and the natural characteristics of rivers that should be applied in the design of river and bridge projects.

1.2 SUMMARY

The dynamic nature of rivers can, to a considerable degree, be understood and predicted. The constant changes in river geometry, size, location, and sediment loads can be anticipated by recognizing natural conditions and the response to outside influences. Laboratory tests and field data have been developed into quantitative, empirical relations and equations that can be used to predict (with varying

degrees of reliability) river slopes, depths, widths, patterns, sediment transport, scour conditions, and both short and long-term stability.

This information can be used in the planning and design of river projects when non-structural alternatives are not feasible. Open channels can and should be designed to enhance the river environment by using the principles of fluvial geomorphology, and need not create sterile or unattractive landscapes.

The following steps have been recommended in this report with respect to designing channel improvement compatible with natural, stable rivers:

a. Study the existing conditions at and adjacent to the project site to understand the river's existing equilibrium conditions.
b. Determine the river's natural channel pattern for the available slope and flow rates, and whether it is stable.
c. Adjust the river's bed slope as necessary to provide the desired channel pattern. This may be done by altering the river's length or pattern, or by providing chutes and drop structures to alter the river's grade.
d. The main flow channel should have a straight or meandering alignment depending on its slow and dominant discharge, with both conditions being more stable than a braided channel.
e. If the river's slope cannot be adjusted sufficiently for the desired channel pattern, then either the channel will need to be lined to resist erosion, or the sediment load will have to be controlled to prevent degradation or aggradation.
f. The main channel should normally be sized to convey the river's dominant discharge, usually a flood with an average return frequency of one to two years. The ratio between the width and depth should be based upon the regime data, tractive stress analysis, and sediment transport needs.
g. The infrequent peak flood flows can be conveyed on the floodplain or a corresponding man-made floodway. The use of a floodway, which is normally dry, allows use of a narrow channel which concentrates normal flows in the main channel. This will increase

the channel's dry weather flow depth and sediment transport capacity, and reduces peak flow velocities during floods to minimize scour and sediment transport.

h. The design of bridges should include detailed studies pertaining to their effect on the river, and to the effect of river bed scour on the bridge.

i. Stream side vegetation should be maintained wherever possible, or reestablished if disturbed, in order to encourage an early return of aquatic life.

j. In-stream structures suchas check dams, boulders, and low flow deflectors can be used to help provide micro-habitats suitable for diverse species within the larger channels.

The use of fluvial geomorphology concepts presented in this report can help reduce the adverse environmental impacts frequently associated with the alteration of natural rivers.

However, the application of the fluvial geomorphology techniques will not eliminate all environmental problems. Every effort should still be made to avoid disrupting the natural riverine conditions and we must still seek to preserve wetlands, watercourses, and floodplains by use of non-structural flood control measures and sound land use planning.

There is still a need for further research on the subject in order to clarify conflicting information and to reduce our dependence on empirical data. Particular attention needs to be given to developing universal sediment transport and stability criteria, and the use of a practical method of analyzing the hydraulics of rivers with mobile boundaries and variable bed forms.

2.0 RIVER CLASSIFICATION

This section will present various classification systems for rivers and streams based upon their physical characteristics. The classification systems are important as they allow one to readily describe a river for comparative purposes and they serve as an introduction to the various types of rivers. In addition, each river class will have different responses to outside influences and it is becoming possible to predict how each river class will react.

2.1 CHRONOLOGICAL CLASSES

Streams and rivers go through a distinct series of stages in terms of their geologic history. Any one river may have any combination of the three stages present along its length at any one time, and may revert back to a previous stage if outside events influence it (such as tectonic uplift or climate changes.) The description of the life stages presented below is that originally published by Davis in 1899, and has been reported and revised by many others since then. It is presented here as an introduction to the dynamic nature of streams and rivers with respect to time.

2.11 The youthful stream of concentrated water erodes a channel as its flow removes and conveys loose soil and rock particles. The steep slopes of upland areas allow water to flow at relatively high velocities and thus a youthful stream is characterized by channel erosion and a deepening of its bed. The youthful stream has an irregular profile with waterfalls, rapids, and lakes located along the river. These features are gradually reduced as higher points are eroded. Deep "V" shape valleys are formed with steep sides and narrow bottom widths due to the vertical erosion being faster than the horizontal erosion. The valley's width is largely occupied by the river, leaving little room for man's activities. Youthful streams are seldom navigable, but the narrow valleys or canyons can be dammed to form ponds, lakes, or used as hydroelectric power plant sites.

2.12 Streams and rivers, or sections thereof, are considered to be mature when they have eroded their channel beds to a fairly smooth even profile. Mature rivers are free of abrupt grade transitions and do not have waterfalls, rapids, or lakes. Mature watercourses have an equilibrium between the rate of erosion and the rate of deposition over a long-term period. Portions of the bed or banks may be eroded by floods, only to be refilled by sediment at other times.

The downcutting of streams in the mature stage will slow or even cease. However, lateral erosion and deposition continues, forming a valley wider than the river. The development of a small floodplain on the valley floor is a sign of the early maturity stage.

Bends in the channel alignment will become smoother and begin to form systematic patterns. By full maturity, the stream is meandering across a flat valley bottom without being limited by the valley sides.

2.13 The width of the meander belt increases as the river enters old age. The old age river has a broad floodplain due to lateral erosion of the valley. The meander belt will be 10 to 20 times wider than the river, with the floodplain several times wider than the meander belt.

The old age floodplain is often intensely used by man for agriculture and cities alike. The deep sedimentary soils are level and easy to farm or construct upon.

The floodplain will be deeply scarred by the river's wandering channel. It may have ox bow lakes, clay plugs, natural levees and extensive backwater swamps.

The old age river and floodplain totally dominate the valley landscape which is formed by the river. It has been estimated that rivers will spend 70 percent of their lives in old age, with 5 percent in youth and 25 percent in maturity.

2.2 ALLUVIAL AND NON ALLUVIAL

The most important river classification system for hydraulic engineers is the distinction between alluvial and non-alluvial rivers, depending on the river's ability to alter its shape and slope.

The alluvial rivers flow over and between beds and banks of unconsolidated sedimentary material subject to being transported and deposited by the river. Because the material was deposited by the river and can be eroded by the river, the channel is able to adjust its dimensions, shape, pattern, and slope in response to the discharge rate and the upstream sediment load. The alluvial river establishes a quasi-equilibrium state where the width, depth, and slope adjust to flow conditions.[12]

Alluvial rivers are generally of the mature or old age groups, and may have a single or multiple channel.[45] An exception are those rivers that may be of a young geologic age and flow through thin deposits of relatively new alluvial deposits. These deposits may be temporary, such as recent flood water deposits that have not yet been eroded by subsequent flows, or they may be at the very beginning of the floodplain formation stage.

Non-alluvial rivers are those that do not flow through modern sedimentary material and must overcome greater resistance before eroding their channels. The material forming the bed and banks resists the action of flowing water and therefore determines the channel morphology. The soils may be from the decomposition of local parent material, dense glacial tills, or old sedimentary soils too coarse or cohesive for the present river to transport.[32]

A special case of the non-alluvial river is the bedrock controlled channel. As the name implies, these rivers are so confined by bedrock that the rock determines the river morphology.[70]

2.3 SEDIMENT TYPE

Alluvial rivers may be further classified according to the type of sediment they carry. Schumm has presented an alluvial classification system based on the percentage of sediment that is in the bed load. The bed load is the material rolled, bounced, or dragged along the bed, and can be represented as being the same size as the bed material. It will generally be a granular sandy material of variable grain sizes.

The "suspended-load channel" is one where only 0 to 3 percent of the total sediment load is bed material. These channels typically have gentle gradients and a meandering pattern with a sinuosity greater than 2.0, and have high suspended sediment loads. If degrading, the bed erodes faster than the banks due to the lack of bed material deposition, resulting in relatively narrow and deep channels. The banks usually have over 20 percent silt and clay in them.

The "mixed-load channels" have a bed load that is between 3 and 11 percent of the total sediment load. This implies a moderately coarse sediment, and hence the channel requires a moderate slope to transport it. The sinuosity is generally between 1.3 and 2.0. The banks will contain from 5 to 20 percent silt and clay.

An "alluvial bed load channel" would have over 11 percent of its sediment load in the form of bed load. This signifies a coarse sediment that corresponds with steep slopes and shallow, wide channels with low sinuosity. The steep channel slope is required in order to have sufficient flow velocity to carry heavy sediments, and the large bed loads tend to fill the bed and force any enlargement of the channel to be in a lateral direction. They usually have less than 5 percent silt and clay in their banks.

2.4 ALIGNMENT

The alignment of the channel when viewed in plan is used as part of the classification system developed by Kellerhals, Church, and Bray, as described below:[32]

a. Straight - With very little curvature, mostly in steep or braided reaches.

b. Sinuous - Slight curvature with a belt width no greater than two channel widths.

c. Irregular - No repeated pattern, sometimes controlled by angular patterns. Includes many braided and non-alluvial channels.

d. Irregular Meanders - Has a vague repeating pattern of various sized meanders.

e. Regular Meanders - Characterized by a clearly repeating pattern in down valley direction. Often are confined meanders or on gravel beds.

f. Tortuous Meanders - A repeated pattern of large meanders that often reverse back upon themselves. Common on underfit streams, where vertical accretion occurs, and on non-alluvial rivers.

An additional method of defining the magnitude of the channel meanders is by a "sinuosity classification". This is based on the ratio between the channel length (following the meanders) and the straight line length between the same reference points.

Sinuosity Classification

Low sinuosity	1.0 - 1.3
Moderate sinuosity	1.3 - 2.0
High sinuosity	Greater than 2.0

3.0 RIVER PATTERNS

The intent of this chapter is to present information on the natural patterns that develop in rivers. Included are patterns of both the horizontal plan and the vertical profile. The information consists of the general relationships that have been found, as well as theoretical evaluations that help explain channel patterns and the interrelationships between the channel plan and profile.

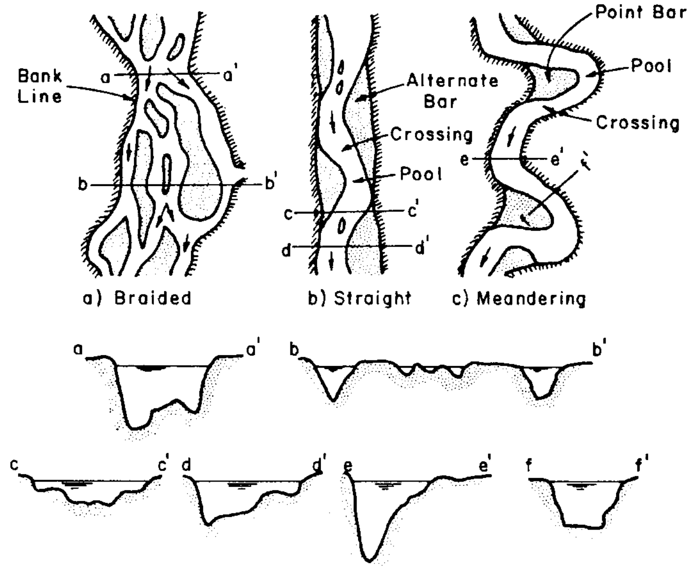
Channel patterns are influenced to a major extent by the past history of the rivers. Because of the long time periods required to carve valleys and create floodplains, the current fluvial geomorphology of the floodplain is often dependent on past flow conditions and sediment loads. As pointed out by Shen, the general characteristics are due to past events, while the smaller in-stream formations and details are a result of recent or present flow and sediment conditions.[72]

The three basic channel patterns are meandering, braided, and straight (see Figure 1). The type of pattern along a river may change from one reach to the next as well as from one year to another, depending on flow rates, sediment loads, sediment size, and upstream and downstream conditions. Occasionally, patterns may overlap, such as when a braided stream has a meandering pattern.

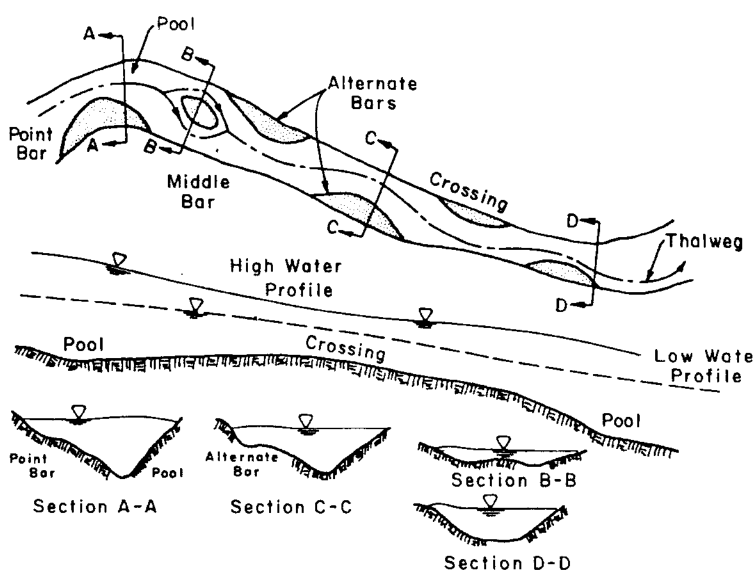
3.1 MEANDERING RIVERS

One of the most common patterns of both alluvial and nonalluvial rivers when viewed from above is that of a meandering alignment (see Figure 2). As the name implies, the river follows a sinuous path with numerous curves. Because most streams do follow a path other than straight, the term "meandering" is reserved for those with a repeating pattern of curves with distinctive geometry.[42] Leopold's definition of the meandering river included having a length of at least 1.5 times the length of its valley, which helps to define the degree of sinuosity for the meandering river.

The classical meandering river has an alignment with evenly spaced symmetrical curves along its length. These rivers are usually



River channel patterns.



Plan view and cross section of a meandering stream.

Figure 1
from Simons
Reference 76

relatively narrow and deep, and have an associated profile pattern of deep pools and shallower riffles. The pools form at the bends due to converging flow, and riffles occur midway between pools.

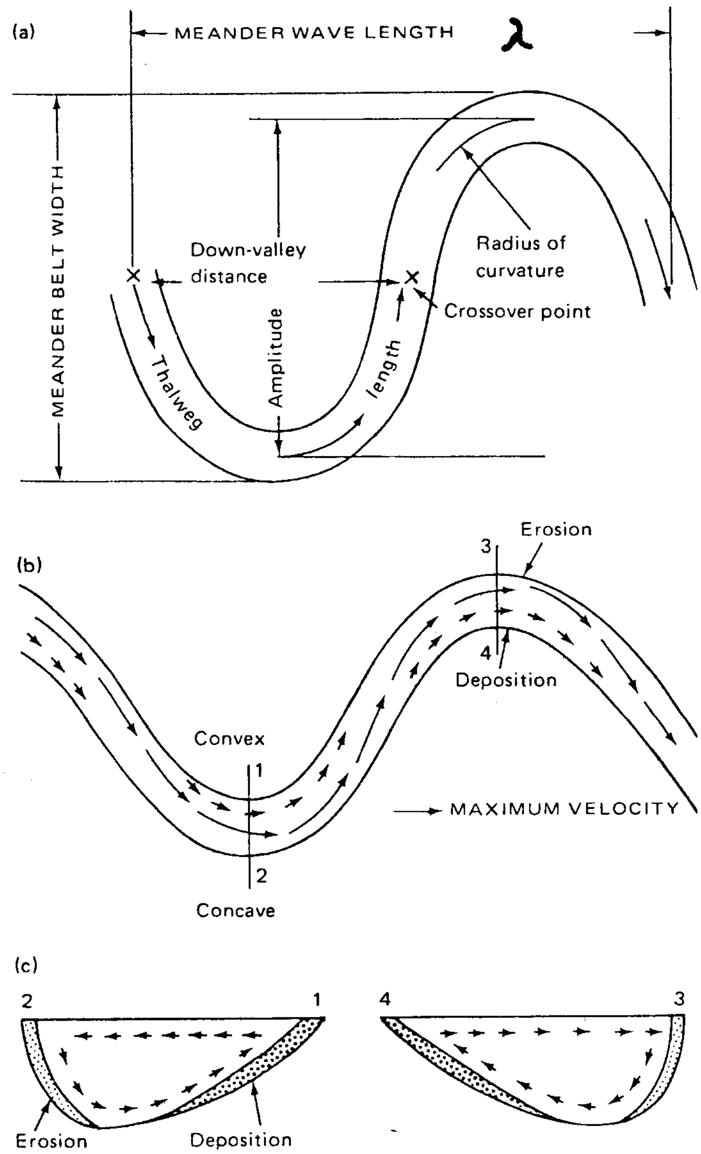
One of the most notable characteristics of many meandering rivers is the channel's ability to shift its position laterally across the floodplain. This is due to the secondary (helicoidal) currents that erode the banks on the outside of the bend quickly enough to have the channel move towards the outside of the bed. The material deposited on the convex (inside) bank at the bend forms a sediment deposit known as a point bar. As the channel shifts position, the point bar fills the old channel with sediment (see Figure 3).

The meanders may also move in a downstream direction. This occurs because the longitudinal flows coming into a bend maintain their linear momentum and strike the concave bank just downstream of the bend. Thus, the bank downstream of the bend erodes, effectively moving the bend downstream. The point bars also tend to form in the downstream direction, as the material eroded at the bend is deposited in the still water on the lee (downstream) side of the existing point bar.[51]

3.11 There are several different theories as to the cause of meanders, with the majority opinion being that it is nature's way of dampening excess energy by increasing the effective river length.[92] In addition, the curvature is such that it minimizes the turbulence of the secondary currents.

Meandering rivers may be an extension of the meandering thalweg concept caused by helicoidal flow. It has been found that the helicoidal flow with cross currents tends to erode the concave side of the thalweg where flow converges. If this lateral erosion were to be initiated in a straight stream at a rate faster than the downstream rate of thalweg movement, it is obvious that the channel as a whole would then also move laterally. Once begun, the meander would force a greater convergence of flow, and hence a larger meander could form.

Morisawa indicates that some geologists believe meandering occurs when rivers can no longer down cut, but says this concept is not valid as even some young streams meander.[51]



The geometry and flow paths of stream meanders and bends.

Figure 2
from Ruhr
Reference 64

Flume studies have found that meandering channels can carry a higher sediment load for a given flow condition than straight channels. Meandering could be response to sediment loads in alluvial rivers.[56]

Schumm also believes that meandering is related to having high suspended sediment loads, based on his field data that show the two conditions occurring together. This may be explained as being due to secondary currents in the meander bends that are able to continually scour and resuspend the sediments and therefore carry sediments more efficiently than straight channels.

3.13 There has been a great deal of research work done in attempts to quantify the properties of meandering channels. The general relations are as follows:[92, 67]

a. Steeper valleys have meanders with longer wave lengths.
b. Flatter valleys have meanders with greater curvature, smaller bend radii.
c. The meander length increases as the discharge rate increases.
d. Large flow rates combined with flat slopes are conducive to forming meanders.
e. Meander width increases as the angle of approach increases.
f. Long flat meanders tend to be unstable, as secondary pools and riffles form at intermediate points and occasionally shift.
g. Deformed meander shapes occur because ideally uniform conditions seldom exist in nature.
h. Streams with little bed load tend to be narrow, deep and sinuous.
i. Meandering streams have a low width to depth ratio.
j. Meandering streams have a high percentage of silt and clay in the surrounding banks and bed.

There have been several empirical formulas developed to describe the length of one meander wave and are summarized here:

Author	Meander Length	Comment
Leopold[42] 1964	7 to 15 B	7 for B $<$ 10' 15 for B $>$ 1,000'
Leopold[42] 1964	11 to 16 B	Measured along channel centerline
Keller 1972[25]	10 to 14 B	
Inglis 1949	$6.6\ B^{0.99}$	
Leopold 1957	$36Q^{0.5}$	Flume discharge
---	$27.4Qmax^{0.5}$	Maximum flow, India
Dury	$30\ Qmaf^{0.5}$	Mean annual flood
Carlson	$106\ Qm^{0.46}$	Annual mean discharge
Schumm[7]	$234(Qmax^{0.48})(M^{-0.74})$	"M" is sediment factor
	$1890\ (Qm^{0.34})(M^{-0.74})$	
Zeller[65]	$10.0\ B^{1.025}$	
Inglis[16]	$36.5Q_D^{0.5}$	Dominant discharge

Note that in all of the above equations, the final meander length can be described as a function of up to three variables, namely width, discharge, and sediment size gradation. The work by Leopold is perhaps the most quoted relationship presented, variously reporting the meander length to be from 11 to 16 times the channel width when measured along the channel. The meander length corresponds with being twice the length of the pool-riffle spacing (5 to 7 times the channel width).

3.14 The amplitude of the meander wavelength is the width of the meander belt as drawn along the outside limits of the river's alignment. Empirical equations for meander amplitudes are:

Author	Meander Amplitude	Comments
Leopold[41]	$A = 2.7\ B^{1.1}$	B = Channel Width
Inglis	$A = 18.6\ B^{0.99}$	
Gregory[16]	A = 14 to 20 B	
Zeller[65]	A = 4.5 B	
Inglis[16]	$A = 16\ Q_D^{0.5}$	Q_D= Dominant discharge
Krombein[24]	A = 15 to 20 B	

There is a lack of consistency in the above data for the meander amplitudes. Leopold explains this by saying that the amplitude is influenced by the erodibility of the stream bank and other local factors.[41]

3.15 Based on data from 50 meandering rivers with various bed material, Leopold concluded that the radius of centerline curvature is usually between two and three times the channel width.[42] This is the same range given by Bagnold.[92] It is interesting to note that the hydraulic engineers have found that the minimum energy loss at bends occurs when the radius of curvature is three times the channel width,[11] and thus the bend radius may be related to its hydraulic efficiency. Additional information on river bends is contained in section 3.5.

3.2 BRAIDED RIVERS

Braided rivers are typically wide and shallow with a series of mid-channel bars and islands. They have multiple flow paths, and are generally unstable with erodible banks. The alignment may be straight or curved.

The dominant feature of braided rivers is the formation and presence of the mid channel sediment bars and islands, composed of the coarser portion of the bed load. Field and laboratory observations by Leopold indicate that braided rivers originate when high flows deposit coarse material in the center of a single stem channel.[41] Once formed, the bars tend to grow in the downstream direction as additional material is deposited in the wake of the initial deposits.[42] Bars that trap sufficient sand and reach the water surface can support vegetation, thus becoming islands.[51]

The bars and islands of the braided channel slowly widen, and deflect the river flow toward the banks where further erosion occurs. The braided channel will thus tend to erode laterally while deposits form on the bed. As a result, they are often wide and shallow.[51]

Braided channels are generally unstable because high flow rates can shift the position and size of the bars. Pools and riffles can form on either a temporary or semi-permanent basis.

3.21 It is commonly believed that aggradation of the river bed is required for braiding to occur. While aggradation and braiding are often associated, the required condition is not aggradation but rather the inability of the river to convey the coarse fraction of the bed load.[51] This situation allows localized sediment deposition to take place in portions of the channel creating multiple flow paths and braiding, without necessarily aggrading the entire river length. Braiding is a process of selectively sorting the bed load, leaving the coarse material in the bed as a lag deposit. It is in response to a sediment load in excess of the transport capacity of a single stream channel.[42]

Among the conditions that often lead to braiding are steep slopes, coarse grained material with low erosion resistance, sediment deposits at junctions, and aggradation that allows flow to overtop the floodplain and cut new flow paths.[16] Fluctuations in flow rates (and sediment transport rates) with high sediment loads during post glacial periods can also contribute to braiding conditions.

3.22 The braided river channels are generally less sinuous than meandering channels and have poorly defined bends at changes in alignment. Braided channels are uncommon when sediment loads are low, or where erosion resistant banks exist that force flows toward the center of a channel.[42] Braided channels are more common in areas with easily eroded banks of sand or gravel.

Leopold has found that braided channels have a slope 1.4 to 2.3 times steeper than undivided channels. The sum of the widths of the several flow paths in a braided channel is 1.6 to 2.0 times the width of equivalent single stem channels. It is noted that if the channel is aggrading, braiding can occur temporarily on either mild or steep slopes.[92]

3.3 STRAIGHT RIVERS

As the name implies, straight rivers have a linear alignment with only minor bends. The bends that are present may or may not have a repeating pattern.

Straight alluvial channels with a single stem seldom exist in nature for long lengths (over ten times the channel width). This is because nonuniform flow, irregular vegetation and bed material will usually induce bends or even meanders. Leopold defines straight channels as having a sinuousity of less than 1.5.[42]

The straight channels found in the field are often a temporary situation, such as the chutes that form between meanders. Other straight channels are the result of man's influence, where he has stabilized banks, dredged, or otherwise altered the river.[76]

3.31 The flow path of water in a straight channel usually does not have a straight alignment. The flow tends to have a sinusoidal path within the channel and is controlled by sediment bars. The thalweg, which is the line of maximum depth, meanders back and forth within the confines of the straight channel banks, except where cascades or flats occur.

3.32 The meandering thalweg in straight channels is accompanied by a series of pools and riffles that are most apparent during low flow periods. The stability of the straight channel is in part dependent upon the resistance of the bank adjacent to the pools on the outside of the thalweg meander bend. If the flow velocity is high enough to erode the bank, then a meandering channel would evolve.

There are two general conditions under which straight channels occur. The first is where very flat slopes prevail, with slow flow velocities that do not erode the bank as mentioned above. The second type of straight channel occurs on very steep slopes, where supercritical flow conditions do not allow helicoidal currents to exist, and flow velociities erode all alluvial material in their path.[65]

3.4 EFFECT OF SLOPE ON RIVER PATTERNS

Several researchers have observed that the slope of a river channel has a strong influence on the pattern that the channel has. They found that for a given discharge, the pattern is related to the slope of the channel bed. On very flat slopes, channels tend to be

narrow and deep with low velocities and straight thalwegs. A slight increase in the slope results in the formation of helicoidal currents that set up a meandering thalweg within the straight channel. There appears to be a threshold slope above which the straight channels with meandering thalwegs form into meandering channels. With the same discharge, and an even greater slope, braided channels will tend to occur if the bank material is erodible, or straight channels, if the material generally resists erosion.[71] Thus the river pattern is influenced not only by flow rates and slopes, but also by the erosion resistance of the bed and bank material.[12]

3.41 Lane was one of the first researchers to establish an empirical formula to define threshold slopes at which the channel pattern changes. He said:

Braided channel slopes $S > 0.010Qm^{-0.25}$

Meander channel slopes $S < 0.0017Qm^{-0.25}$

The discharge used as an index factor was the mean annual discharge in cubic feet per second. Note that an intermediate zone exists between the two distinct threshold slopes, where either pattern may occur. His data was based on rivers with alluvial sand beds.[76, 71]

3.42 A series of flume studies by Ackers and Charlton led to the presentation of a second set of empirical slope-pattern equations. Their data indicated:

Meander channel slopes $S > 0.0021Q^{-0.12}$

Straight channel slopes $S < 0.0015Q^{0.12}$

Between the two channel conditions above, Ackers and Charlton said that a "shoaled" channel pattern would exist. Their formulas are based on constant flume flow rates, which are equivalent to a mean annual discharge.[16]

The meander slope equations by Lane, Ackers and Charlton agree quite well when plotted on log paper.

3.43 Information on the patterns that evolve at bankfull discharges versus slope were developed by Leopold and Wolman as follows, for

rivers with a coarse bed material (median bed size 1/4 inch diameter or larger).

Braided channel slopes $S > 0.06Qmaf^{-0.44}$

Meander channel slopes $S < 0.06Qmaf^{-0.44}$

Straight channels were said to occur on all slopes, and were not found to correlate to the bankfull discharge. The data base used was from 58 natural rivers located in such diverse areas as Alaska, Wyoming, Virginia, and Montana.[41] The flow rate used as an index was the mean annual flood.

Henderson reanalyzed Leopold's and Wolman's data considering the size of the bed material.[20] By plotting the term $(S/0.06Q_{MAF}^{-0.44})$ versus d_{50} grain size, a relation was found where two thirds of all straight and meandering stream slopes fell on the line, and all braiding channel slopes fell above the line marked by:

$$S = 0.64d_{50}^{1.14}Q_{MAF}^{-0.44}$$

This is very similar to his theoretical equation for stable channel slopes as discussed in Section 4.32.

3.44 The above four sets of data and equations that quantify channel patterns lead to several generalizations on pattern, slope, and flow rate, as noted below:

a. The pattern of a natural river at equilibrium is influenced by its discharge rate and bed slope.
b. An increase in slope can change the pattern from straight to meander, or from meander to braided.
c. For a given slope, a braided channel has a higher capacity than a meander channel.

3.5 RIVER BENDS

The changes in the river's alignment may take place at either gradual or abrupt bends. Shen describes three basic types of bends that are found in nature:[72]

(a) Free bends - these occur where the banks are composed of alluvial material that is erodible. This type of bend is typical of meandering channels on floodplains, and encourages symmetrical meanders.

(b) Limited bends - occur where the river banks are of consolidated parent material which limits the lateral erosion at bends. The author expands on Shen's description by adding that the soils may be native soil formed from underlying bedrock, glacial tills of unsorted material, ancient sedimentary soils, or floodplain and river deposits too coarse or cohesive to be easily moved by the present flow condition. Shen states that limited bends often correspond to entrenched rivers which degrade with little lateral movement.

(c) Forced bends - occur where the river strikes a non-erodible material, forcing the river into an angular bend which is only slowly transformed into a rounded curve. The obstacle may be a non-erodible soil, or the side of a valley. A special case of the forced bend is where the river's alignment is controlled by a hard bedrock that erodes at a very slow rate. This special case is called a "fixed bend."

The geometry of the limited and forced bends may be variable and irregular, depending on the degree to which erosion has smoothed the curve.

The rate of lateral bank erosion and the stability of river bends appears to be closely related to the rate at which material is eroded from toe of the bank,[3] and the angle of the bend.[55]

3.51 Bend Radius

The radius of river bends is an important factor in determining the stability and hydraulic energy losses of the bend. The following data is noted:

Source	Radius/Width	Comment
Shen[72]	4.5 - 5.0	Free Bends
	7 - 8	Limited Bends
	2.5 - 3.0	Forced Bends
Leopold[42]	2.0 - 3.0	Range for 50 alluvial rivers
	2.7	Mean for 50 alluvial rivers

Other formulas for estimating the radius of bends are:[72]

Source		
Riply	$R = \sqrt{40\ A}$	A = cross section area, FT
Pazin	$R = \dfrac{181\ \sqrt{(V^2/GD)}\ -15}{Q^2}$	D=depth, Feet S= bed slope
Makaueyer	$R = \dfrac{0.00726\ \sqrt{V^2/GD)}\ \ -15}{S}$	

3.6 BED PROFILE

Many natural river channels have been observed to have a bed profile with a series of shallow and deep sections in an alternating sequence. During periods of low flow, the deeper sections along the profile are pools of still water, while shallow areas appear as riffles of fast moving water. This sequence of pools and riffles occurs on both straight and meandering channels. The sequence is found in conjunction with many bed materials, including bedrock, but is most pronounced where the riverbed consists of coarse sand or gravel.[42]

River pools are defined by the Soil Conservation Service as a deep and wide stream segment, with slower currents than adjacent areas. They can have a bed of mixed grain sizes, including finer sediments than the mean.[90] Keller explains that pools are a topographically low area produced by scour and usually containing fine sediments, and have an average length of "several" times the stream width.[31]

Riffles are said to be areas of the river with a coarse gravel or rubble bed and turbulent flow at low stages. Flow velocities are high enough to remove finer grained sediments.[90] The riffle is thus a topographically high area, and will include accumulated coarse material.[30]

The other two profile patterns that can occur interspaced with pools and riffles are "flats" and "cascades." Flats are a stream segment of uniform grade, where velocities are too slow to be a riffle, and not deep enough to be a pool. Cascades are steep segments with high flow velocities and no pools or flats. The long, steep characteristics differentiate them from riffles. The bed is often bedrock with little loose material.[90]

3.61 The typical length of channel between the low point of a pool and the high point of a riffle has been found to be 5 to 7 times the width of the channel.[30, 42, 27] This repeating interval has been found on many rivers, both large and small, all over the world. The length of the pool is usually longer than that of a riffle.

Being an area of local aggradation, the bed of the riffles tends to have a horizontal cross slope from bank to bank. The pools on the other hand are located near the bends of the river and alternate banks with every other pool on one side. This encourages water to flow across the channel from side to side when flowing from a pool through a riffle to the next pool, which is on the opposite bank. The river's thalweg (deepest line along its length) thus develops a meandering path, even where it is between a pair of straight parallel banks.[30] The length of the thalweg meander is twice the distance between a pool or riffle, or 10 to 14 times the channel width.

The shoals or mid-channel bars on which the riffles form are perpendicular to the thalweg and flow path, and are thus skewed at an angle to the channel centerline and banks.

The cascades occur where the riverbed is controlled by an area of erosion resistant material, and interrupt the natural pool and riffle sequence at points where the normal river slope is variable. Flats occur in uniformly graded sections, and are often a result when major

floods "wash out" the channel bars, or when they are disturbed by man.

3.62 The flow conditions in the pools and riffles vary tremendously depending on the river stage. During low flows, the mid-channel bars at the riffles act as a dam, holding water in the pool. The pools are thus deep and quiet, with a low hydraulic gradient and low velocities that allow fine sediment to settle. The riffles at low stage have faster, erosive flow velocities on the downstream side of the bars. The high velocity selectively removes the lighter particles and carries them to the pools, leaving coarse material behind in a single layer covering or "armoring" the bed.

As discharge rates and flow depths increase, the hydraulic gradient at riffles decreases as the riffle becomes submerged by water in the downstream pool. Conversely, the hydraulic gradient of the pool increases to convey the additional flows. This is then a reversal of the flow conditions at low stage. At the bankfull stage, the riffle is "drowned," and is no longer visible. The water surface profile at this stage is uniform from pool to riffle to pool.[42]

3.63 The pool riffle sequence is formed by the flow characteristics at high stages. The biggest and deepest pools form at channel bends, where flows are directed at the bank and "converge." This convergence scours the channel bottom, creating pools. Convergence of flow, and the resulting scour of pools, also occurs in straight channels due to bank obstruction of asymmetric cross sections. The waters leaving the pool spread (diverge) across the channel, losing velocity and allowing coarse sediments to settle in a bar that becomes a riffle at low flow.[28] The pools that form at channel bends are considered primary pools, while other pools not located at bends are called secondary pools.

Channels that do not show evidence of pools and riffles have either erosion resistant beds or such low velocities that converging flows cannot scour the bed.

In summary then, pools are scoured by converging flow at high stage, while riffles are formed by deposition as the waters diverge.

This concept of convergence and divergence was first proposed by N. de Leliavsky in 1894.[37]

The maximum water depth at pools occurring at bends has been quantified by Fargue.[37] He said that the pool depth increases with the degree of curvature, and provided an empirical formula for it.

$$H = 1.5 \left(1 + \sqrt{C^2 + 1.7C} \right)$$

The depth of water, H, depends on the value of "C" which is a function of the curvature.

Laboratory tests have shown that the maximum depth of pools at bends occurs near the downstream part of the bend, and that the pools' depth increases with the angle of the bend.[72]

3.7 Secondary Currents

The mechanics of the scouring of pools at bends can be described in terms of secondary flow currents. The process is initiated when flow is directed towards a river bank due to curvature, channel obstructions, asymmetric flows, or waves. The converging flow builds up the quantity of water against the bank, making it super-elevated above the water at the channel center. Thus, there is a greater depth of water against the bank, with a higher pressure head than at the center of the bed. Since fluids flow from the points of higher energy to points of lower energy, water will move laterally along the bed from the channel bank towards the center.

When looking downstream, a thalweg that bends to the right will have a counterclockwise spiral, and a bend to the left will have a clockwise spiral.[11]

The secondary flow currents, which are vertical at the bank and horizontal at the bed, combine with the longitudinal flow to form a helicoidal path. The high velocity plucks particles of soil from the bank and adjacent bed, and moves the particles downstream and across the channel. The eroded material is then deposited as the flow diverges away from the bank, creating a point bar or channel side bar opposite or downstream from the pool.[65, 37]

Scheidegger has presented an alternate opinion to explain the formation of helicoidal flow, even in straight uniform laboratory flumes. He feels that the Coriolis force created by the earth's rotation would tend to make northern hemisphere channels erode to the right. Once the right bank was attacked in this manner, flows would rebound and converge on the opposing bank, setting up the sequence of converging and diverging flow paths that scour pools and form riffles.[65]

3.71 The secondary currents decay after leaving the bend, with a reduction in strength in the downstream direction. The length of the decay zone has been described as:[72]

$$L = \frac{1.77\ YC}{\sqrt{g}}$$

, Y = Depth of water
C = Chezy coefficient

Others have described the decay length as being up to 50 times the depth of flow.[3]

The strongest secondary currents often occur during periods of medium flow.[3] This is because the low flows lack sufficient velocity to cause strong second currents, while the peak flows overtop onto the floodplains and bypass the channel curves, often cutting new channels across the floodplain.[55]

3.72 The helicoidal flow has nonlinear velocity components that result in a loss of energy due to friction and turbulence, above and beyond the normal loss of energy due to the surface friction of linear flow. This energy loss can be computed using empirical methods of classical hydraulics.

The energy loss is expressed as a fraction of the change in the total velocity head as shown below.

$$H = K\ (\alpha V^2/2G)$$

H = energy loss
K = coefficient
V = linear velocity
G = gravity acceleration
α = velocity distribution coefficient

The velocity distribution coefficient "alpha" (α) is used to correct the linear velocity head ($V^2/2G$) for the nonlinear and helicoidal flow components. The eddy loss coefficient (K) varies from 0.1 and 0.2 for gradual converging and diverging reaches to 0.5 for sudden transitions in cross sections.[11]

Hydraulic engineers found that the strength of the secondary (spiral flow) current decreases as the ratio of depth to width increases. In addition, it will decrease gradually as ratio of bend radius to width increases, with a minimum strength when the radius of curvature is three times the channel width.[73]

4.0 NATURAL CHANNEL CAPACITY AND GEOMETRY

The purpose of this portion of the report is to present information on the capacity, width, depth, and slopes of natural channels. Sufficient information is available to allow some understanding of why some alluvial channels are larger than others, and why some are deeper, or wider, or on steeper slopes than others. The information is a mixture of field data, laboratory data, empirical equations, and theoretical analysis.

Rivers are constantly changing their properties through the erosion and deposition of sediments, seeking a balance between the rate of sediment supply and transport. In many alluvial rivers, an equilibrium is reached where erosion rates equal deposition rates. When this situation occurs, the river is said to be stable, graded, or in equilibrium. The stable river has a profile where flow rate, channel size, and velocity balance the sediment load.[51] This concept is in common with both the Regime Theory from India and geological methods of analysis.

The quantitative data and empirical formulas on channel flow capacity, width, depth, and slope are all intended for the graded river.

4.1 CHANNEL FLOW CAPACITY

Numerous studies have determined that the size and flow capacity of a natural alluvial channel is related to its flood flow frequency and is fairly constant from one river to another. This is an interesting phenomena, since it means that the channel size is a function of the flow rate, and hence of the watershed size and climate.

The peak flow rates in the river are generally expressed in terms of their recurrence frequency. For instance, a "25-year flood" indicates a peak rate of flow that will statistically occur on an average of once in twenty-five years. The average daily flow is defined as the mean of the flow rates of each day of the year. The mean annual flood (MAF) is the arithmetic mean value of the largest flood in each year of record.

4.11 Leopold and Wolman have found that there is a relationship between the depth of flow, depth of bankfull flow, and the ratio of discharge to the discharge rate at the bankfull stage. The data is based upon rating curves for 13 gaging stations in the eastern half of the United States. The bankfull depth is considered to be at a point that is at the mean height of the stream banks.[42, 95]

NON-DIMENSIONAL RATING CURVE DATA

Ratio Between Depth of Flow and Depth at Bankfull	Ratio Between Discharge and Discharge at Bankfull	Average Return Frequency, Years (Based Upon Annual Flood Peaks)
0.35	0.12	Average Daily Flow
1.0	1.0	1.5
1.25	1.7	5
1.4	2.1	10
1.6	3.4	25
1.8	4.3	50

The above data indicates that for the streams involved,

a. the average daily flow (mean annual discharge) occurs when the flow depth is one third of the bankfull depth;

b. the channel capacity, when flowing bankfull, is equal to a discharge with a return frequency of 1.5 years;

c. the fifty-year storm will inundate the floodplain to a depth equal to 80 percent of the bankfull depth.

Gaging station data indicates that the average daily flow is equalled or exceeded only 25 percent of the time. It is thus a good indicator of the "normal" flow condition.

4.12 A report by the U.S. Geological Survey, written in part by Leopold, presents similar information on the significance of the 1.5-year storm frequency.[38] It states that "Studies of river channels have shown that rivers construct and maintain channels which will carry

without overflow a discharge somewhat smaller than the mean annual flood. In fact, the recurrence interval of the bankfull stage in most rivers is a flow having a recurrence interval of about 1.5 years."

4.13 Leopold's data from the eastern United States has been confirmed by later research by others at 34 sites in the southeastern United States. The channels were found to contain the mean annual flood (2.3 year frequency) within their banks on steeper reaches, with some overflow of the mean annual flood in flat reaches of the river.[16]

4.14 A researcher in England named M. Nixon found the bankfull discharge to occur on 0.6 percent of the days, or 1 day every 6 months. This is based on a partial duration series of all flood events, rather than the series of peak floods. Converting the U.S.G.S. data to a record of all floods, the bankfull discharge would have a statistical average return period of 9 months.[20]

This is also mentioned in a more recent publication by Leopold, which states that bankfull stages "occur approximately twice each year."[39]

It is thus apparent that the designation or identification of the frequency of the bankfull flow condition is sensitive to the type of data available. The frequency analysis based upon annual maximum floods indicates that the bankfull stage occurs once every 1.5 years, while an analysis based upon daily flow rates indicates that the bankfull stage occurs twice a year.

4.15 Researchers in Australia have identified up to three benches within channels in that country and related them to flow frequencies. Each bench is a physical level in the valley cross section. The high bench is equivalent to the flood plain level, and has been found to have an elevation corresponding to a return flow frequency of 1.3 to 2.7 years.[96]

The middle bench is generally identified as being at the limit of terrestrial vegetation on the channel bank, being above the normal water level but below the floodplain. The elevation of this bench was found to be equivalent to a flow frequency of 1.0 to 1.2 years.

The lowest bench, which is usually under water, is at the toe of slope on either side of the stream bed.

This approach, used in Australia, differs from the American view of stream form and capacity, but results in a bankfull discharge frequency (1.3 to 2.7 years) not too different from Leopold's data based on the mean annual flood series.

4.16 In a similar case, the U.S. Army Corps of Engineers identifies three flow conditions which tend to influence the river's geometry and size, as noted below:[24]

a. Low flows are responsible for forming the low water channel and thalweg that typically meander across the wider main channel bottom.

b. Minor flood events (mean annual flood±) create and maintain the overall size of the main channel, and generally stay within the top of banks.

c. Major floods that occur on an infrequent basis overtop the river banks and flow on the floodplain, shaping the floodplain.

The size and shape of an alluvial river and valley at any one time will depend in part on the recent flow conditions. For example, a major flood observed by the author on the Farm River in East Haven, Connecticut removed significant quantities of material from both the bed and banks and completely reshaped protions of the channel. In other areas, large amounts of sediment and debris were left in the channel where overbank flows on the floodplain by-passed reaches of the channel.

When observed one year later, the scoured areas showed signs of aggradations with fresh sediment and river banks that had been scoured to a near vertical slope that had already sloughed and was being reduced to a lesser slope.

4.17 Although the average natural channel appears to have a capacity equal to or less than the mean annual flood, some alluvial rivers have channels with much higher capacity. Some researchers feel that the river form is a function of the long-term sequence of events. The feeling is that if the ratio of the individual flood discharges to the mean annual flood is small, then the channel can be in

equilibrium with a capacity of the mean annual flood. When the largest floods are much greater than the mean annual flood, the channel will enlarge to abnormal size due to erosion.

The conditions that would lead to an abnormal channel size are steep, compact watersheds, high percentages of impervious surfaces, and droughts followed by floods. Numerous cases of the latter have been reported along the Gila, Republican, Cimarron, Little Missouri, and Farmington (Connecticut, 1955) Rivers.[79]

4.2 CROSS SECTION GEOMETRY

It has already been shown that the flow capacity of natural river channels is fairly well-defined, and the data hereinafter will attempt to quantify the cross sectional properties found in natural channels that are practically stable.

The primary elements of the channel cross-section are its width, depth, perimeter length, and area. Together, they fully describe the size and shape of the channel cross section.

The shape of the river's cross-section is strongly influenced by the type of soil forming the bed and banks. Channels in cohesive soils tend to be relatively deep and narrow due to the erosion resistant soils that are stable on the banks, and only erode along the bed where the velocity and tractive force is greatest.

Rivers flowing through areas with fairly uniform sandy soils that are easily eroded tend to be wide and shallow. Under these conditions, the banks are of non cohesive material and rapidly collapse whenever the stream bed erodes at the toe of the slope. The bank material then slides into the channel, filling the bed with loose material that prevents the river from having great depth or steep bed cross slopes.

Soils with a wide grain size gradation, such as glacial tills and old, unsorted floodplain deposits, have moderate erosion resistance in both the beds and banks. These channels initially erode the bed due to the higher velocity and shear stresses there. However, as they erode their beds, only the fine grained materials are removed, leaving the coarser material behind to form an armored layer on the bed. This armored layer essentially stops bed erosion under normal flows

but allows slow erosion to continue on the banks where the armoring process cannot occur (the coarse material tends to slide down the bank to the bed after the fine material is removed, leaving the bank unprotected). In time, the river width either reaches an equilibrium, or the accumulated coarse material at the toe of the bank eventually stops bank erosion.

The width to depth ratio of rivers in areas with unsorted granular soils varies with the soil gradation, but tends to exhibit shallow to moderate depths with moderate to high widths.

The width, and hence the depth, of river channels may also vary with the sediment load and vegetation. Vegetation on the banks tends to reduce bank erosion where dense roots exist. Where high bed loads are carried, the river tends to be shallow, and thence wider. If high wash loads exist, the fine materials are often deposited along the banks during low flow periods, encouraging vegetation on the lower banks. The combination of cohesive sediment and vegetation near the water line helps to stabilize the surface and reduce bank erosion.

4.21 A number of generalizations on cross-section geometry are presented below, prior to the various empirical methods for estimating cross-section properties.

a. Channel depth and width increase in size with increasing flow rates.[40]
b. The final channel form results from the relative resistance of the channel bed and banks to the forces of erosion.[50]
c. Channels on erosion resistant material are narrow and deep, while erodible soils correspond to channels that are wide and shallow.[92]
d. Vegetation tends to encroach into the edges of channels. This reduces bank erosion, but encourages bed erosion. Vegetated channel banks thus promote channels that are deeper and narrower than in unvegetated areas.[16]
e. Wide flow fluctuations encourage wider and incised channels with internal braids at low flow stages. The braids change form, shape, and size after each flood.[92]

f. In alluvial channels, changes that lead to equilibrium persist, while those changes that do not lead to equilibrium are destroyed.[46]

g. The channel depth and width vary when the sediment transport rate or load changes.[46]

4.22 The empirical equations defining the channel cross section properties are for channels that are stable over a long period of time. Alluvial channels are constantly changing in shape and size in response to short term changes in water and sediment flow rates. The term "regime" is applied to those alluvial channels where the net effect of short term channel changes was a long term balance of equilibrium. The regime channels are still subject to the short term scour, deposition, depth and width changes and slope changes, but the net effect does not result in a long term change.

The typical channel widths, depths, and average velocities described hereinafter are for regime channels. Equations for natural and manmade regime rivers and canals were first developed in India during the nineteenth century by British engineers. They noted which rivers and canals had long term equilibrium, and obtained field data that led to their original regime cross section equations. The regime concept has been further refined with use of additional data from other countries in this century.

The width of a channel can be measured in several different ways, depending on the definition of width. In perennial channels in vegetated areas, the most common width definition is the distance between the limits of permanent vegetation on the two opposing river banks.

Several channel geometry equations are presented below. In all cases, the equations are valid only in channels or canals that have long term stability and are selfcut in alluvium similar to that of the present sediment load. This group of equations is presented in the format that is favored by geologists and geographers.

CHANNEL GEOMETRY

Author	Surface Width	Depth	Velocity
Leopold & Maddock	$aQ^{0.5}$	$cQ^{0.4}$	$KQ^{0.1}$
Lacey	$2.67Q^{0.5}$	$0.64Q^{0.33}$	$0.8Q^{0.167}$
Pettis	$2.45Q^{0.5}$	$0.715Q^{0.3}$	$0.8Q^{0.02}$
Bray	$2.38Q_2^{0.527}$	$0.266Q_2^{0.33}$	$8.0(D)^{0.6}(S)^{0.29}$
Ackers & Charlton	$3.6Q^{0.42}$		
Smith	$KQ^{0.5*}$		

*K = 1.7 for clay sediment loads, 2.2 for sand sediment loads

Leopold and Maddock have found that width, depth, and velocity of flow in natural channels at stable cross sections are related to the discharge rate by simple power function equations.[5]

Their expressions are:

$$B = aQ^b$$

$$D = cQ^f$$

$$V = kQ^m$$

The width "B" is the surface width at the specified flow condition, and the depth "D" is the mean depth. The equations are considered valid for discharge rates "Q" up to the bankfull discharge rate. The data base was obtained from 20 gaging stations in Wyoming and Montana, under generally semi-arid conditions. Note that the discharge rate is equal to the width times depth times velocity, and thus the sum of the coefficients b, f, and m must equal one, and the sum of a, c, and k equal one.

The equations by Lacey were developed in India based on data from silt laden canals. His information has been published in several different forms of varying complexity. They have been widely used in the British Commonwealth nations for the design of stable canals with near constant flow rates. Henderson has confirmed their format by using theoretical approaches and basic assumptions.[19] Leliavsky, on the other

hand, questions their accuracy for use in other geographical areas.[37] The Lacey surface width equation coefficients ranged from 2.5 to 2.8, and Leopold felt that this range was somewhat narrower than on American rivers.[40]

Measurements along the Miami River served as the data base for the Pettis' equations, which were set up in the style of Lacey for ease of comparison and found to be similar.

Bray developed his equations based upon the data from 70 rivers in Alberta, Canada at a discharge rate of the two-year flood. They were generally gravel bed rivers, as opposed to the rivers of Lacey which were in silt and sand sediments.[71] In recent work, Bray found that the Lacey equation for estimating mean flow velocity was as reliable as the Mannings equation for alluvial gravel bed rivers in Canada.[9]

The equation presented by Ackers and Charlton is based on flume experiments in a laboratory using erodible sand as a base material. The coefficient 3.6 was found for straight channels in the flume, and it increased up to 7.2 for meandering channels in the flume.[16]

It is reemphasized that the rather simple equations above are for stable channels, neither degrading or aggrading. Leopold does point out that for channels of constant width and discharge, an increase in bed load will lead to higher velocities and shallower depths.[40] This relates to Schumm's concept that channel size and shape are related to sediment in some manner. Schumm, in fact, does hint at this, stating in one publication that the reason for variations in the coefficient of the Lacey equation is the effect of sediment size.

4.23 The "Regime Theory" formulas were originally developed in India as an empirical approach to define the properties of movable sandbed canals that are in equilibrium. They are based upon data from irrigation canals with fairly steady flow rates and significant sediment loads. The basic equations developed by Gerald Lacey for the geometry of alluvial channels are:

$$V = 60\ (f)^{-0.25}(R)^{0.75}\ (S)^{\frac{1}{2}}$$

$$\frac{P}{R} = 7.1\ V$$

These have been modified over the years by numerous researchers. The channel design code adopted by the Indian Roads Congress is based upon the modified Lacey Regime equations, as noted below:

$$Y = 0.47 \quad \frac{Q}{f}^{1/3}$$

d = mean depth of flow, feet

Q = design discharge, CFS

f = silt factor

q = average flow per foot of width, CFS/FT

The mean depth of flow in a constriction is:[52]

$$Y = \frac{0.9\ (q)^{0.67}}{f^{1/3}}$$

The value of "f", which is known as the Lacey silt factor, is dependent on the bed material size. It serves as an indicator of the bed and bank erodibility. The values given below are as used by the Indian Road Congress Bridge Code:

Mean Grain Size, MM	Bed Material Description	Value of "f"
0.08	very fine sand	0.5
0.16	fine sand	0.7
0.23	fine-medium sand	0.85
0.32		1.0
0.50	medium sand	1.25
0.72		1.5
1.00	medium-coarse sand	1.75
1.30	coarse sand	2.0

The value of "f" has also been presented in the form of an equation by Lacey (1930).

$$f = 64\ (D_{50})^{0.5} \qquad D_{50} \text{ is in inch units}$$

Blench has modified the basic Regime Theory to account for the erosion resistance of the bank and bed material individually.[6]

$$d = \frac{V^2}{F_b} = \left(\frac{FsQ}{F_b^{\ 2}}\right)^{0.33}$$

$$V = \left(F_b F_s Q\right)^{0.166}$$

$$b = \frac{V^3}{F_s} = \left(\frac{F_b Q}{F_s}\right)^{0.5}$$

where d = channel depth, FT

b = channel width, FT

v = flow velocity, FPS

Q = flow rate, CFS

F_b is the Blench bed factor which is a function of the flow stage, bed load sediment load concentration and the bed material size. Under normal conditions,[6]

$$F_b = 1.9 \sqrt{D_{50}}$$

where D_{50} = mean bed material particle size, (MM)

F_s is a factor representing the erodibility of the channel sides. The values are not available in a formula format because of their empirical nature, but erosion of the banks is said to begin when:

friable sandy loam	$F_s = 0.1$
silty, clay loam	$F_s = 0.2$
very cohesive, clay	$F_s = 0.3$

ASCE cautions that it takes considerable judgement and experience to properly select the coefficients F, F_b, and F_s, and that the Regime Theory equations should not be used where the channel, flow rates, or sediment loads differ from those used to develop the theory.

Note that the values of channel width and depth can be expressed as a constant times the square root of the flow rate, as with the geologic relations, and this allows the Regime Theory equations to be compared with the geologic data as in section 4.22.

As already noted, the "Regime Theory" equations have been modified

several times and appear in many different forms. Many (but not all) of the "Regime Theory" equations are summarized on the following Figure, prepared by Simons and Richardson for ASCE and reproduced from Chow.[11]

4.24 Methods and quantitative equations intended for the design of stable alluvial rivers have been proposed in recent years by Blench,[6] Peterson,[60] and by Simons and Albertson.[74] The Simons and Albertson method has been accepted as a viable approach to designing stable alluvial channels by the U.S. Soil Conservaton Service as presented below.[91]

Equations for the perimeter, hydraulic radius, cross section area, velocity and the width to depth ratio are available for three types of channels. The froude number must be lower than 0.3 for the channel to be stable.

Channel Type	Description
A	Sand bed and sand banks
B	Sand bed and cohesive banks
C	Cohesive bed and cohesive banks

Cohesive conditions are when the Plasticity Index is greater than 7. The basic equations and the coefficients for each channel type are:

	Coefficients by Channel Type		
	A	B	C
$P = C_1 Q^{0.512}$	3.30	2.51	2.12
$R = C_2 Q^{0.361}$	0.37	0.43	0.51
$A = C_3 Q^{0.873}$	1.22	1.08	1.08
$V = C_4 (R^2 S)^{0.33}$	13.9	16.1	16.0
$W = C_5 d Q^{0.151}$	6.5	4.3	3.0

The depth and width are defined for two conditions:

$d = 1.23\ r$, for $1 < R < 7$

$d = 2.11 + 0.934\ R$, for $7 < R < 12$

$W = 0.9\ P$

$W = 0.92\ W_T - 2.0$

Summary of Regime Equations*

Engineer	Date	Velocity V, ft	Slope S	Silt factor	Area A, ft²	Wetted perimeter P, ft	Hydraulic radius R, ft	Width B, ft	Depth D, ft
Kennedy	1895	CD^m or $0.84D^{0.64}$		$C = \phi(d)$					
Lindley	1919	$0.95D^{0.57}$ or $0.57B^{0.36}$						$3.80D^{1.61}$	$0.44B^{0.62}$
Lacey	1929	$1.17f^{1/2}R^{1/2}$	$\frac{f^{5/3}}{2{,}587Q^{1/6}}$	$\phi(d)$ and $\frac{0.75V^2}{R}$	$\frac{3.8V^5}{f^2}$	$2.67Q^{1/2}$			
	1929†	$\frac{1.346R^{3/4}S^{1/2}}{N_a}$ $16.12R^{2/3}S^{1/3}$	$\frac{f^{3/2}}{2{,}614R^{1/2}}$	$\left(\frac{N_a}{0.0225}\right)^4$	$\frac{4.0V^5}{f^2}$				
	1934	$1.155f^{1/2}R^{1/2}$ $16.0R^{2/3}S^{1/2}$ $0.79Q^{1/6}f^{1/3}$	$\frac{5.47 \times 10^{-4}f^{5/3}}{Q^{1/6}}$	$f_{VR} = \frac{0.75V^2}{R}$	$\frac{1.26Q^{5/6}}{f^{1/3}}$	$\frac{8}{3}Q^{1/2}$	$\frac{0.47Q^{1/3}}{f^{1/3}}$		
	1939		$\frac{3.5 \times 10^{-4}}{R^{1/2}}$	$f_r = Kd^{1/2} = 1.76d^{1/2}$ $f \propto \frac{d^{1/2}g^{3/2}}{(\nu g)^{1/2}}$					
	1946	$\frac{Kw}{S}$ $\frac{K(R^{1/2}S)^n}{S}$ $\frac{1.60(R^{1/2}S)^{3/4}}{S}$		$f_{SV} = 48S^{1/2}V^{1/2}$					
Bose	1936	$1.12R^{1/2}$	$\frac{2.09 \times 10^{-3}d^{0.86}}{Q^{0.21}}$		PR	$2.8Q^{1/2}$	$0.47Q^{1/3}$		
Malhotra	1939	$18.18R^{0.63}S^{0.34}$							
White	1939	$\frac{0.7\omega^{1/4}g^{3/5}R^{1/2}}{Q^{1/20}}$		$f_{VR} = \frac{0.37g^{4/5}\omega^{1/2}}{Q^{1/10}}$					
Inglis	1941	$12.0(R^2S)^{1/3}$	$\frac{f_m^{1.63}}{2{,}110Q^{0.145}}$						
	1947	$\frac{\alpha_3 g^{7/18}Q^{1/6}(C\omega d)^{1/12}}{\nu^{1/36}}$	$\frac{\alpha_5(C\omega d)^{5/12}}{\nu^{5/36}g^{1/18}Q^{1/6}}$	$\frac{\alpha(C\omega)^{1/2}}{g\nu^{1/6}}$	$\frac{\alpha_2\nu^{1/36}Q^{5/6}}{g^{7/18}(C\omega d)^{1/12}}$			$\frac{\alpha_1 Q^{1/2}(C\omega)^{1/4}}{g^{1/3}\nu^{1/12}d^{1/4}}$	$\frac{\alpha_4\nu^{1/6}Q^{1/3}d^{1/6}}{g^{1/18}(C\omega)^{1/3}}$
Blench	1939 1941 1946 1952 1957	$(F_bF_sQ)^{1/6}$	$\frac{F_b^{5/6}F_s^{1/12}\nu^{1/4}}{3.63(1 + aC)gQ^{1/6}}$	$F_b = \frac{V^2}{D} \propto d$ $F_s = \frac{V^2}{B} \propto d$	BD			$\left(\frac{F_bQ}{F_s}\right)^{1/2}$	$\left(\frac{F_sQ}{F_b^2}\right)^{1/3}$

* Prepared by D. B. Simons and E. V. Richardson for the American Society of Civil Engineers Task Force on Resistance to Flow in Alluvial Channels.

† Equations developed in answer to original discussions.

NOTATION: a = about $\frac{1}{400}$ for uniform sands originally used in the experiment and probably about $\frac{1}{233}$ for natural river-bed sands; C = silt factor in the Kennedy equation, sediment charge or sediment transport divided by water discharge in the Inglis equations, or bed-load charge in hundred-thousandths by weight; d = representative diameter of sediment particles, usually d_{50} or the size smaller than 50 per cent of the weight; f, f_{VR}, and f_{SV} = silt factor relative to the sediment; f_m = a constant; F_b = bed factor; F_s = side factor; g = gravitational constant, or 32.2 ft/sec²; K = a constant = $D^{1/2}S$; m = an exponent; N_a = rugosity factor = $0.0225f^{1/4}$; Q = discharge of water; w = width of channel; ω = fall velocity of the particle of size d; α, α_1, α_2, α_3, α_4, and α_5 = coefficients; and ν = kinematic viscosity.

All dimensions are in English units except d, which is in millimeters.

Figure 3

from Chow

Mean Annual Discharge Series	Mean Annual Flood Series
$B = \frac{37\ Qm^{0.38}}{M^{0.39}}$	$B = \frac{2.3\ Qmaf^{0.58}}{M^{0.37}}$
$D = 0.6(M)^{0.342}(Qm)^{0.29}$	$D = 0.09(M)^{0.35}(Qmaf)^{0.42}$
$F = 255\ M^{-1.08}$	$F = 21.4\ \frac{Qmaf^{0.18}}{M^{0.74}}$

Where B = channel width
D = channel depth
F = ratio (width/depth)
Qm = mean annual discharge
Qmaf = mean annual flood

The value of "M" is the percentage of bed and bank material that is silt or clay (finer than 0.074 mm). Schumm uses this term as an indicator of channel properties and considers it as being important in identifying channel characteristics.

Schumm's equations clearly indicate that the channels with fine grained beds (high values of "M") will be narrow and deep, and channels with coarse material (low values of "M") will be wide and shallow. The relations developed by Schumm are supported by the research in England by Gregory.[16]

4.25 GRAVEL BED ALLUVIAL RIVERS

Alluvial Rivers with gravel beds have special characteristics that are not in common with the typical sand bed alluvial rivers. Gravel bed alluvial rivers have banks and beds of coarse non-cohesive material including sand, gravel, and cobbles. They frequently have river beds

"armored" by a layer of the coarsest gravel due to selective erosion of the finer material. They are common in both mountain and glaciated areas, with much of the research and available data being from Canada. The bed of these alluvial rivers consists of gravel and cobbles. This bed material is often only a thin layer of coarse material overlying material of mixed grain sizes. The "armor" layer of the soil matrix is eroded, and the coarser material accumulates on the bed. An effective "armor" layer can occur from only a single layer of coarse particles of the $d_{90}\pm$ size.

Gravel bed rivers can qualify as being alluvial when the material is subject to general movement by the river. Unlike sand bed rivers, the gravel (and cobbles) material is moved only at high flow rates and is often stable at normal flows. The channel size is altered primarily by the dominant discharge, and hence is in equilibrium for that flow. Tests have found that even the armor layer material of the bed is subject to movement by bankfull discharges.[58] The angle of repose of the gravel and cobbles determines the slope of the channel banks. It also affects the overall width to depth ratio because flat channel bank slopes increase the width of the channel. Parker has found that the gravel bed alluvial rivers tend to have stable banks with moveable beds. He indicates that this is the result of marginal shear stresses that are able to sustain sediment movement on the bed where the greatest flow depths and velocities occur, but not on the banks. The relatively stable banks discourage the formation of meanders, and thus the gravel bed streams tend to be relatively straight.

The gravel streams with coarse bed material have high grain roughness, and most energy losses are due to this grain roughness. There are few, if any, bedforms because the bed load transport rate is low.

Parker developed a mathematical model for the geometry of gravel bed alluvial rivers using both regime considerations and dimensional analysis. He found:

$$B = 4.4\ \tilde{Q}^{0.5}\ D_{50}$$ B = Top of bank width

$$H = 0.253\ \tilde{Q}^{0.415}\ D_{50}$$ H = Average cross section depth

$$S = 0.223\ \tilde{Q}^{-0.410}$$ S = Energy grade line slope

V = Velocity

Where $$\tilde{Q} = \frac{Q}{D_{50}^{2}\sqrt{\left(\frac{Ps - 1}{P}\right) g\ D_{50}}}$$

In comparing the above equation results versus a large amount of field data, the predicted channel depths were found to average 20 percent to low, with some scatter to the slope data.[58]

Howard Chang has recently developed a computer model for analysis of the geometry of gravel bed alluvial rivers at San Diego State University. It has been used to evaluate gravel bed rivers by assuming that the channel banks are stable and at predetermined slopes.

The model solves for the channel geometry by use of three independent hydraulic equations. The three conditions that must be satisfied are for flow resistance, sediment load, and minimum stream power. The three geometric terms for width, depth, and slope are solved for by successive trials until the three independent equations have acceptable results. The computer was used to solve for the three geometric terms as a function of flow rate and median grain size.

The resulting threshold channel bed slope for bankfull discharges is:

$$S = \frac{0.000442\ d_{50}^{\ 1.15}}{Q^{0.42}}$$

S = threshold

d_{50} = median particle size, mm

Q = bankfull discharge, CFS

Rivers with bed slopes above the threshold value would have an active mobile bed at the bankfull discharge.

The channel width was determined to be:

$$B = \left[1.905 + 0.249\left(\frac{\ln\ 0.001065(d)^{1.15}}{SQ^{0.42}}\right)^{2}\right] Q^{0.47}$$

The channel depth was determined to be:

$$D = \left[0.2077 + 0.0418\left(\frac{\ln 0.000442(d)}{SQ^{0.42}}\right)^{1.15}\right] Q^{0.42}$$

The analytical results reportedly agree fairly well with field data, with the widths being underestimated on large rivers and overestimated on small rivers. The analytical depths are overestimated on small streams, and overestimated on large streams.

In separate research, Kellerhals developed empirical equations for the geometry of alluvial gravel streams as noted below:

$$B = 1.8\ Q^{0.5}$$

$$D = 0.166\ Q^{0.4}\ K_s^{-0.12}$$

B = width, feet
Q = dominate discharge, CFS
D = flow depth, feet
K_s = grain roughness, feet

4.26 A topic related to the total depth of the channel is the average depth fo flow at normal discharge rates. Wolman and Leopold tabulated this data for localities in North and South Carolina, and found that the mean depth of flow for the mean annual discharge (exceeded 25 percent of the time) is related to the distance from the headwater as follows:[95]

MEAN DEPTH OF FLOW $\propto$ LENGTH$^{0.64}$

Mean Depth, Feet	Distance from Headwater, Miles
0.5	2.0
1.0	6.5
2.0	22
3.0	42
4.7	100

This type of data is of course dependent on local climate and flow conditions and may not represent other areas, but shows how slowly the mean depth increases downstream.

4.27 The approximate depth of flow for the 100 year flood frequency has been used in Connecticut for the approximate mapping of flood prone areas. The data, compiled from USGS gaging stations, relates the flow depth to the drainage area, as noted below.[82]

Drainage Area Square Miles	Depth Feet	Range
2	4	3-8
10	6	4-10
30	8	6-12
70	10	7-14
150	12	11-16
300	15	15-20
1000	20	

4.3 CHANNEL GRADIENTS

The slope of river beds is initially controlled by the topography and geology of its watershed area, and thus is predetermined.[12] The river slowly alters the initial slope by erosion and deposition along its path, and will eventually achieve a bed slope determined by both the flow conditions and watershed topography and geology. The river may eventually reduce the effect of initial conditions and solely reflect a slope based upon fluvial conditions. This slope should then be fairly stable until fluvial conditions or geologic conditions change. A river whose bed slope is determined by fluvial conditions and is relatively stable is said to be "graded", or in a "regime" state.

The profile of natural river beds have been found to ultimately concave upwards. The profile is nearly horzontal at the river's downstream end, with the slope becoming steeper near the headwaters.

The sediment grains of upland areas tend to be larger in diameter than in the lowlands due to shorter transport distances and lower flow rates that are less able to transport coarse sediments on a mild slope. The headwater areas require a greater channel slope for an

equilibrium transport rate to occur.[51] The finer sediments found in the downstream river valley are due to selected transport of fine grains from upstream decomposition, and greater abrasion of the particles. The finer grain sediments are easily moved and require less river slope for transport.

4.31 Several empirical relations have been found between the channel slope and basin characteristics. Schumm developed data and an equation that defines stable river bed slopes. The data is based upon rivers in the semi-arid portions of the United States.

Stable "graded" channels were found to occur along a line defined by the equations:

$$F = 255(M)^{-1.08}$$

F = Width-depth ratio
M = Percentage of silt and clay in the bed

Channels that fall above the line described by the equation are said to be aggrading, while those below the line are degrading. Schumm summarized other river slope data as noted below:[71]

Empirical Slope - Equilibrium Equations

Researcher	Location	Equation	Comment
Lacey	India	$S = (8\ d)^{3/2}/2580\ Q^{0.11}$	d= sed. dia., in.
Hack	Virginia	$S = 18\ \frac{d50}{A}^{0.6}$	FT per mile A= area, miles
Brush	Appalachia	$S = 0.34\ L^{-0.81}$	Shale
		$S = 0.019\ L^{-0.71}$	Limestone
		$S = 0.046\ L^{-0.67}$	Sandstone L is river length, miles
Bray	Alberta	$S = 0.965\ (Q_2)^{-334} d_{50}^{0.58}$	
Schumm	Great Plains Australia	$S = 60(M)^{-0.38}(Qm)^{-0.32}$	Qm = mean discharge M = % silt-clay

The slope equations by Lacey, Hack, Bray and Schumm all include a term for sediment size or concentration. The equations by Brush include a term for length, which in turn can be said to relate to sediment size. This leads to the conclusion made by Hack that at any section of a river, the slope is dependent on the resistance offered by the rock being eroded and the sediment transport capacity required for equilibrium.

The regime slope equations by Lacey is of interest because it has been widely used in British Commonwealth countries for the design of stable channels. Henderson reports that his recent work, based on pure theory, supports the Lacey equations as confirmed when the threshold point of sediment movement begins.[19]

4.32 The threshold theory of shear stress, as developed by Lane and used by the U.S. Bureau of Reclamations, was modified by Henderson to an equation of slope for stable channels:[19]

$$S = 0.44 \ d_{90}^{\ 1.15} \ Q^{-0.46}$$

Note that this equation, although developed from pure theory, contains an expression for the sediment size in the same manner as the empirical equations that are based on field measurements.

There is an additional equation for channel bed slopes at equilibrium based upon theoretical considerations of sediment transport.

$$S = \left(\frac{0.00021 \ DB}{Q}\right)^{0.75}$$

Where: D = mean bed particle size

B = channel width

Q = discharge

Slopes greater than this allow transport of the mean bed particle (hence erosion), while slopes less than this are stable (in theory).

4.33 The regime slope can also be obtained from the regime velocity equation by first solving for the cross section area and finding the average velocity and corresponding slope. The regime slope can be checked by using the regime geometry data and the flow rate with Manning's equation.

4.4 PREDICTING DISCHARGE RATES FROM RIVER MORPHOLOGY

4.41 The U.S. Geological Survey is adopting standardized techniques for measuring channels, computing bankfull stages, and relating them to flood flow rates and frequencies.[63] They are using fluvial morphology observations of channel widths to predict flow rates in perennial channels in arid and semi-arid portions of 8 western states.

They use two channel widths as indicators of the 10-year and 50-year flood flows. The "whole channel section" width used is the width from top of bank to top of bank at the edge of the floodplain. The "active channel section" is within the whole channel section, and is the width of the low water channel between vegetation lines or at an elevation equal to that of the highest channel bar.

The whole channel width (WC) is related to the active channel width (AC) as follows:

$$WC = 1.75\ AC^{0.96}$$

The published curves relating the whole or active channel widths to the 10 and 50 year storms are being expanded into Kentucky and Virginia. The curves are not valid for vegetated, braided or ephemeral channels.

4.42 Relationships between channel capacity and discharge frequency have been investigated by Brown who produced regression equations for actual flows.[10] He evaluated channel capacity data and watershed area versus discharge data and found that channel capacity was superior in terms of predicting discharge frequencies. The watershed area was found to be of value as an indicator of low flow rates during periods without rain.

The following equations were developed:

$$Q_{min} = 0.0023\ Q_b^{1.14}$$

$$Q_1 = 1.49\ Q_b^{0.96}$$

$$Q_5 = 8.62\ Q_b^{0.84}$$

$$Q_{10} = 18.01\ Q_b^{0.78}$$

$$Q_{max} = 77.93\ Q_b^{0.64}$$

Where:

Q_{min} = minimum monthly discharge

Q_b = channel capacity, bankfull

5.0 FLOODPLAINS

Floodplains are one of the most important landscape features formed by fluvial processes due to their widespread use by man. A thorough understanding of floodplain deposits, characteristics, and formation is an essential part of understanding man's relation to the fluvial environment.

Maddock suggests that the definition of floodplain differs depending on the viewpoint of the scientist involved.[46] Geologists will define the floodplain as that area of the river valley covered with sediment material deposited by floods, while the hydrologist defines it as that area of the river valley periodically overflowed by water that is in excess of the channel capacity. Although the definitions vary, they do convey the concept that a floodplain involves both the flow and sediment load of a river. The purpose of this section then is to explore the concept of the floodplain, its formation, and connection with river discharges and sediments.

5.1 FLOODPLAIN FORMATION

The formation of river valleys and their sedimentary deposits that create floodplains is directly attributed to the river system. The wandering path of river channels and their meander patterns leads to lateral erosion of the river banks. Since the channel size will remain constant for a given discharge and slope, erosion on one bank requires deposition on the opposite bank.[51] In this manner, the streams shift their position from side to side with occasional movement all the way to the valley walls. This process gradually widens the valley and eventually creates a plain. The plain is composed of the material that was deposited by the river as it eroded sideways and filled its old channel. This material is called alluvium. The elevation of the plain will not be far above the river, as all of its material was deposited by the river. Although floodplains are usually associated with a mature meandering stream, it is important to note that even a wandering, straight channel can

create a floodplain, as well as stable channels and those gradually eroding or filling their courses.[95]

5.2 FLOODPLAIN DEPOSITS

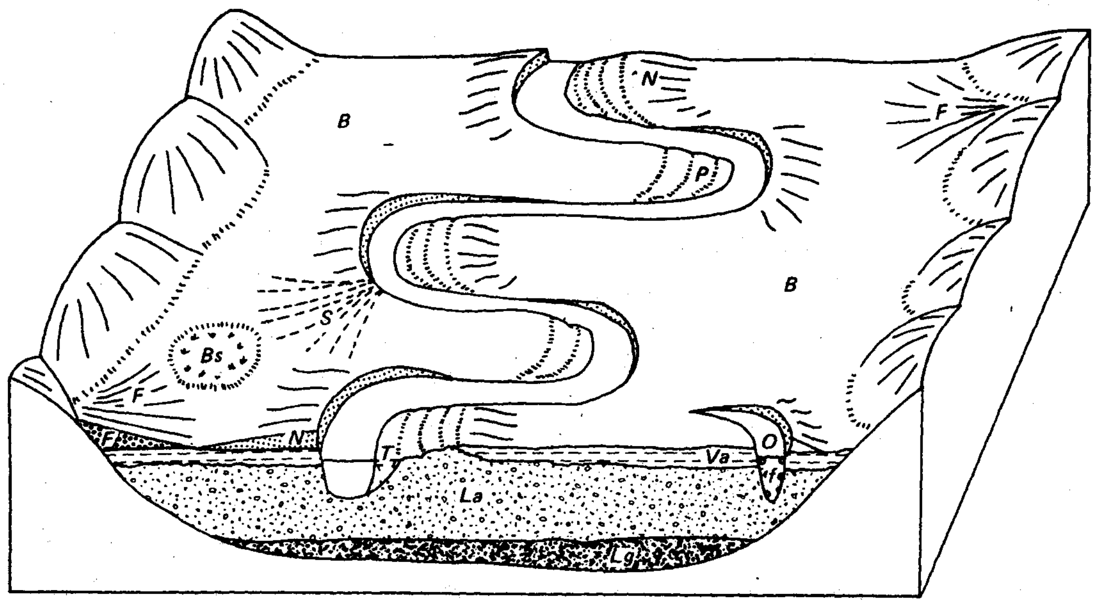
There are several types of floodplain deposits that have been identified by Harp,[18] and Mackin,[45] as described below and shown on Figure 4.

a. Channel fill deposits occur when the stream is unable to convey the full sediment load and is forced to deposit portions of it. It is the net result when local fill rates exceed scour rates. The material is left in the channel bed, gradually filling the bed and raising the bed or forcing the bed to shift laterally towards the sides as the center is filled. Mid-channel deposits often are in the form of sand bars, or islands. That material is poorly sorted, irregularly placed, and rounded from abrasion.

b. Lateral accretion deposits are formed along the edge of the channel, particularly in the form of point bar deposits that occur on the inside of meander bends. They consist of coarse bed load sediments, and are frequently covered with finer vertical accretion material from later over bank flows.

c. Channel lag deposits are composed of very coarse material that accumulated in the streambed. They are usually found in steeper reaches or riffles where the finer particles were washed away by selective erosion. They are frequently left behind when a channel shifts its course, and are covered with lateral accretion deposits of the shifting channel.

d. Vertical accretion deposits are the result of the over bank flood flows that are unable to convey sediments in areas of low velocity. The coarser particles settle first, along the edge of the floodplain nearest the channel and form natural levees. The finer particles are carried further from the channel and deposited in stratified layers over the entire floodplain. The material is almost entirely from the suspended sediment load, and may be high in silt and clay.



Landforms and alluvial deposits in a valley: B, backland; Bs, backswamp; F, alluvial fan; f, channel fill; La, lateral accretion; Lg, lag concentrate; N, natural levee; O, oxbow lake; I, bank accretion; S, flood-plain splay; Va, vertical accretion.

Classification of Valley Sediments

Place of deposition (1)	Name (2)	Characteristics (3)
Channel	Transitory channel deposits	Primarily bed load temporarily at rest; part may be preserved in more durable channel fills or lateral accretions.
	Lag deposits	Segregations of larger or heavier particles, more persistent than transitory channel deposits, and including heavy mineral placers.
	Channel fills	Accumulations in abandoned or aggrading channel segments; ranging from relatively coarse bed load to fine-grained oxbow lake deposits.
Channel margin	Lateral accretion deposits	Point and marginal bars that may be preserved by channel shifting and added to overbank flood plain by vertical accretion deposits at top.
Overbank flood plain	Vertical accretion deposits	Fine-grained sediment deposited from suspended load of overbank flood water; including natural levee and backland (backswamp) deposits.
	Splays	Local accumulations of bed load materials, spread from channels on to adjacent flood plains.
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.

Figure 4

From ASCE - Ref. 92

e. Floodplain sprays are fan-shaped deposits of coarse material left at points where the river flows overtopped low points along its banks. They are found at and behind breaks in the natural levels described above.

f. Colluvial deposits occur along the edge of the floodplain at the bottom of the valley walls. They consist of material from sheet erosion and talus of the valley sides, and are usually within 100 feet of the valley sides. They often include angular pieces of bedrock and unsorted till.

Other less common floodplain deposits include valley plugs, which are in abandoned channels filled with sediment when blocked by debris, driftwood, or sediment from tributaries. Meanders that are cut off become oxbow lakes, and eventually fill with fine grain sediments that are called "clay plugs."[51] A U.S.G.S. report by Wolman and Leopold indicates that they feel most floodplain deposits are due to channel deposits.[95]

Although active floodplains are often thought of as being in an aggrading stage, U.S.G.S. data from North and South Carolina found that the mean depth of floodplain deposits was proportional to the distance from the headwaters. In addition, it appeared that the scour depths of the river would be sufficient to reach the bottom of the deposit during major floods. Thus, an active floodplain, subject to periodic change, may be in a stable condition without aggradation. The fine sediments periodically deposited on the floodplains by minor over bank flows can be washed away by major floods, without any net aggradation.[95]

Since the river channel scour may well reach to bedrock during peak floods, the bedrock of the valley bottom may be slowly degrading even though an active floodplain exists.

A "stable" floodplain has erosion volumes roughly equal to fill deposits, and this is interrupted only when sediment supply is increased from upstream or by channel erosion at the valley walls.

5.3 SECONDARY DEPOSITS

The initial floodplain deposits are often reworked by the river as it meanders back across the floodplain.[16] This process tends to erode and wash away the finer particles and leaves the heavier, coarser material behind. As a result, the deeper, older deposits become very coarse in relation to the newer surface deposits that have not been subject to as many periods of erosion[51]. The constant reworking of the deposits may result in a material that differs from the original sediment load.[45]

Some floodplains subject to high flow rates and velocities can be composed mainly of the channel fill, lateral accretion, and lag deposits with little of the finer grained vertical accretion deposits.[95] In other floodplains, the situation may be reversed.[45] The in-channel deposits may account for as much as 80 to 90 percent of the floodplain deposits, with vertical accretion deposits making up the remainder.[16]

The deposits that form in the channel as bars or islands have stratified layers that are inclined at the angle of the downstream face of the bar. Vertical accretion deposits form horizontal stratas that may be continuous across much of the floodplain.[44]

5.4 ALLUVIAL TERRACES

Alluvial terraces are a landform created when rivers degrade or down cut a deeper channel, abandoning their former floodplain at an elevation above new flood levels. Wolman and Leopold say "a floodplain becomes a terrace when the channel incises itself to a point where the former floodplain is no longer overtopped by the annual flood, which on the average occurs less than once every two years."[95] They state that "the floodplain can only be transformed into a terrace by some tectonic, climatic, or man-induced change which alters the regimen of the river, causing it to entrench itself below its floodplain."

Terraces consist of a flat bench of floodplain deposits, separated from the active floodplain or channel by a steep scarp. The terraces

may be paired with one on either side of the valley at equal elevations, or may be singular. There may be more than one level of terraces, of different ages.[51]

5.5 MISFIT STREAMS

Those streams whose natural meander lengths do not match those of its valley are said to be "misfit streams." They occur where the meander length and amplitude of the valley were formed by flow conditions that were different from those of the present stream. A typical case would be where climatic conditions or glacier melt waters created high flows that carved the valley to proportions larger than possible with present flows.[51]

Misfit streams often occupy only a portion of the valley bottom, and if downcutting, may leave large terrace areas at the former floodplain level. The curves of the valley or upper terrace scarps can give an indication of the former meander sizes and discharge rates.[25] Note that the curve radius and curve length for valley meanders must follow a repeating pattern to identify valley meanders cut by the river from irregular winding valleys.[14]

Misfit streams are called underfit streams when their meander lengths are smaller than those of the valley. They are the result of lower flow rates than those which existed at the time the valley was formed.[14]

The ratio of the valley meander length to the stream meander length varies from three to one in the unglaciated Ozarks to up to ten to one in glaciated regions of Wisconsin. Dury indicates that a ratio of five suggests that the prior valley forming flows would be twenty times as large as the present flows.[16]

5.6 Entrenched Channels

Entrenched channels are those that have degraded to a level where the river does not have or does not frequently interact with a floodplain. Entrenched channels are confined by well defined banks that are

higher than the mean annual flood stage, thereby preventing the frequent inundation of the floodplain.

Entrenched meanders occur where the entrenched channel had a meandering pattern that was preserved as the channel degraded. By definition, entrenched meanders are those that have eroded vertically, but not laterally. They are identified by steep valley walls on both sides of meander bends.

Incised meanders occur where the channel has eroded both vertically and laterally. The incised meanders are moving downstream by eroding the outside of the bends as well as eroding vertically. They are identified by steep valley walls on the outside of bends, with mild sloping walls on the inside of bends.[14]

6.0 SEDIMENT MOVEMENT

A thorough knowledge of sediment movement is required to understand the dynamics of both natural rivers and man-made channels. Our knowledge of sediment movement is a combination of empirical and theoretical data, and neither has been able to provide consistently accurate results. However, present knowledge does provide a basis for estimating sediment movement rates and volumes, as well as the location of areas subject to scour and deposition. The sediment movement process begins with the erosion of surface soils in the watershed, and movement of soil particles by surface runoff to rivers. The subsequent movement of sediments through the river system to the oceans is a complex process, generally including many cycles consisting of the initiation of movement, transport, and deposition.

The initial erosion of land areas is not discussed in this report, as it is not caused by rivers and is discussed in great detail in other publications. The initiation of movement is a key factor in analyzing rivers and designing channels, and therefore this report summarizes the practical methods available. Several sediment transport methods that can be applied with limited data are also summarized.

6.1 INITIATION OF MOVEMENT

The movement of sediments in a river occurs when the flow of water exerts sufficient force on the individual particles to move them. The driving force is due to the drag caused by the flow and related lift forces: the resistance forces are the weight of the particle, friction, and cohesion.

Critical threshold conditions are said to occur when motion of the particle is imminent. The critical threshold condition is difficult to measure because not all particles will begin to move at the same time. It is therefore somewhat subjective, and various values are reported.

The two procedures in wide use for determining critical threshold conditions are the velocity method and the critical stress method.

6.2 THRESHOLD VELOCITY METHODS

The simplest method of estimating when sediment movement begins is by use of empirical test data that is the result of observing visible movement of various size sediment particles versus the flow velocity. The terms "critical" and "threshold" velocity are used to define when sediment motion begins. The term "permissible" velocity is used to define the maximum safe velocity without movement of sediment particles, and is essentially equal to the threshold velocity. The plotted data relating flow velocities to particle sizes and sediment movement often has scatter to it, in part because it is difficult to accurately define when general movement begins.

The value of the threshold and/or permissible velocity can be estimated from several different tables, charts and formulas based upon laboratory and field data, as summarized hereafter.

6.21 Fortier and Scobey developed and published one of the most widely used tables of permissible velocities in 1926 (see Figure 5). Their data has been published and used by the U.S. Bureau of Highways, ASCE, Chow and others. The data is from seasoned canals that had flow depths of up to 3 feet and were relatively stable. Note that their table of permissible velocities distinguishes between three different flow types: clear water, flow with colloidal silts (wash load), and flow with a bed load of sand and gravel.

6.22 Recommended maximum permissible velocities were published in Russia in 1936, and reproduced by Chow with widespread usage.[11] The Russian data covers a wide range of soil particle sizes from clay soils of various densities to 4 inch diameter cobbles. The values are accompanied by a chart of correction values for use with variable flow depths of one to ten feet. This recognizes that bottom velocities lag behind the mean velocities as the depth increases, and thus deeper

Permissible canal velocities, according to Fortier and Scobey (1926)

Canal excavated in	Velocity, after ageing, of canals* carrying: Clear water (m/s)	Clear water (ft/s)	Water-transporting colloidal silts (m/s)	Water-transporting colloidal silts (ft/s)	Water-transporting non-colloidal silt, sands, gravels, or rock fragments (m/s)	Water-transporting non-colloidal silt, sands, gravels, or rock fragments (ft/s)
Fine sand (non-colloidal)	0.46	1.50	0.76	2.50	0.46	1.50
Sandy loam (non-colloidal)	0.53	1.75	0.76	2.50	0.61	2.00
Silt loam (non-colloidal)	0.61	2.00	0.92	3.00	0.61	2.00
Alluvial silts when non-colloidal	0.61	2.00	1.07	3.50	0.61	2.00
Ordinary firm loam	0.76	2.50	1.07	3.50	0.69	2.25
Volcanic ash	0.76	2.50	1.07	3.50	0.61	2.00
Fine gravel	0.76	2.50	1.52	5.00	1.52	3.75
Stiff clay (very colloidal)	1.14	3.75	1.52	5.00	0.92	3.00
Graded, loam to cobbles, when non-colloidal	1.14	3.75	1.52	5.00	1.52	5.00
Alluvial silts when colloidal	1.14	3.75	1.68	5.50	0.92	3.00
Graded, silt to cobbles, when colloidal	1.22	4.00	1.83	6.00	1.52	5.00
Coarse gravel (non-colloidal)	1.22	4.00	1.68	5.50	1.83	6.50
Cobbles and shingles	1.52	5.00	1.68	5.50	1.98	6.50
Shales and hard pans	1.83	6.00	1.83	6.00	1.52	5.00

*Depth of 1 m or less.

Figure 5

From Chow - Ref. 5

channels can have higher mean velocities without scour.

6.23 Hughes has published recent data on threshold velocities in erodible channels and compared it with the data mentioned in Sections 6.21 and 6.22. Hughes' data is from some 300 natural streams in Colorado, New Mexico and Oklahoma for sandy-silt, silty clay, and clay soils. Flow velocities of 1 to 10 feet were used in the study with flow depths of 0.1 to 10 feet.

The data revealed that threshold velocities for each soil type formed a linear pattern when plotted on log paper. For sandy silts, Hughes' data is similar to the Fortier Scobey results, but are higher than the Russian values. Hughes' threshold velocities for cohesive soils (silty-clay, clay) are lower than the Fortier Scobey Table, but similar to the Russian values. Hughes feels this is because the cohesive soils tested by Fortier Scobey were very stiff and thus less erodible.[23]

The recent Hughes' data seems to have good agreement with applicable previous data and has the advantage of covering a wider range of flow depths, but has not yet been widely published or utilized.

The related equations for the threshold velocities, based upon Hughes data, are:

$$V = 2.12\ Y^{0.2}$$ for sandy silt, silt clay

$$V = 2.0\ Y^{0.2}$$ for silty clay

$$V = 2.69\ Y^{0.25}$$ for clay soils with a P.I. over 10

Y = flow depth, feet

V = velocity, feet per second

6.24 Mavis and Laushey developed a formula for maximum permissible velocities that is used by bridge engineers in Great Britain.[26] Their "competent velocity" is for actual velocities at the channel bottom:

$$V_b = 0.5\ (d)^{4/9}\ (s-1)^{1/2}$$

Where: V_b = competent velocity, ft /sec.

d = particle diameter, millimeters

s = specific gravity of particle

The Bureau of Reclamation used revised versions of the Mavis and Laushey equation in their manual "Design of Small Dams."[85]

$$V = 0.64\ (d)^{4/9} \qquad d < 6.0\ \text{mm}$$

$$V = 0.5\ (d)^{1/2} \qquad d > 6.0\ \text{mm}$$

6.25 Neill suggested the theoretical permissible mean velocities for stable channels that have been published by the Canadian Roads and Transportation Association.[53] For gravel and cobbles, Neill mathematically combined Shields tractive stress equation and the shear velocity to solve for the permissible velocity:

$$V = 143\ (y)^{1/3}\ (d)^{2/3}$$

Y = flow depth, feet

d = grain size, feet

It is intended for uniform flow in wide, straight channels.

For cohesive soils, Neill based his suggested permissible velocities on critical shear stress data as noted below:

$$V_b = 7.5\ (y)^{1/6}\ (\tau_o)^{1/2}$$

where τ_o is the critical shear stress (see section 6.33).

6.26 The U.S. Army Corps of Engineers uses the Permissible Velocity Method to estimate when a channel will be subject to erosion.[87] Their values of permissible velocities were agreed upon at a series of conferences between 1958 and 1960. The values given (see Figure 6) are generally higher than the Fortier, Scobey and Hughes' values which are presented in a similar form. The Corps of Engineers also use the tractive force method, which is discussed in Section 6.3, and refer to the ASCE for the other information on critical velocities.

Suggested Maximum Permissible Mean Channel Velocities

Channel Material	Mean Channel Velocity, fps
Fine sand	2.0
Coarse sand	4.0
Fine gravel††	6.0
Earth	
Sandy silt	2.0
Silt clay	3.5
Clay	6.0
Grass-lined earth (slopes less than 5%)‡	
Bermuda grass - sandy silt	6.0
- silt clay	8.0
Kentucky Blue Grass - sandy silt	5.0
- silt clay	7.0
Poor rock (usually sedimentary)	10.0
Soft sandstone	8.0
Soft shale	3.5
Good rock (usually igneous or hard metamorphic)	20.0

Figure 6

From U.S. Army Corps of Engineers

Ref. 87

6.27 The ASCE Sedimentation Committee has published in their recent Sedimentation Engineering Manual a curve for threshold velocities as a function of grain size (see Figure 8). For silt and clay particles, ASCE uses the Hjulstrom curve, part of which is based on data from Fortier and Scobey. For sand, they use the Mavis and Laushey equation which ASCE considers to be conservative.

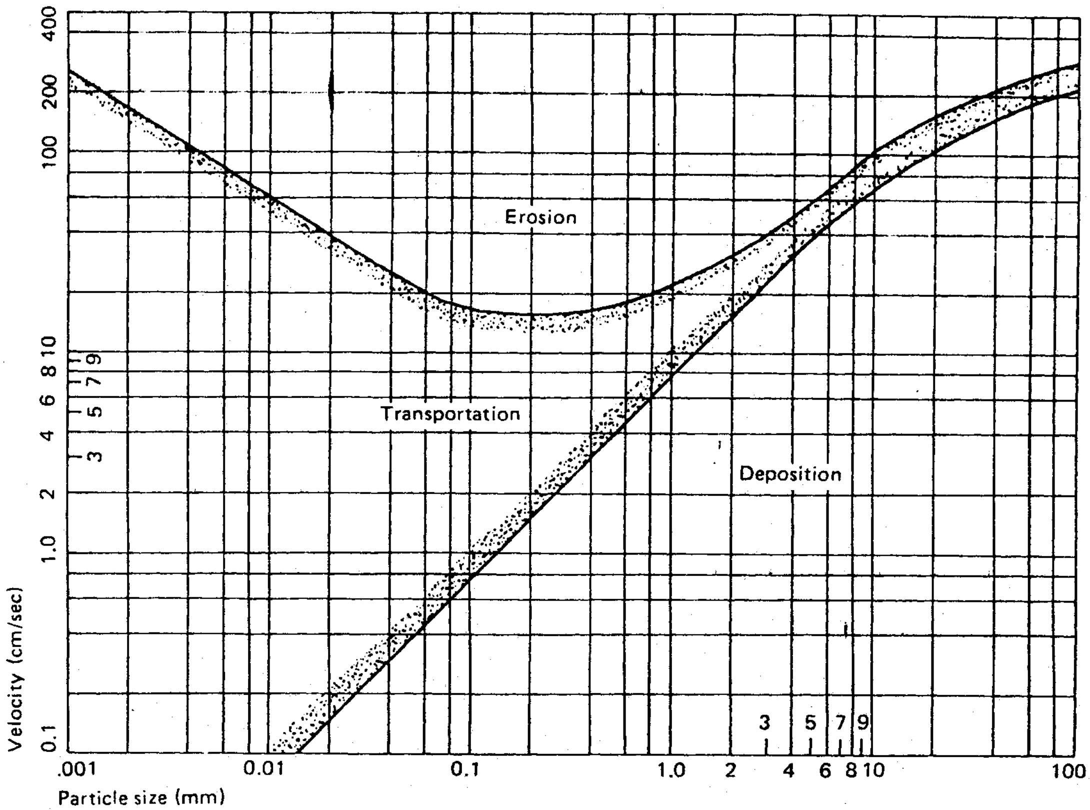
6.28 The U.S. Department of Agriculture Soil Conservation Service (S.C.S.) has recently revised their procedure for determining non-eroding velocities in earth channel. They determine their basic allowable velocity from the combined data of Fortier, Scobey, Lane, and from the Russian publications while distinguishing between clear water and sediment laden flow.[91]

For cohensionless particles of coarse sand and larger, the basic velocity from the S.C.S. chart (see Figure 9) is multiplied by correction factors for the flow depth, bank slope and alignment to determine the actual permissible velocity. They are thus modifying the original data in order to account for these additional factors which are often ignored.

The permissible velocity for fine grained cohensionless soils and for cohesive soils with a P.I. less than 10 is set by S.C.S. at a fixed value of 2 feet per second.

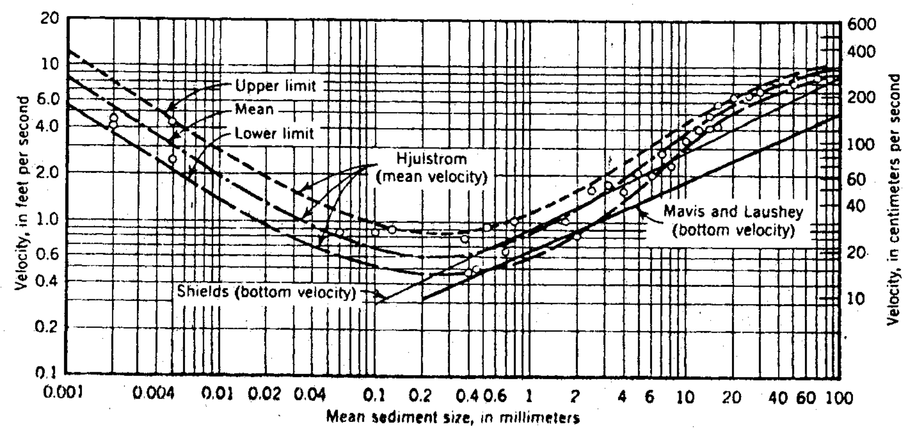
For highly plastic cohesive soils (P.I. $>$ 10), the permissible velocity is the basic curve value multiplied times correction coefficients for the depth of flow, alignment, frequency of flow, and density (see Figure 10).

6.29 The ASCE Committee on Sedimentation felt that the various data on threshold and permissive velocities have significant variation and poor agreement. They mention that one reason for this is the fact that the threshold velocity for a given soil type does vary with the depth of flow. They thus favored use of tractive force types of equations to define when sediment movement would begin.[92]



The effect of velocity on particle size in erosion, transportation, and deposition.

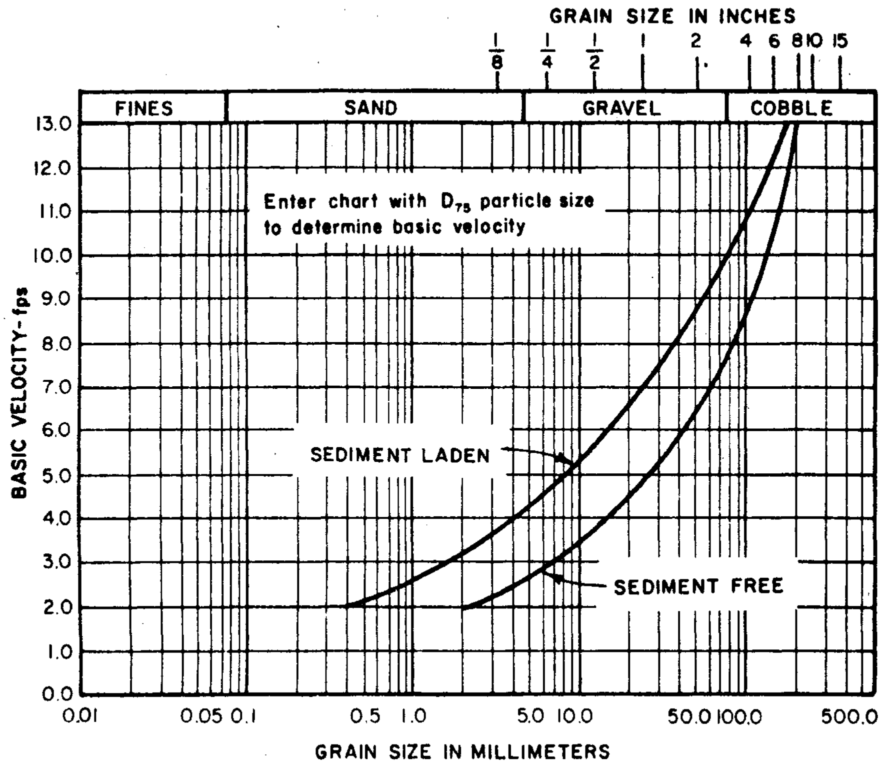
Figure 7
from Ruhr
Ref. 64



—Critical Water Velocities for Quartz Sediment as Function of Mean Grain

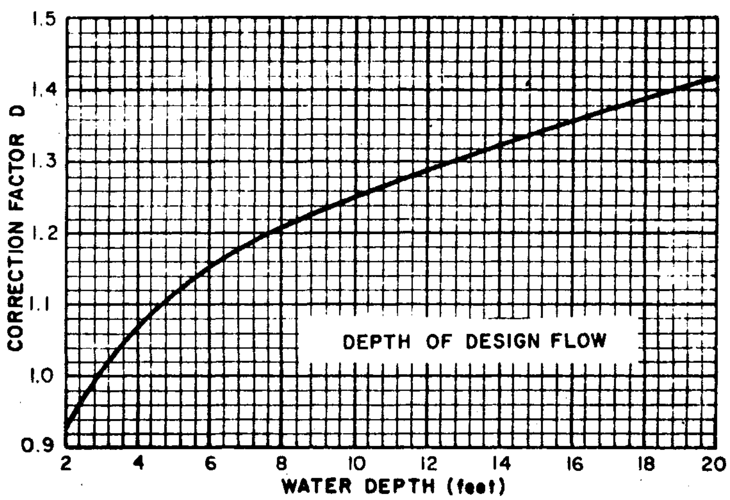
Figure 8
from ASCE
Ref. 92

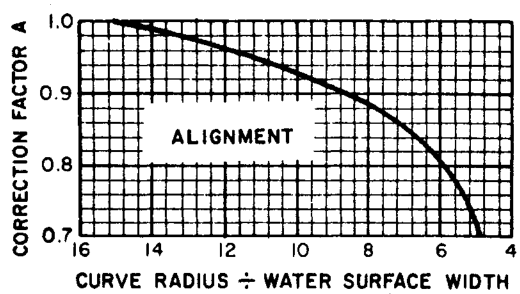
ALLOWABLE VELOCITY - NON COHESIVE SOILS - SCS PROCEDURE



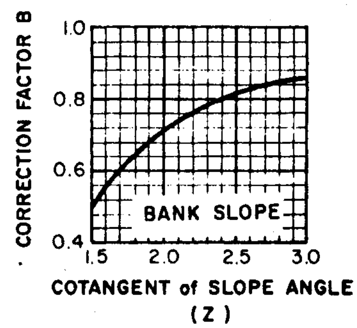
BASIC VELOCITY FOR DISCRETE PARTICLES OF EARTH MATERIALS (v_b)

THESE FIGURES ARE REPRODUCED FROM "DESIGN OF OPEN CHANNELS", T.R.-25 U.S. SOIL CONSERVATION SERVICE, OCT. 1977





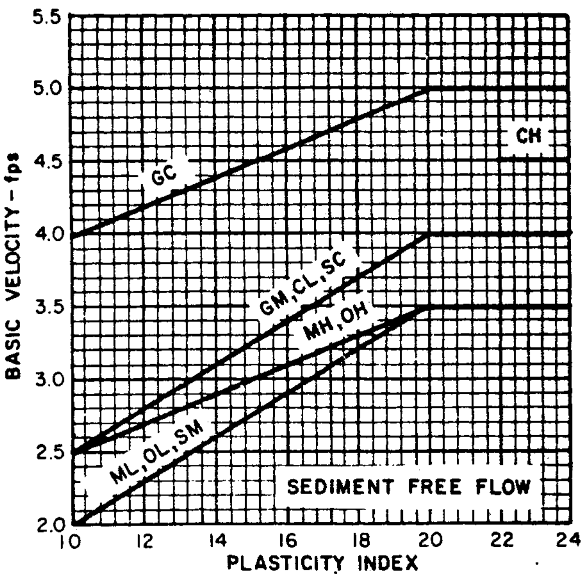
CHANNEL BOUNDARY MATERIALS	ALLOWABLE VELOCITY
DISCRETE PARTICLES	
Sediment Laden Flow	
$D_{75} > 0.4$ mm	Basic velocity chart value x D x A x B
$D_{75} < 0.4$ mm	2.0 fps
Sediment Free Flow	
$D_{75} > 2.0$ mm	Basic velocity chart value x D x A x B
$D_{75} < 2.0$ mm	2.0 fps



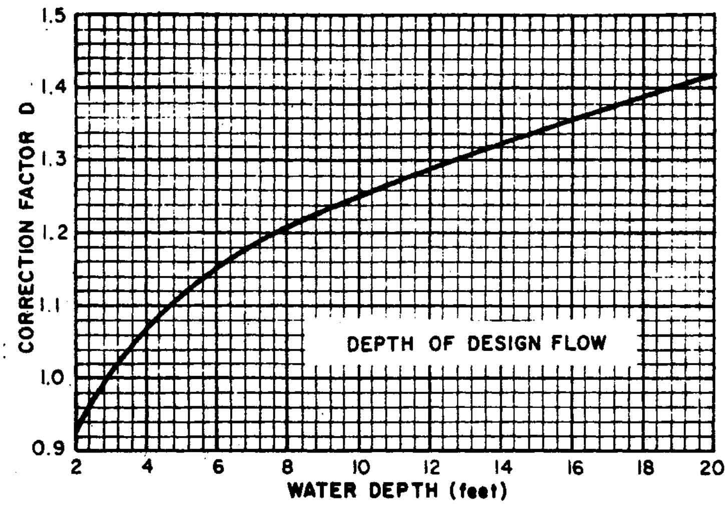
1) PARTICLE SIZE D75 = ______ VB = ______
2) DEPTH OF DESIGN FLOW = ______ CORRECTION FACTOR D = ______
3) RADIUS ÷ WIDTH = ______ CORRECTION FACTOR A = ______
4) COT. SLOPE ANGLE = ______ CORRECTION FACTOR B = ______
5) ALLOWABLE VELOCITY = VB X D X A X B = ______X______X______X______=______FPS

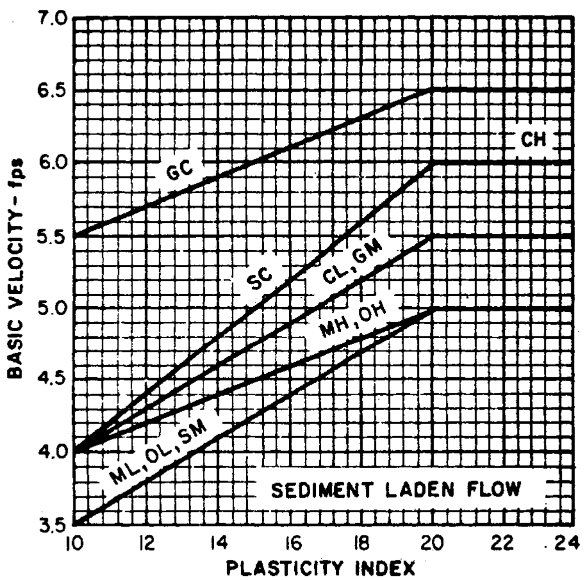
Figure 9 from SCS Ref. 90

ALLOWABLE VELOCITY - COHESIVE SOILS - SCS PROCEDURE

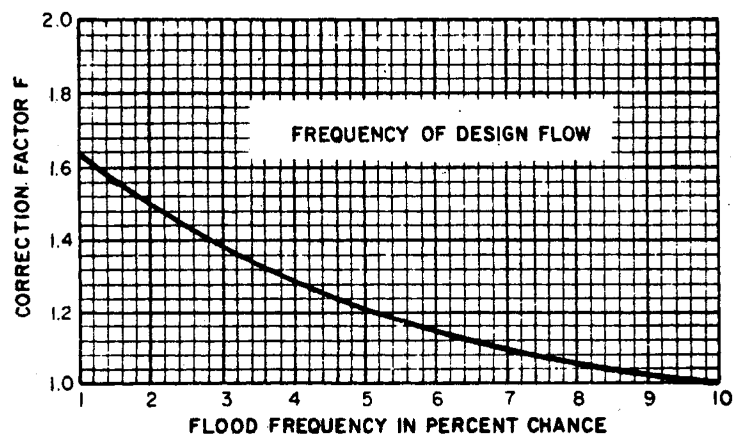


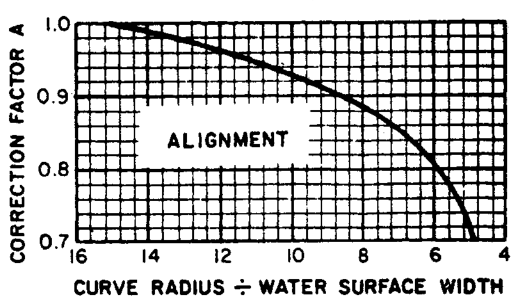
BASIC VELOCITIES FOR COHERENT EARTH MATERIALS (v_b)

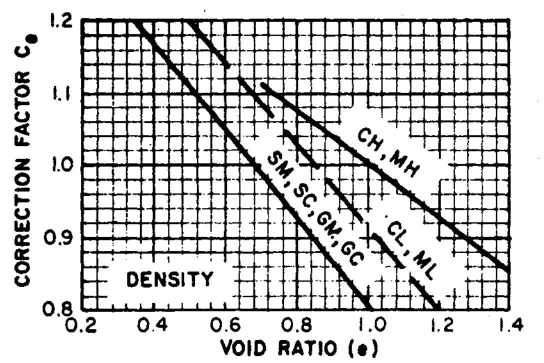




BASIC VELOCITIES FOR COHERENT EARTH MATERIALS (v_b)







A) FOR P.I. < 10, ALLOWABLE VELOCITY = 2.0 FPS

B) FOR P.I. > 10, 1, P.I. = ______ VB = ______

DEPTH OF DESIGN FLOW = ______ CORRECTION FACTOR D = ______

RADIUS ÷ WIDTH = ______ CORRECTION FACTOR A = ______

FLOOD FREQUENCY, % = ______ CORRECTION FACTOR F = ______

VOID RATIO E = ______ CORRECTION FACTOR CE= ______

ALLOWABLE VELOCITY = VB X D X A X F X CE = ______ FPS

Figure 10 from SCS Ref. 90

However, since publication of the 1975 ASCE manual, new data of high quality has become available. The Hughes' data on threshold velocities for the beginning of scour include the effect of flow depths, and the Soil Conservation Service has adopted a threshold velocity procedure with correction factors for flow depth, bank slope and alignment. In addition, Simons, Neill, and others have solved theoretical equations for threshold velocities based on allowable shear and flow depths.

The result of these recent studies tends to renew confidence in the threshold velocity approach as a reliable method to estimate when sediment movement begins.

Sections 7.52 and 7.61 of this report give directions on how to apply the threshold velocities in estimating scour depths in channels and at bridges.

6.3 TRACTIVE FORCE METHOD

The tractive force (also called the shear force or shear stress) is the downstream force on the river bed and bank material due to the flowing water. The individual particles on the bed and banks will have motion initiated when the tractive force exceeds a maximum critical value at which the particles are stable. By finding the applied tractive force for a specified flow condition and the critical tractive force of a given particle, one can determine if the particle will be moved by the water.

The average tractive force is:[92]

$$\tau_o = \gamma_w RS$$

Where τ_o = average tractive force

γ_w = weight of water, pounds per foot^3

R = hydraulic radius

S = river bed slope

The maximum tractive force on the bed of a trapezoidal channel approaches the value of:

$$\tau = \gamma_w YS$$

where Y = flow depth, feet

6.31 The tractive force of shear stress on a channel's perimeter varies with the geometry of the channel. For a uniform flow condition, the shear stress in narrow trapezoidal channels reaches a maximum in the center of the bed, and decreases near the side slopes. The shear stress on the banks reaches a maximum at the lower one third point. The stress on the channel bottom approaches that of infinitely wide channels when the base width is over twice the flow depth.

The U.S. Soil Conservation Service has published figures showing the ratio of actual maximum shear to the theoretical maximum shear stresses for trapezoidal channel beds and banks, for straight reaches at river bends, and immediately downstream of river banks. They are based on research work by the Bureau of Reclamation, and Lane.

The U.S. Department of Transportation (D.O.T.) uses a shear distribution pattern plotted at the University of Minnesota using Bureau of Reclamation data. The D.O.T. figures give the relation between the maximum shear on the bed and sides to the average shear stress.

The maximum shear stress on the sides of typical trapezoidal channels is about 80 percent of the bottom value for a side slope ratio of 1.5, and increases as the side slope becomes flatter.

6.32 Research at Colorado State University has found a correlation between the boundary shear stress and the average longitudinal velocity. This allows computation of the boundary shear stress using the convenient velocity term, as shown below:[75]

$$\tau = \left[\frac{V}{2.5 \ \ln \left(12.3 \frac{y}{d}\right)} \right]^2$$

V = mean velocity

Y = water depth

d = stone diameter

ρ = density of fluid sediment mixture

This formula was derived from the theoretical velocity distribution in turbulent open channels (by Einstein, Simons and Senturk) and the shear velocity definition.

The U.S. Army Corps of Engineers has a similar type of expression for the boundary shear stress, derived from the Chezy equation for flow velocity and an equation for hydraulic roughness.[87]

$$\tau = \frac{\gamma V^2}{32.6 \log_{10}\left(\frac{12.2y}{D_{50}}\right)^2}$$

γ = weight of water

The use of these equations can ease the use of the tractive force method in natural rivers with irregular shapes. This is because the shear can be computed based on average flow velocities for each subsection of the channel and floodplain cross section.

6.33 The theoretical critical shear stress is expressed as:

$$\tau_c = K\,(\gamma_s - \gamma)\ D$$

where τ_c = Critical shear stress

K = Constant

γ_s = Weight of particle

D = Size of particle

For practical purposes, most federal agencies involved in water resources have assigned values of the coefficient K and γs for various particle sizes and particle weights that are considered to be representative. For example, the U.S. Department of Transportation

uses the term:[88]

$$\tau_c = 5.0\ D_{50} \quad (D_{50} \text{ in feet})$$

The U.S. Soil Conservation Service uses the terms: [86, 91]

$$\tau_c = 5.0\ D_{50}$$

$$\tau_c = 4.8\ D_{75} \text{ for coarse sand and gravel}$$

The U.S. Army Corps of Engineers uses the term:[87]

$$\tau_c = 0.04\ (\gamma s - \gamma)\ D_{50}$$

by setting γ_s = 165 lbs./FT^3

$$\tau_c = 4.1\ D_{50}$$

The Shield's diagram is one of the best known sources of data on critical shear stresses. Shulits and Hill used a modified version of the Shield's diagram to develop equations of the critical shear stress assuming γ_s/γ to be 2.65:[76]

$$\tau_c = 0.0215\ D_s^{0.25} \qquad 0.0003 < D_s < 0.0009 \text{ FT}$$

$$\tau_c = 0.315\ D_s^{0.633} \qquad 0.0009 < D_s < 0.0018 \text{ FT}$$

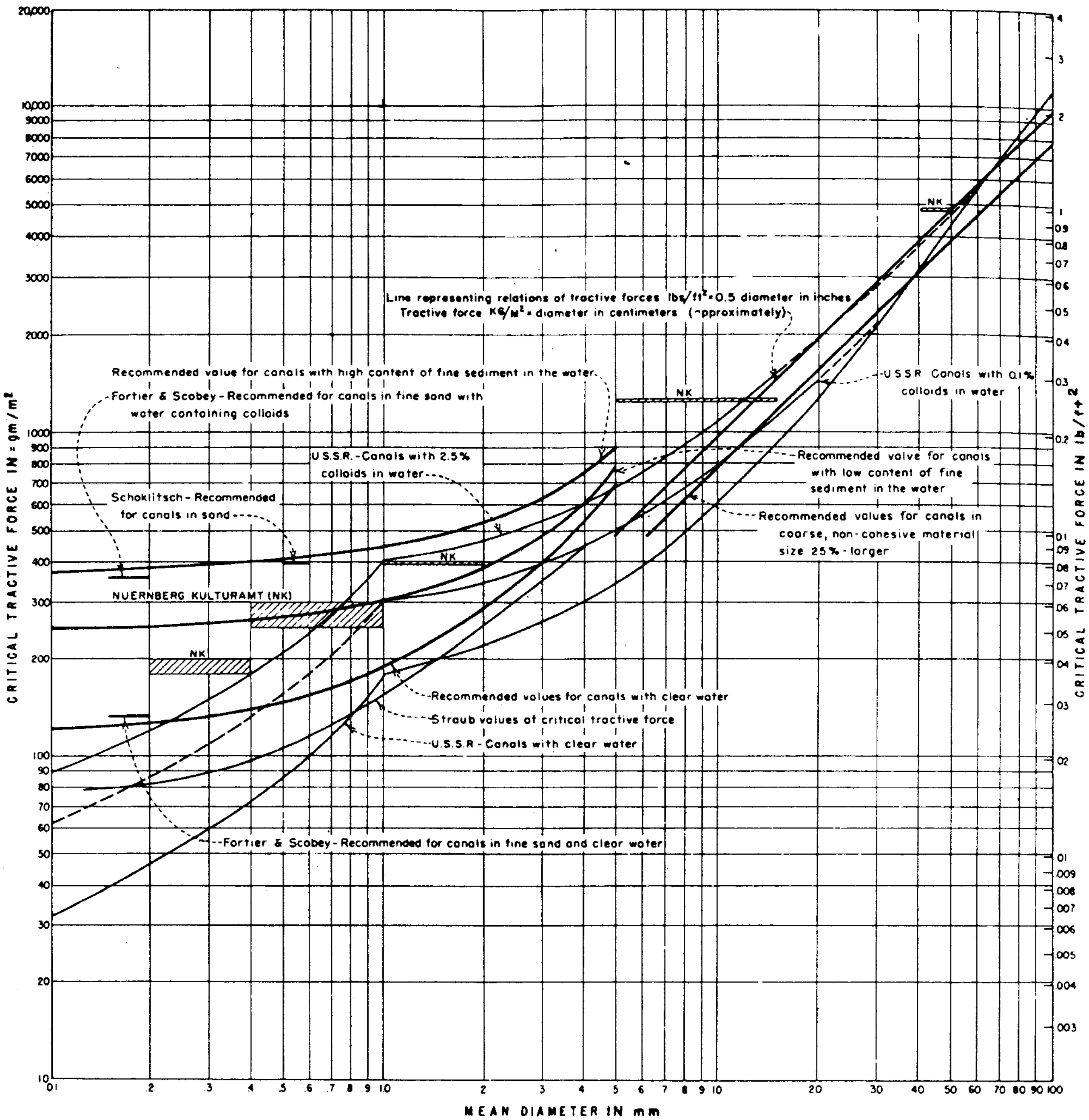
$$\tau_c = 16.8\ D_s^{1.262} \qquad 0.0018 < D_s < 0.022 \text{ FT}$$

$$\tau_c = 6.18\ D_s \qquad D_s > 0.022 \text{ FT}$$

Where D_s = Characteristic particle diameter

Lane has summarized much of the data on critical tractive forces on a graphical plot of the data in 1953 (see Figure 11).

6.34 For non-uniform soils, Egrazaroff has proposed an equation to define the point of initiation of motion for the mixed particles:[76]



Tractive force versus transportable sediment size.

Figure 11
from U.S. Bureau of
Reclamation
Ref. 85

$$\tau_c = \frac{0.1\ (\gamma s - \gamma)\ D_{50}}{\left[\text{Log } 19\ \frac{D_{50}}{\bar{D}}\right]^2}$$

Where $\bar{D}$ = the average particle size

D_{50} = the median particle size

6.35 On the sloped surfaces of channel banks, the lateral slope reduces the critical shear stress and thus reduces the stability. The critical shear stress of channel banks is given below:[88]

$$\tau_{cs} = \tau_c \left[\sqrt{1 - \frac{\sin^2 \phi}{\sin^2 \theta}}\right]$$

τ_{cs} = critical shear on channel slope

In the above equation, used by The Department of Transportation and others, ϕ is the angle of repose of the material and θ is the angle of the channel bank. The Soil Conservation Service uses the modified version below:[91]

$$\tau_{cs} = \tau_c \left[\sqrt{\frac{z^2 - \cot^2 \phi}{1 + z^2}}\right]$$

Where z is the side slope ratio of the horizontal distance and vertical distance.

6.36 The U.S. Bureau of Reclamation developed a simplified method to define the size of stable channels based on tractive force theory. As reported by Simons, terms are:[76]

$$A = \frac{2\ d^2}{\tan\ \emptyset}$$

$$d = \frac{\tau_c}{0.97\ S}$$

$$T = \frac{d\ \pi}{\mathrm{Tan}\ \emptyset}$$

A = area of flow

d = maximum depth

$\emptyset$ = angle of repose

T = top width

6.4 SEDIMENT TRANSPORT

There are numerous methods available and in use for predicting sediment transport rates in rivers. However, none of the methods developed to date are considered as completely describing or predicting the sediment rates for all flow and sediment grain size conditions.

Visual inspection and random spot sediment samples are not reliable methods to determine if a river is transporting sediment. Much of the total sediment movement, particularly of the bed material, occurs only during periods of high flows at or above the bankfull stage. The bulk sediment movement can thus be limited to only a few days a year.

The various sediment transport formulas have been derived from both laboratory flume tests and field measurements of streams and rivers. The methods developed by Duboys, Schoklitsch, Shields, Laursen, Meyer, Peter and Muller are all related to the concept of critical shear stresses on the bed material. The methods proposed by Colby and Inglis-Lacey are based upon the concept of threshold velocities, while the Einstein and Brown methods of analysis are from the concept of having fluid-like layers of sediment in turbulent motion along the river bed.

The writer has selected two of the above mentioned sediment transport prediction methods for further discussion. The Schoklitsch method was selected for discussion because the U.S. Soil Conservation Service refers to it as "one of the more extensively used empirical formulas" and it is easy to apply with limited data.[86]

The Colby method, which has been adopted by the U.S. Army Corps of Engineers for rough estimates of sediment transport capacity, is also discussed.[87] It appears to most accurately predict sediment discharge rates in the field tests performed by Vanoni, Brooks, and Kennedy in South Carolina, Nebraska, and Arizona. The tests of field measurements of sediment loads versus predicted levels indicate that the methods by Shields, Einstein-Brown, and DuBoys tend to give high results, while the Meyers-Peters formula gave low results.

Other writers have their own preferred methods. For example, Shen recommends that the following methods be used:[72]

a. Einstein's 1950 method be used when the bed load is a high percentage of the total load
b. Colby's 1964 method for small rivers with flow depths of up to 10 feet
c. Toffaleti's method for large rivers

6.41 The Schoklitsch formula for predicting the rate of bed load sediment transport was published in London by Shulits in 1935. The formula is based upon data from flume experiments conducted by G.K. Gilbert (U.S.G.S.) in 1914. It is expressed as:[86]

$$G_s = \frac{86.7}{\sqrt{d_{50}}} \left(S_e^{1.5} \right) (q - q_o)$$

Where: $q_o = 0.00532 \left(\dfrac{d_{50}}{S_o^{1.33}} \right)$

G_s = sediment discharge, pounds per second per foot of width

d_{50} = median grain size, inches

Se = energy gradient of the river

q = unit flow, DFS per foot of width

q_o = unit flow at incipient particle motion

The American Society of Civil Engineers had published a modified version of the equation that accounts for variations in particle size. It is:

$$G_S = \sum P \left(\frac{86.7}{\sqrt{d}} \right) S^{1.5} (q - q_o)$$

The term "P" is the percentage of particles of the size "d". The equation is used to solve the sediment rates for each major fraction of the range of particle sizes, and the values are summed to arrive at the total load.

The Schoklitsch equation was found to fit Gilbert's flume data for mean particle sizes of 0.33 mm to 7 mm, with bed slopes of 0.004 to 0.030 feet per foot. A.S.C.E. suggests that the equation should not be used on streams that carry much of the bed material in suspension. The best results are on river sections of near constant depth and straight alignment.[92]

6.42 The Colby method of estimating the bed material sediment transport rates for sandy soils is based upon empirical graphs of flume and river data (see Figures 12 and 13). The basic values of the transport rates of sand (G), in tons per unit width, are plotted on a series of curves for four flow depths, six median grain sizes, and velocities of one to ten feet per second.

To use the charts, one must first know the median grain size, flow velocity, and depth of flow. The grain sizes that are applicable

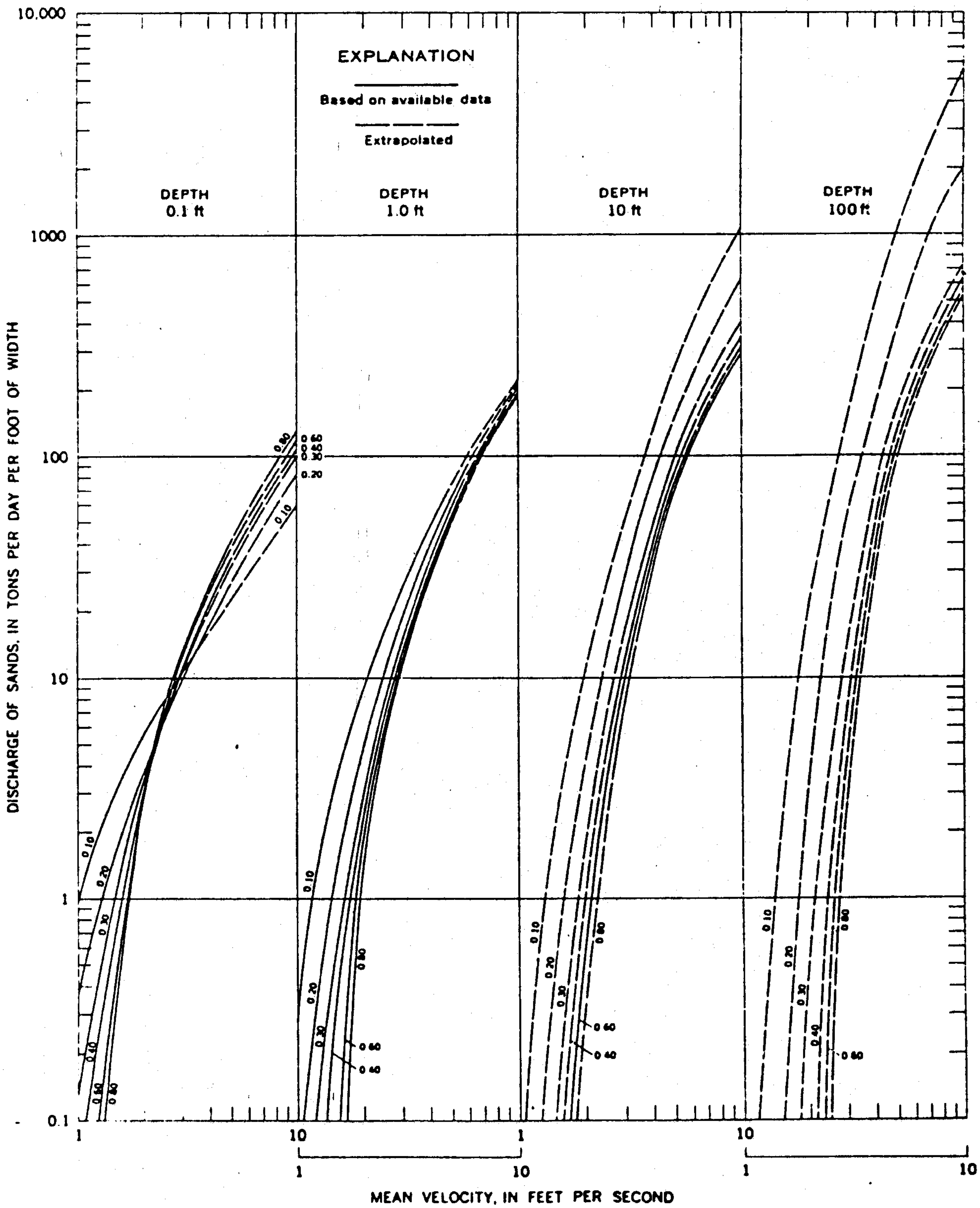
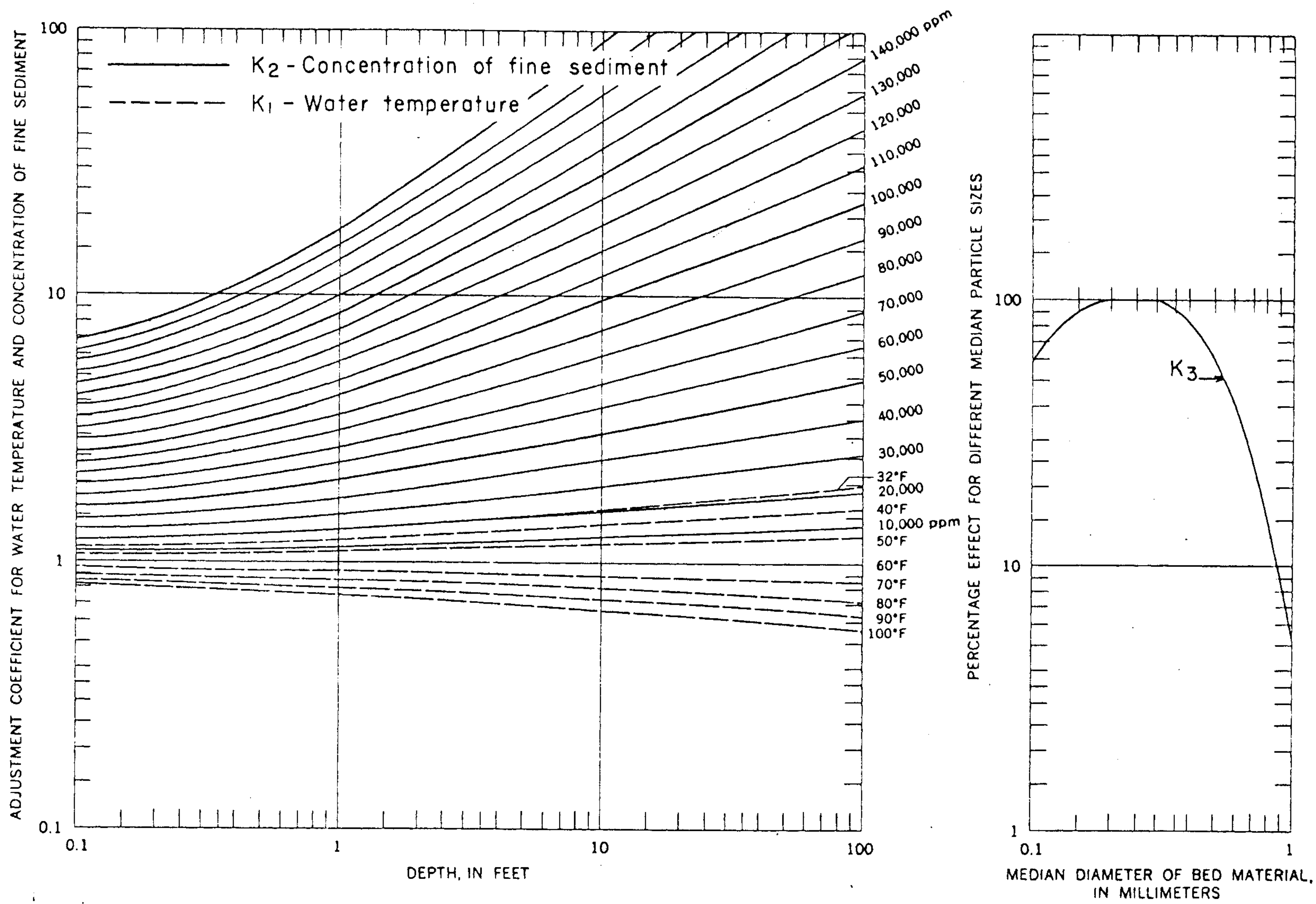


Figure 10.—Relationship of discharge of sands to mean velocity for six median sizes of bed sands, four depths of flow, and a water temperature of 60° F. (From Colby, B. R., Discharge of Sands and Mean-Velocity Relationships in Sand-Bed Streams; U.S. Geol. Survey Prof. Paper 462-A).

Figure 12



Approximate correction factors for the effect of water temperature and concentration of fine sediment and sediment size on the relationship of discharge of sands to mean velocity (From Colby, 1964, p.A31)

Figure 13

are 0.10, 0.20, 0.30, 0.40, 0.60, and 0.80 mm. The curves are for flow depths of 0.1, 1.0, 10.0, and 100 feet. For actual depths between these values, the sediment transport rates are plotted on log-log paper for depths below and above the desired depth, and interpretation used (on log-log paper) to find the transport rate at the desired depth.

The solution obtained from the figures may be refined by use of three coefficients for temperature (K1), concentration of fines (K2), and deviation of mean grain size (K3). The final formula becomes:

$$G_s = [1 + (K_1 K_2 - 1) \; 0.01 \; K_3] \; G$$

6.43 The American Society of Civil Engineers states that there can be wide discrepancies between the predicted and actual sediment transport rates. They feel that the best use of the various prediction methods is not in determining absolute sediment loads, but in determining the transport rates in the river at the desired points for comparison.[92] The higher values will indicate points subject to scour, while the lower values indicate areas subject to sediment deposition.

7.0 PROFILE STABILITY AND SCOUR

The purpose of this portion of the report is to present information on the stability or equilibrium conditions of natural river beds over long periods of time, and the predicting of short term scour depths in channels and at bridges.

7.1 EQUILIBRIUM CONDITIONS

Alluvial rivers can adjust their bed elevation in response to discharge and sediment load so as to maintain equilibrium bed slopes. There will be periods of time during which the river is readjusting itself, and thus the equilibrium is said to be dynamic.

Schumm points out that the term "equilibrium" must be referenced to a particular time span, and that with respect to rivers it refers to the modern geologic period.[68] It is obvious that a river's "modern" equilibrium is not the same as it was during the glacial or preglacial periods of history when the climate was much cooler than at the present. In addition, over a great many years, surface erosion could reduce the drainage basin relief to where the river would alter its flow characteristics.

The River in equilibrium can exist even where scour and filling occurs, as these are only short-term processes that do not permanently change the river's mean altitude. Alluvial rivers in equilibrium can only exist for subcritical flow, as the supercritical flow is always unstable and does not allow deposition to occur.[65]

With the above factors in mind, Schumm indicates that the present basin relief, valley size, shape, and floodplains of our rivers were formed in the past, and are largely independent of the present flows. The size of the actual river channel and its slope are dependent upon the present conditions.

7.2. DEGRADATION

Degradation is a general lowering of the streambed elevation over a long period of time. It occurs in rivers where the slope, discharge,

and velocity are sufficient to transport more sediment than is supplied to the river section. The river will steadily erode its bed until the slope and velocity are reduced to a point where the river bed erosion no longer produces additional sediment.

The degradation can occur as a uniform lowering of the bed if the factors inducing the erosion are uniform. In other cases, the degradation can occur in the form of upstream migration of scarps or headcuts, called "kickpoints." In effect, two graded portions of the river are separated by a steep reach where active degradation occurs. The steep reach continuously erodes its base, and thus travels upstream.[42]

Natural degradation can be a result of an uplift of the land, climatic changes that increase runoff, or even an increase in vegetation that reduces the natural supply of sediment.

Degradation of the river will usually continue until a level of regime or equilibrium is reached. A common exception occurs in streams with a mixed grain bed material. In such cases, the river will tend to erode the fine grained sediments first, leaving heavier, coarser material behind. If the coarse material is large enough, it will resist erosion and prevent further degradation. The layer of coarse material covering the bed of the river is referred to as an "armored" layer. The river may then erode laterally if an armored layer prevents downcutting, or it may eventually reduce the size of the coarse particles by abrasion until they can be transported.

Research shows that the armor layer of coarse material on a stream bed can also exert a strong influence on the overall sediment transport rate of rivers.

During periods of rising flow rates, the fine particles are lodged between the coarser particles and thus there is a limited source of sediment. At high flows the armor layer prevents entrainment of underlying particles and thus limits sediment supply after the initial fines in/on the armored layer are removed.

Unusually large flows can dislodge and transport the armor layer particles and allow general bed movement to occur when underlying

particles are exposed. The largest particles of the armor layer will tend to sink to the bottom of the scour hole and be trapped there, and possibly be buried by later sediment deposits.

As high flow rates recede, the flow sorts the stream bed particles and forms a new armor layer out of the coarser particles too heavy to be moved by intermediate velocities. Further flow reductions then allow the voids between the armor particles to be filled in with fine grained sediments. Thus the armor layer can serve as a trap for fine sediments, during low flow rates, and they are the first particles moved when flow rates increase.[50] The overall rate of bed material transport can be very slow on streams with naturally armored beds, and they may tend to appear nonalluvial.[13]

7.3 AGGRADATION

Aggradation is the general increase in elevation of a long reach of a riverbed over a long-term period. It is a process of deposition in which sediment is continually added to the riverbed and possibly even to the floodplain. Aggradation occurs when the river does not have sufficient slope, velocity, or flow rate to transport the sediment load and sizes supplied to it. The riverbed and floodplain elevations will increase until equilibrium is reached with sediment transport being equal to sediment supply rates.

The downstream face of the aggrading reach has an increase in slope as the aggrading reach is raised above the previous bed level. This new steeper slope will increase with the sediment transport level until a new equilibrium is reached.

The slope of the riverbed upstream of an aggrading reach is reduced by the downstream deposits. This reduction in slope reduces sediment transport, and hence the area of deposition will migrate upstream, with progressively finer material being deposited due to the lower transport capacity. The sediments will be segregated by weight into stratified layers sloping downstream.[66]

The aggrading channel often becomes braided as it fills with new sediment, and overbank flows on the floodplain will occur with

greater frequency leaving vertical accretion deposits on the floodplain's surface. Thus the floodplain surface may rise with the river, but at a slower rate.

Among the factors that lead to aggradation are a decrease in watershed vegetation leading to increased surface erosion, climatic changes reducing runoff rates, where river water infiltrates into alluvium, and some phases of urbanization.

7.4 DEPTH OF SCOUR

Riverbed scour is a short-term erosion of the bed in a well defined reach, and differs from the degradation that occurs over a long-term period of time and a long river reach.

Scour is thus a local condition, and is often traced to an irregular flow cross-section, obstructions, constriction (natural or man-made), bridges, and piers. Among the factors are the river bed slope, bed material, flow velocity, flow rate, sediment load and size, and debris loads of ice or logs.[79]

The total scour that is present at a point is often the sum of several scour types. For instance, the scour under a bridge may include that due to contraction, obstruction, and geometry. Each must be estimated separately, then summed. Scour may also occur in combination with a general degradation of the riverbed.

General principles of scour are:[92]

a. The rate of scour is equal to the difference between sediment transport and supply.
b. The rate of scour will decrease as the flow section is enlarged by erosion.
c. For given initial conditions, there is always a maximum limit of scour.
d. The limit of scour will be approached asymptotically with respect to time.

There are numerous theories and methods concerning the prediction of river scour depths. The writer has categorized these into groups for scour in channels, scour at bridges, and scour at bridge piers (see

Figure 14). Predicting the depth or even the occurrence of scour is not an exact science, it is only beginning to be quantified, and there are still many conflicting theories.

Klingeman has identified the various parameters affecting scour and arranged them into three groups related to the stream, bridge structure, and sediments.[33]

a. Stream related parameters include discharge rates, variation in discharges, flood frequency and characteristics, flow depths, velocities and velocity patterns, channel alignment, channel gradient, channel cross-section, overbank flow conditions, and debris.

b. Parameters related to hydraulic structures include bridge size and orientation with respect to channel, pier size and shape, pier alignment and the pier spacing and number of piers.

c. Sediment parameters are the type of bed material, and the size distribution of the bed material.

All of the scour prediction methods will include some of the above parameters, and few will include all the parameters.

Some scour prediction methods are for "clear water" conditions with no sediment transport. These methods assume that the scour hole is not being refilled with sediment from upstream areas.

The "sediment laden" condition is where significant sediment transport does occur. Under this condition, sediment is continuously carried into the scour hole as well as from the scour hole. The hole gets deeper until the rate of sediment removal is equal to the rate of sediment inflow. These scour holes tend to be shallower than those occurring under clear water conditions.

For alluvial rivers scour holes in the main channel will usually occur under the "sediment laden" condition, and clear water formulas could over-estimate scour. Scour holes removed from the main channel would generally be of the clear water type due to low sediment loads.[12]

The side slope angle of scour holes is similar to the angle of repose of the material being eroded.

SUMMARY OF SCOUR PREDICTION METHODS

SCOUR TYPE	REPORT SECTION	METHOD OR AUTHOR	ORIGIN OF METHOD OR DATA
Channels	7.51	Lacey	Empirical regime relations
	7.52	Velocity	Empirical
	7.53	Straub, Griffith	Tractive force theory
	7.54	Laursen	Model tests, theory
	7.55	Bureau of Reclaimation	Tractive force theory
	7.56	Corps of Engineers	Bed load theory, field data
Bridges	7.61	Velocity	Empirical
	7.62	Inglis-Lacey	Empirical, regime
	7.63	Laursen	Model tests, theory
	7.64	Lui	Model tests
	7.65	Holmes	Field data
Bridge Piers	7.71	Inglis-Poona, Blench	Empirical regime relations
	7.72	Shen	Model tests
	7.73	Larras	Model tests, field data
	7.74	Neill	Model tests
	7.75	Callander-Laursen	Model tests, theory
	7.76	Canada (RTA)	Empirical - design data

Figure 14

7.5 SCOUR IN CHANNELS

7.51 The Regime Theory approach to fluvial geomorphology provided some of the earliest techniques to estimate scour depths. Lacey found that maximum scour depths ranged from 1.75 to 2.0 times the regime depth of Indian canals. Leopold's geological data supported this, indicating that depths of scour in alluvial, natural rivers could be 1.5 to 2.0 times the normal depth of flood flows.[95]

In a specific example of the regime technique for estimating scour, the Indian Bridge Code provides a series of empirical coefficients that are multiplied times the mean regime depth to estimate the maximum depth of scour. The mean regime depth is determined using the Regime Theory equations discussed in section 4.23.

Location	Factor
Straight reach of channel	1.25
Moderate bend	1.5
Severe bend	1.75
Abrupt 90 degree bend	2.0
Nose of round piers	2.0
Alongside cliffs and walls	2.25
Nose of guide banks	2.75

Neill explains that the Regime Theory coefficients for maximum scour at abrupt bends were determined by assuming that the normal cross-section area of an assumed rectangular channel section is transformed to a triangular channel cross-section of equal area and width. The depth of the triangular cross section is thus double that of the "equal" rectangle.

7.53 The depth of scour and cross-sectional area of flow in erodible channels with uniform flow may be estimated by the method of permissible velocity. This technique is also frequently used at bridges with few secondary currents. This is probably the most common method used in the United States to estimate when scouring conditions occur, and is often used under conditions of non-uniform flow where it is of questionable value.

The maximum possible velocity is defined by Chow as being the

"greatest mean velocity that will not cause erosion of the channel body."[11] If flow velocities are higher than the maximum permissible velocity, the river or channel will scour a greater waterway area until the larger flow area reduces the mean flow velocity below the maximum permissible velocity. At that point, scouring will cease. Specific values of permissible velocities are contained in section 6.2 of this report.

Lane recommends that the maximum permissible velocity be reduced in sinuous channels to reduce chance of scour. His reductions would be 5 percent for slightly sinuous channels, 13 percent for moderate sinuosity, and 22 percent for very sinuous channels.

The procedure for using this method has been described as follows:[53]

a. Determine the mean velocity for the design flow, assuming no scour.
b. Determine the non-eroding, maximum threshold velocity of the bed material.
c. Compare the mean velocity against the threshold velocity. If it is greater, scour is possible.
d. Assume a new depth of flow, with an estimated scour depth. Repeat items a, b, and c until the mean velocity is below the threshold velocity.

7.53 The depth of scour in long uniform channels with a gradually constricted width and uniform flow has been theoretically analyzed using the concept of boundary shear stress and critical tractive forces. By applying the continuity theory and DuBoys' equation of sediment motion, Straub found that the depth of scour in the constriction of a channel with sediment movement could be simplified to:[92]

$$d2 = d1 \left(\frac{b1}{b2}\right)^{0.642}$$

d = depth

b = width

Point 1 is upstream

Point 2 is downstream

For channels without a supply of sediment into the constriction, the scour depth is expressed as:

$$d2 = d1 \left(\frac{b1}{b2}\right)^{0.857}$$

The relations are theoretical, and assume that the bed in the constricted section will be scoured until the shear stress is reduced below the critical tractive force level, at which time the movement of the bed load will cease. The equations show that in a uniform sediment, with an upstream equilibrium, the depth in the constriction is proportional only to the upstream widths and depths.

It was also determined that although the theoretical scour was independent of grain size, the rate of scour was dependent on the sediment with fine soils eroding faster.

A similar result was obtained by Griffith using modifications of the basic regime equations and some field data. The only difference was the exponent, which Griffith found to be 0.637.[92] Use of Colby's bed material transport data, combined with Manning's equation and the same assumptions as Straub, leads to an equation with the same form, but an exponent of 0.67.[12]

Note that the above are only valid for long constrictions with uniform flow, and where uniform flow conditions exist in the upstream reach.

7.54 One of the more rigorous studies of scour in long channels was conducted by Emmett Laursen in Iowa. He used a large flume to conduct model studies that led to theoretical and empirical relations for scour depths under various conditions.[34]

For the case where the overbank (floodplain) flow is gradually contracted without upstream sediment supply, his equation is:

$$d_s = \left[\left(\frac{Y1^2}{120\ (Y1^{0.33})\ \ (D_{50}{}^{0.67})}\right)^{3/7} \frac{B1}{B2}{}^{6/7} - 1\right] Y1$$

Which can be simplified to:[81]

$$ds = \left[\left(\frac{QT}{Qc}\right)^{6/7} - 1\right] Y1 \text{ , OR } Y_2 = \left(\frac{QT}{Qc}\right)^{6/7} Y1$$

For a long gradual contraction of both the channel and overbank flow. Laursen proposed a simplified expression as:[35, 34]

$$ds = Y1\left[\left(\frac{B1}{B2}\right)^{K} - 1\right] \text{ , OR } Y_2 = \left(\frac{B1}{B2}\right)^{K} Y1$$

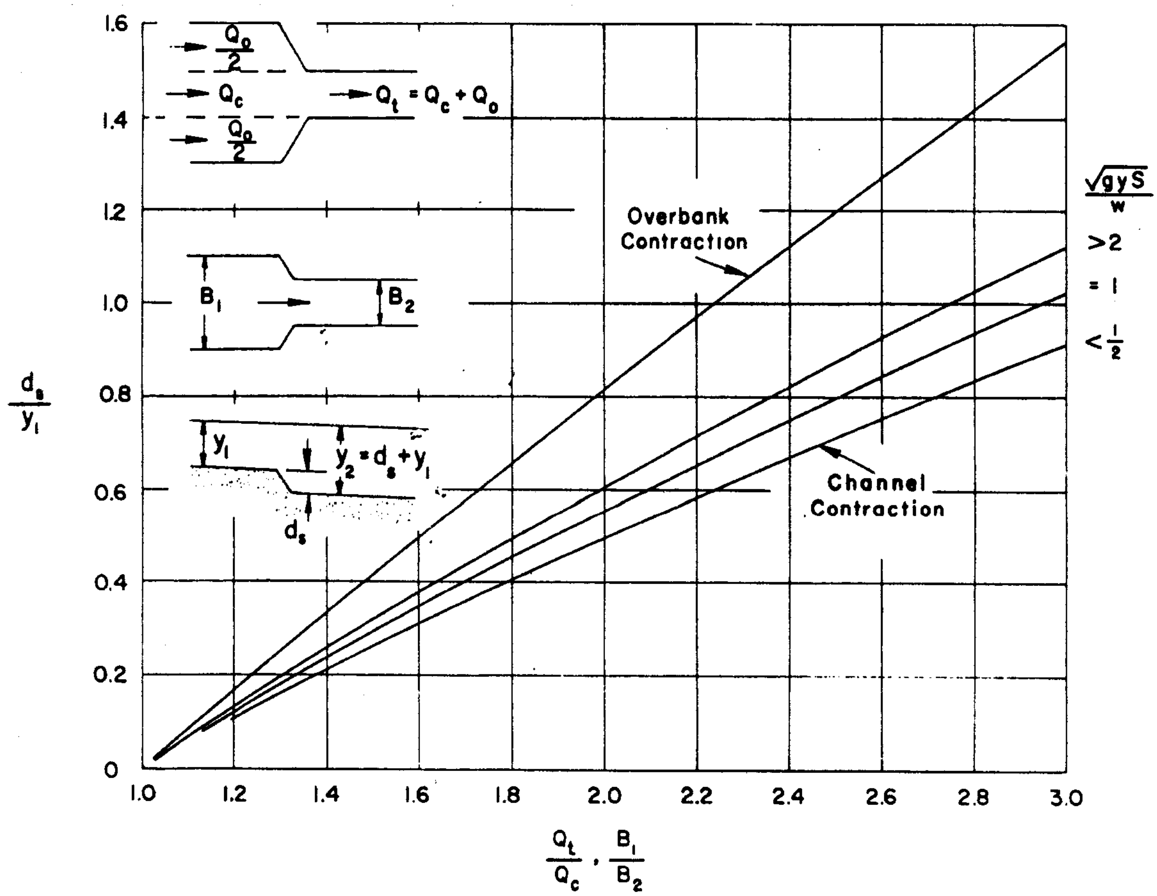
The exponent K varies as follows:

K	$\sqrt{\frac{gys}{FV}}$
0.59	< 0.5 (channel contraction)
0.64	1.0
0.69	> 2.0

ds = scour depth below streambed
Y1 = upstream channel depth
QT = total flow
Qc = channel flow
W = sediment fall velocity
B1 = upstream channel width
B2 = contracted channel width
V = velocity

7.55 The formation of an armoring layer of coarse sediment can influence or even control the depth of scour or degradation when ten percent or more of the bed material is large enough to resist movement.

The U.S. Bureau of Reclamation (U.S.B.R.) has an empirical approach to calculate the diameter of the particle required to armor the bed, and the depth to the armored layer.[85] The procedure is based upon the tractive force required to move the particle at the channel forming dominant discharge of the bankfull stage.



Depth of scour in a long contraction

Figure 15
from Laursen
Reference 34

$$\tau_c = \gamma d S$$

τ_c = tractive force
γ = unit weight of water
d = mean water depth
S = Stream gradient

Knowing the tractive force, the published curves or formulas of tractive force versus particle size can be used to determine the size of the stable particle. Specific values of critical tractive forces are contained in section 6.3 of this report.

A second method used by U.S.B.R. to estimate the size of the stable particle is based on the bed load transport formulas by Meyer-Peter and Schoklitsch. After rearranging their equations, the minimum transportable particle (or armoring size) can be calculated as shown below:

$$D_A = \frac{4762 \quad S^{1.33} Q}{B}$$

D_A = mean particle size, mm
S = slope
Q = dominant discharge, CFS
B = channel width

After determining the mean size of the particle that resists movement, the depth from the original streambed to the top of the armored layer can be readily calculated as shown below.

$$Y_D = Y_A \left(\frac{1}{p} - 1 \right)$$

Y_D = depth to armor layer
Y_A = depth of armor layer, (three x D_A or 6 inches whichever is smaller)
p = percent of bed material larger than D_A

This analysis thus provides information on the required size of the particle needed for the armored layer, and the depth to which degradation can occur.

7.56 The U.S. Army Corps of Engineers has developed a computer program identified as HEC-6 to predict maximum scour depths in rivers above

armored layers. As with the Bureau of Reclamation procedure, it computes the depth and particle size based upon a bed load transport formula. In this case, the bed load formula is one developed by Hans Eienstein, which is, in turn, combined with the Manning equation and Strickler's friction data. The result is:

$$D = \left(\frac{q}{10.21\ d^{0.3}}\right)^{6/7}$$

d = grain diameter
q = unit discharge
D = depth

When a variety of grain sizes are present, the depth is increased until enough particles of size "d" are accumulated to coat the bed.

The program has successfully modeled the depth of the armored layer on the laboratory flumes, at Fort Randell Dam, and on the Boise River in Idaho.[83]

7.6 SCOUR AT BRIDGE CONSTRICTIONS

The scour holes or pools that frequently form under bridges are a direct result of the bridge abruptly constricting the natural flow paths of floodwaters. Common practice for small bridges is to center them over the main river channel, with embankments on either side of the bridge supporting the highway or railroad. The embankments tend to prevent the free flow of overbank flows, and thus the water is forced to go under the bridge. This is a convergence of flow similar to the condition that causes scour at channel bends and at constrictions in channel width.

The scouring process at bridges has been described by hydraulic engineers using the concept of energy.[34] The sequence is that the contraction of flow by bridges causes water to accelerate and gain velocity through the bridge opening. The only way this water can increase velocity is for an increase in energy to occur, and this results in higher upstream water stages. The high velocities result in an increase in the sediment transport capacity, and scour occurs at the channel bottom and along the abutments.

As the flows pass the bridge, flow velocities decrease and the coarsest sediment settles to the bottom as it is too heavy for

transport at the lower velocities. There is a tendency for bridges with scour holes under them to also have mid-channel sediment bars a short distance downstream. As the scour hole increases in depth, the gross waterway area with the bridge also increases. The larger area decreases velocity, reducing the sediment transport rate under the bridge until equilibrium exists.

7.61 The use of the maximum threshold velocity is the simplest way to estimate scour depths at bridges, assuming that flow conditions are fairly uniform. Using this method, the cross-sectional area and depth of scour for a given bridge width are assumed to occur when the mean flow velocity under the bridge is equal to the maximum permissible velocity of the exposed strata of the bed material. This may involve a trial and error solution when the permissible velocity is tabulated as a function of depth. The procedure for bridges is similar to that described in section 7.52 for open channel scour.

The Canadian Roads and Transportation Association recommends that the cross-sectional area of flow with scour be redistributed to find the maximum possible scour.[53] This generally means that the area of flow be transformed from a "rectangular" shape to a triangular shape with the same base width. The triangular area depth will generally be 1.5 to 2 times deeper than the rectangle, depending on the side slopes of the so-called "rectangular" section. In previous discussions on this transformation of the scour hole shape, Blench mentions that Inglis found that scour in Indian Rivers was twice the depth of the regime condition.[6]

In nonalluvial rivers, only the area below the normal bed needs to be transformed in shape when using this method.

7.62 A common method for estimating bridge scour in the British Commonwealth nations is to use one of the several equations for the regime depth. The Inglis-Lacey bridge scour relation is presented below:

$$D_S = (0.946)2 \left(\frac{Q}{1.76 \sqrt{d_{50}}} \right)^{0.33}$$

Where: D_S = depth of scour from water surface
Q = flow rate, CFS
d_{50} = mean bed material size (mm)

The data base was from silt and fine sand rivers in India. It assumes that the river is at equilibrium, and does not contain any terms relative to the bridge size or degree of flow contraction. Note that the term in the denominator is the Lacey silt factor.

This expression is the regime normal depth of flow, times a factor of two.

7.63 Laursen has published several technical papers on scour at bridges, based on extensive model studies.[34, 35, 36] He modified his equations for scour in long contractions for applications at bridges. The Laursen model studies found that the deepest scour was adjacent to the abutments, and he focused his attention on that area of the waterway.

For bridges that span the channel but obstruct overbank flow, the Laursen equation on the floodplain is:[34]

$$\frac{Qo}{Qw} \frac{W}{Yo} = 2.75 \frac{ds}{Yo} \left[\left(\frac{1}{r} \frac{ds}{Yo} + 1 \right)^{7/6} - 1 \right]$$

Where:

Qo = floodplain flow
Qw = flow over width W, which is contracted
W = width of scour hole next to abutment (2.75 ds)
ds = depth of scour
Yo = upstream depth of flow
r = 4.1 (typical)

This is solved by trial with use of the formula or design figures, by assuming a value of W, and solving the equation to see if W = 2.75 ds.

Similar expressions and design aids (see Figures 16 and 17) were obtained for conditions where there was a constriction of the main channel by bridge abutments.

The depth of scour obtained using the Laursen equations is independent of velocity and bed material. The reason given is that for alluvial rivers at equilibrium, the upstream velocity and the velocity at the bridge are related to the depth. Knowing this, the upstream depth is thus an easy to measure indicator of velocity and erodibility as changes in velocity or bed material are reflected in the upstream depth of flow.[92]

Bradley, in a technical discussion of Laursen's work, said that the model studies were successfully duplicated at Colorado State University, and by river data from India.[7] However, Bradley mentioned that the model studies present a case of steady flow on fairly uniform material. The model-based scour formulas were said to overestimate scour, as bed material in natural rivers often has erosion resistant strata in the deeper alluvium, and do not have fully movable beds and sustained flow rates. The U.S. Geological Survey recommended against use of the Laursen method due to the lack of confirming data as of 1967.[12]

7.64 Model studies and dimensional analysis by Liu provide data on scour depths at bridge embankments, and are partially confirmed by measurements along the Mississippi River.

Liu proposed that the scour below the mean streambed is:

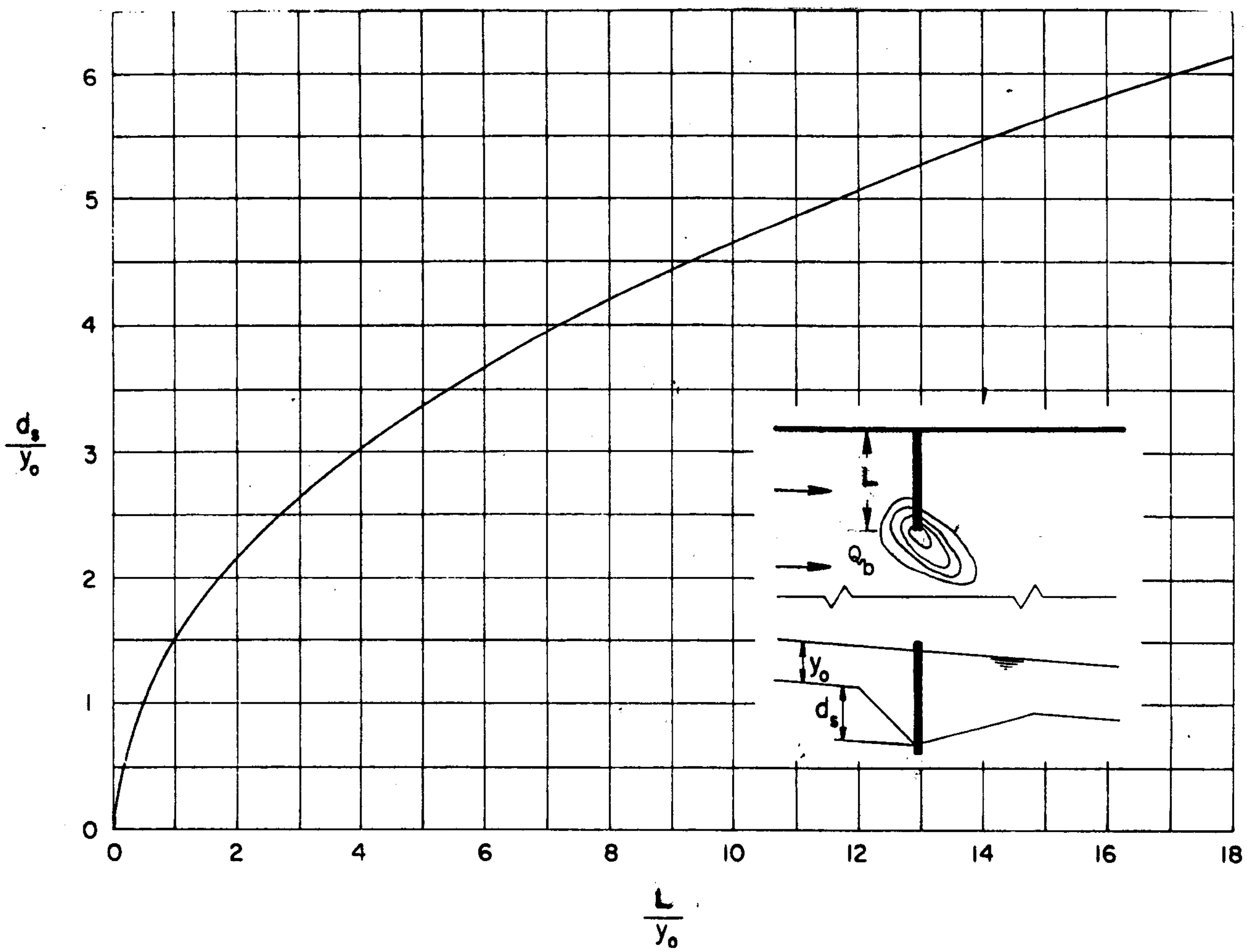
$$d_{sc} = (Y1) \quad (K) \left(\frac{L}{Y1}\right)^{0.4} \quad (F1)^{0.33}$$

Where: d_{sc} = scour below streambed

Y1 = upstream depth

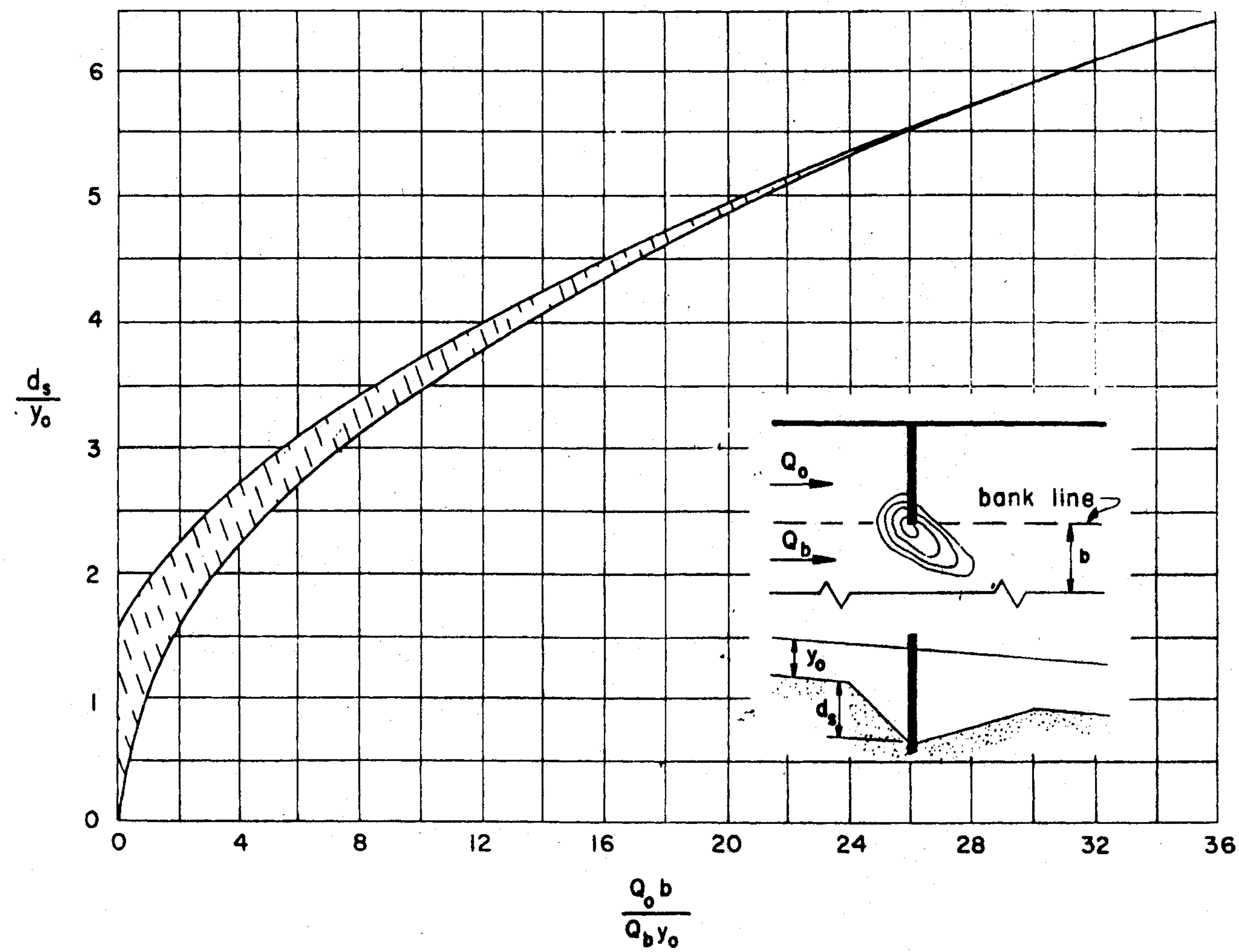
L = embankment length on floodplain

F1 = Froude number $\frac{V1}{\sqrt{gY1}}$



Basic Laursen design curve for bridge abutments extending into the main flow channel.

Figure 16
from Laursen
Ref. 34



Basic Laursen design curve for bridges constricting overbank flow on the floodplain (Q_O). Use the lower part of the shaded curve when overbank flow is minor, and where there is little cross flow. The upper part of the shaded area is used when there is significant overbank flow.

Figure 17
from Laursen
Ref. 34

The coefficient "K" is 2.15 for a vertical wall or abutment at the end of the embankment, or 1.1 for a sloped end on the embankment. Liu recommended that the value of dsc be increased by 30 percent for a conservative maximum depth.

On the Mississippi River, the field data indicates that a slightly revised version of Liu's equation can be used at embankments.

$$d_{sc} = 4\ (Y1)\quad (F1)^{0.33}$$

Simon suggests using the Mississippi equation when the embankment length is greater than 25 times the upstream depth.[76]

7.65 The scour records of 36 railroad bridges in New Zealand were analyzed by P.S. Holmes who then presented an equation for the general scour depth below the mean streambed.

$$d_{sc} = 3.15\ (Fr)\ (K)\ (\Delta Y)$$

Where: d_{sc} = scour below streambed

$$Fr = \frac{Vo}{(GD)^{0.5}} \qquad \text{Froude Number}$$

$$K = \left[\frac{B}{4.83\ (Q)^{0.5}}\right]^{0.5}$$

ΔY = incremental rise in water surface profile from normal stage to flood stage

B = waterway width

Vo = approach velocity

Holmes stated that the approach velocity should be increased by 20 percent to allow for local flow irregularities. In addition, Holmes found that the Shen equation (Section 7.72) for scour at bridge piers fitted his data for piers, and should be added to the general scour depth to predict the total scour.[62]

7.7 SCOUR AT PIERS

There are several formulas available for predicting the depth of scour for the specialized case where a bridge pier is located in the channel. They include the use of regime data, model studies, and empirical field data.

As previously indicated, the U.S. Geological Survey does not recommend use of regime type equations for estimating scour at bridges.

7.71 The first two formulas presented are both based upon relating the pier scour to the normal regime depth of a river. They should only be used where a river is already stable or "graded." Both are based upon model tests with clear water flow conditions, without a general movement of the bed material.

a. Inglis-Poona Equation, 1938

$$D_S = 1.70\, b\, (q^{0.67}/b)^{0.78}$$

Where: D_S = depth of scour below water surface

q = discharge per unit width of bridge

b = pier width, normal to flow

b. Blench Equation, 1966

$$D_S = 1.8\, Y \left(\frac{b}{Y}\right)^{0.25}$$

Where $Y = \sqrt[3]{\frac{q^2}{F}}$, $F = 1.9\, (d_{50})^{0.5}$mm

The term "Y" is the regime flow depth for the given flow rate and bed material.

The Blench equation has been reported as correctly predicting scour on 17 rivers in India.[5] It is noted that it gives a scour depth of roughly twice the depth of equilibrium, as defined by Lacey. Both of the above relationships are from model studies, and once again are only considered valid for regime rivers. In a 1967 report, by members of the U.S. Geological Survey, recommendation was against the use of regime equations for estimating scour conditions at bridges.[12]

7.72 Model studies at Colorado State University by Shen led to a pair of pier scour formulas for both clear water and sediment transport conditions.

Shen Equations (1966)

No. 1 $$d_{sc} = 0.00073 \left(\frac{Vb}{u}\right)^{0.619}$$

No. 2A $$d_{sc} = 11.0b\, F^2 \qquad F \leq 0.4$$

No. 2B $$d_{sc} = 3.4b\, F^{0.67} \qquad F > 0.4$$

Where: V = mean velocity
b = pier width
u = kinematic viscosity
d_{sc} = depth of scour below mean bed level

$$F = \frac{V}{\sqrt{gB}}$$

Equation No. 1 is used for clear water conditions with little sediment transport. It relates the scour depth to the Reynold's Number. Equation No. 2 is for rivers carrying sediment, and relates scour depths to the Froude Number.

The above Shen equations are intended to define the point beyond which sediment movement begins. The data base was from bed material with a mean diameter from 0.16 to 0.68 mm, with most mean diameters between 0.20 and 0.50 mm. The material tested was rather limited in size range, and this writer believes the equations should be limited to use on medium sand beds similar to those tested.

Raudkivi states that the Shen equation fits the data well for

$d < 0.5$ mm, but overpredicts scour for the coarse sands.[62] The usefulness of the Shen (No. 1) equation is also supported by field data recorded at 38 bridges by Holmes, and at 4 bridges reported by Melville in New Zealand.

7.73 Larras developed a method of predicting scour at piers based upon field measurements of scour after floods in France and data from model studies.

$$d_{sc} = 1.42\ Kb^{0.75}$$

Where: d_{sc} = maximum depth of scour below mean bed level

K = 1.0 for round piers, 1.4 for rectangular piers

b = pier width normal to flow

The author questions the validity and usefulness of this empirical equation as it does not have any terms representing velocity, discharge, or sediment size. The Larras equation cannot be used as a predictive tool as his data did not identify the factors of velocity, discharge, and sediment size.

7.74 An equation for pier scour depths below the streambed based on model tests was reported by Neill and supported by Blench. It represents the "deepest unrestrained scour that can occur at any velocity for a given approach depth."[26]

$$d_{sc} = KW\left(\frac{d}{W}\right)^{0.3}$$

Where: d = depth of local approach flow

W = pier width at 90^{o} to flow

d_{sc} = depth of scour below the riverbed

K = 1.5 for rectangular piers

K = 1.2 for circular pier nose

The value of "K" should remain at 1.5 for all piers located at an oblique angle to the flow. There is no term representing regime conditions or sediment size.

7.75 The U.S. Geological Survey, U.S. Department of Transportation, and many State highway departments recommend the Laursen method for estimating scour at piers. The Laursen (II) equation of use in solving the depth of scour at piers is a complex phrase difficult to solve directly. As a result, it is frequently used in the form of a "design curve", which has been widely published in references 24, 12, and 36 (see Figure 18). The basic value for scour at piers, as determined by Laursen's equation or design curve, is multiplied by separate empirical coefficients to account for the shape of the pier's upstream face and for the angle of the pier with respect to the direction of flow. Semi-circular pier noses were found to have 90% of the scour at the basic rectangular pier, while elliptical and lenticular pier shapes have 70% to 80% of the basic scour depth. The effect of the flow angle can be quite significant for sharp angles and for long, narrow piers, greatly increasing scour depth.

Laursen cautions that his design criteria should only be used for steady subcritical flow with an active alluvial bed. For streams subject to flash floods, he recommends increasing the projected scour depths by 50 percent. Laursen also mentions that his procedure should not be used for very fine bed materials, where particles may be transported primarily by suspension.[36]

The Laursen tests were conducted in a flume with fairly uniform fine sand and coarse sand.

Although recommended for use by the U.S.G.S., they point out that many engineers are not in agreement with the Laursen method, in part due to the lack of confirming data from actual rivers.[12]

Raudkivi reports that a simpler form of the Laursen II equation was developed by Callander with little loss of accuracy, as shown below:[62]

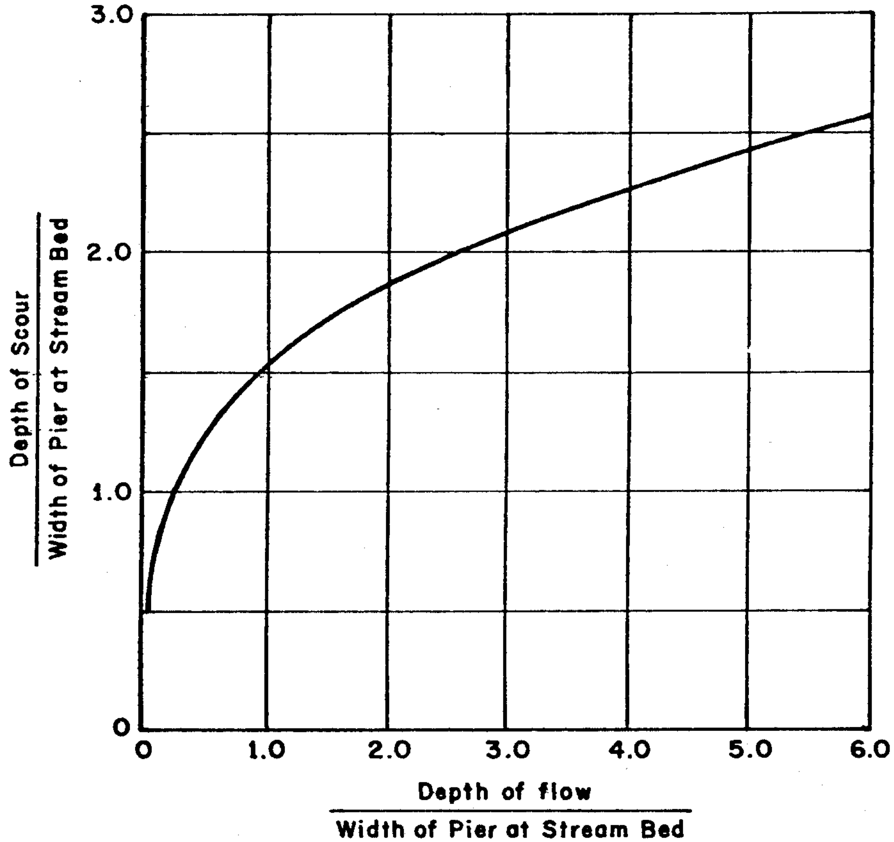


Figure 18
from Laursen
Ref. 36

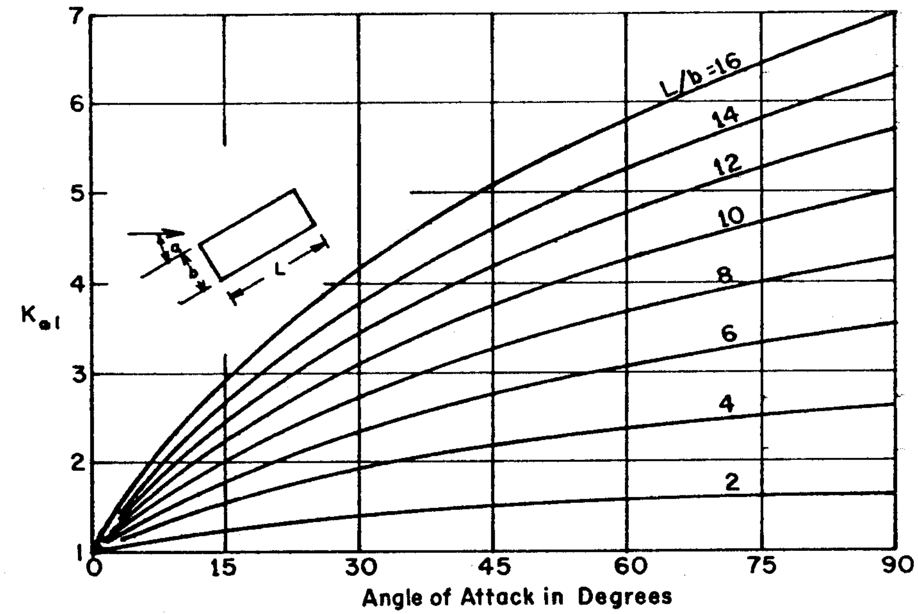


Fig. 6.05. Multiplying Factor for Angle of Attack

Shape coefficients K_s for nose forms

(To be used only for piers aligned with flow)

NOSE FORM	LENGTH-WIDTH RATIO	K_s
Rectangular		1.00
Semicircular		0.90
Elliptic	2:1	0.80
	3:1	0.75
Lenticular	2:1	0.80
	3:1	0.70

$$d_{sc} = 1.11b \left(\frac{Yo}{b}\right)^{0,5}$$

b = pier width

Yo = normal depth

d_{sc} = scour below riverbed

Note that this simplified version is similar to the equation by Neill in Section 7.74 and is for equilibrium scour conditions where the grain size of the bed material is not significant. The basic depth of scour is then multiplied for factors for the shape of the pier and the angle of the flow to the pier.

7.76 A committee on Bridge Hydraulics, of the Roads and Transportation Association of Canada has recommended guidelines for bridge design criteria and estimating scour at piers.[53] It is said to be based upon model studies and its simplified approach is to make it easy to use.

Using this Canadian guideline method, the depth of scour below the riverbed for design purposes is expressed as a coefficient times the pier width, for various shapes.

Pier Shape	Depth of Scour Below Bed	
Circular	1.5 W	W = pier width
Rectangular	2.0 W	
Boat shape, pointed ends	1.2 W	
Rectangular, rounded ends	1.5 W	

The above values should be increased by 50 percent when the depth of flow exceeds five times the pier width (W). The above values also have to be multiplied by another coefficient when the flow approaches the pier at an angle to the pier's axis.

Skew Angle	Scour Coefficients for Various Length-Width Ratios			
	4	8	12	L-W
0°	1.0	1.0	1.0	
15°	1.5	2.0	2.5	
30°	2.0	2.5	3.5	
45°	2.5	3.5	4.5	

Note that the skew coefficient for long narrow piers is much higher than for the short piers.

DISCUSSION OF SCOUR PREDICTION METHODS

The above review of scour conditions indicates that a widely accepted or universal methodology for predicting scour depths does not exist. The available methodologies are based upon a mixture of empirical, theoretical and experimental research.

There are conflicting reports on the effect of the bed material size on scour; the influence of the normal, uniform flow depth; and whether there is a maximum depth of scour. There is thus a need for further research on this subject, and at the same time engineers should proceed to cautiously utilize scour prediction methods as part of their channel and bridge design work.

7.8 STONE RIPRAP

The analysis and design of channel linings is an important element in the long-term stability of channels in erosive material. Of the several materials available for linings, stone is one of the most common and advantageous. Lining the channelbed or banks with a layer of stone, called riprap, can prevent erosion of the underlying material. The advantages of stone riprap channel linings include:

a. Flexibility and ability to withstand settlement or frost heaves
b. Easily repaired
c. Locally available
d. Allows vegetation to grow
e. Low cost
f. Natural appearance
g. Effective protection from scour.

In order to fully protect the underlying soil from erosion, the stone riprap must be able to remain in place and prevent movement of underlying soil through it. Among the factors to consider in

riprap stability are the size of the stone, gradation, surface slope under the riprap foundation or bedding, flow velocity, flow turbulence, weight of the rock and shape of the rock.

Most of the above factors have been discussed in other publications and will not be repeated here. However, this report will discuss the methods commonly used to determine the required size of the stone riprap (see Figure 19). This is being done because there is a need for greater awareness of the various methods of determining riprap size, and when they are applicable. In addition, the discussion will present recent information on a theoretical analysis of the factor of safety against failures of the riprap. Factors of safety above 1.0 indicate a stable condition.

The reader should be aware of the importance of the surface slope upon which the riprap is placed. The stability of riprap on sloped surfaces is subject to gravitation forces as well as those due to the tractive force of the flowing water, and thus requires special concern. Recent studies by the Federal Highway Administration indicate that riprap on slopes steeper than 3 parts horizontal to 1 part vertical should be studied for the effect of slope. In addition, the riprap must be on a surface that will not erode from under the stones. Common practice is to use graded filters of sand and gravel, or filter cloth. The fact that the various methods to size riprap give different results means that we do not yet know everything we need to about riprap stability, so special attention must be used in design to use the method most applicable to the situation.

7.81 The current U.S. Department of Transportation method of designing riprap (Hydraulic Engineering Circular No. 15) was recently developed based on research work at the University of Minnesota.[88] Empirical data on the critical boundary shear conditions at the point of incipient motion of the riprap was set equal to the theoretical maximum shear stress, and solved for the allowable depth of flow for a given stone size. This can be rearranged to solve for the stone

Figure 19

SUMMARY OF RIPRAP DESIGN METHODS

REPORT SECTION	METHOD	SURFACE SLOPE	ORIGEN	THEORETICAL SAFETY FACTOR
7.81	DOT HEC- 15	Variable	shear stress theory	Unknown
7.82	DOT HEC-11 and HDS-4	Variable	empirical	Less than 1.0
7.83	Bureau of Reclamation	Flat	empirical	1.09
7.84	Raudkivi	Unknown	theory	Unknown
7.85	ASCE - Isbash	Variable		1.2 plus
7.86	Corps of Engineers - Isbash	Flat		1.0 plus
7.87	Corps of Engineers - Culverts	Flat	empirical	
7.88	Corps of Engineers - Channels	Variable	shear stress theory	

sizes as follows:

$$D_{50} = \frac{d \gamma So}{5}$$

Where: D_{50} **= mean stone diameter**

d = depth of flow (maximum)

γ **= density of fluid**

So = slope of channelbed

Several coefficients are provided for adopting the above to flow around bends or on steep slopes on the sides of the channel.

The above formula is for uniform flow, and may not be valid for non-uniform turbulent flow. Another possible restriction is that most of the values for the critical boundary shear are based upon small stone sizes.

7.82 The U.S. Department of Transportation publication "Design of Roadside Drainage Chanels" HDS-4 (1965, 1973) contains two figures for determining riprap size (see Figure 20). The first curve is from the Corps of Engineers, and relates the average channel velocity to the velocity against the stone, for various size stones.

The second curve, from the American Society of Civil Engineers (1948), relates the velocity against the stone to the required stone weight and the mean equivalent stone diameter.

This method of riprap design was also published in the later publication by DOT, entitled "Use of Riprap for Bank Protection, Hydraulic Engineering Circular No. 11" (1967, 1970), and has recently been replaced by their later publication of HEC-15.[88]

They recommended at that time that the velocity against the stone be increased by a factor of 2.0 for turbulent flow, and at bends, and that the actual mean flow velocity be used to size riprap at culverts.

The theoretical analysis of riprap safety factors by Stevens,

Simons, and Lewis found that the above procedure formerly used by DOT gave a safety factor of less than 1.0.[78] This method is probably one of the most common in use today due to the wide circulation of it during the 1960's.

7.83 The U.S. Bureau of Reclamation uses an empirical "curve" to size riprap below stilling basins and in channels with flat beds (shown as curve #3 on Figure 20). The curve is a plot of the bottom velocity versus the stable stone diameter based upon model tests and field test data.[59]

The empirical curve was presented in the form of an equation by N. K. Berry of the University of Colorado

$$d = \frac{V^2}{6.75}$$

Where: d = stable stone diameter, inches

V = velocity, feet per second

This method was found to be satisfactory in a laboratory test of 2 1/2 inch stones, and in field tests with stones of 18-inch diameter. The Bureau of Reclamation suggests that for design projects, the average velocity be used as a conservative approach. The safety factor for this procedure on flat slopes, was found to be 1.09.[78]

7.84 The method discussed by Raudkivi is based upon the Strickler and Manning equations for channel resistance and flow. The upper limiting velocity at which stone movement would begin was defined as:[62]

$$V = 9.4\, d^{1/3}\ Yo^{1/6}$$

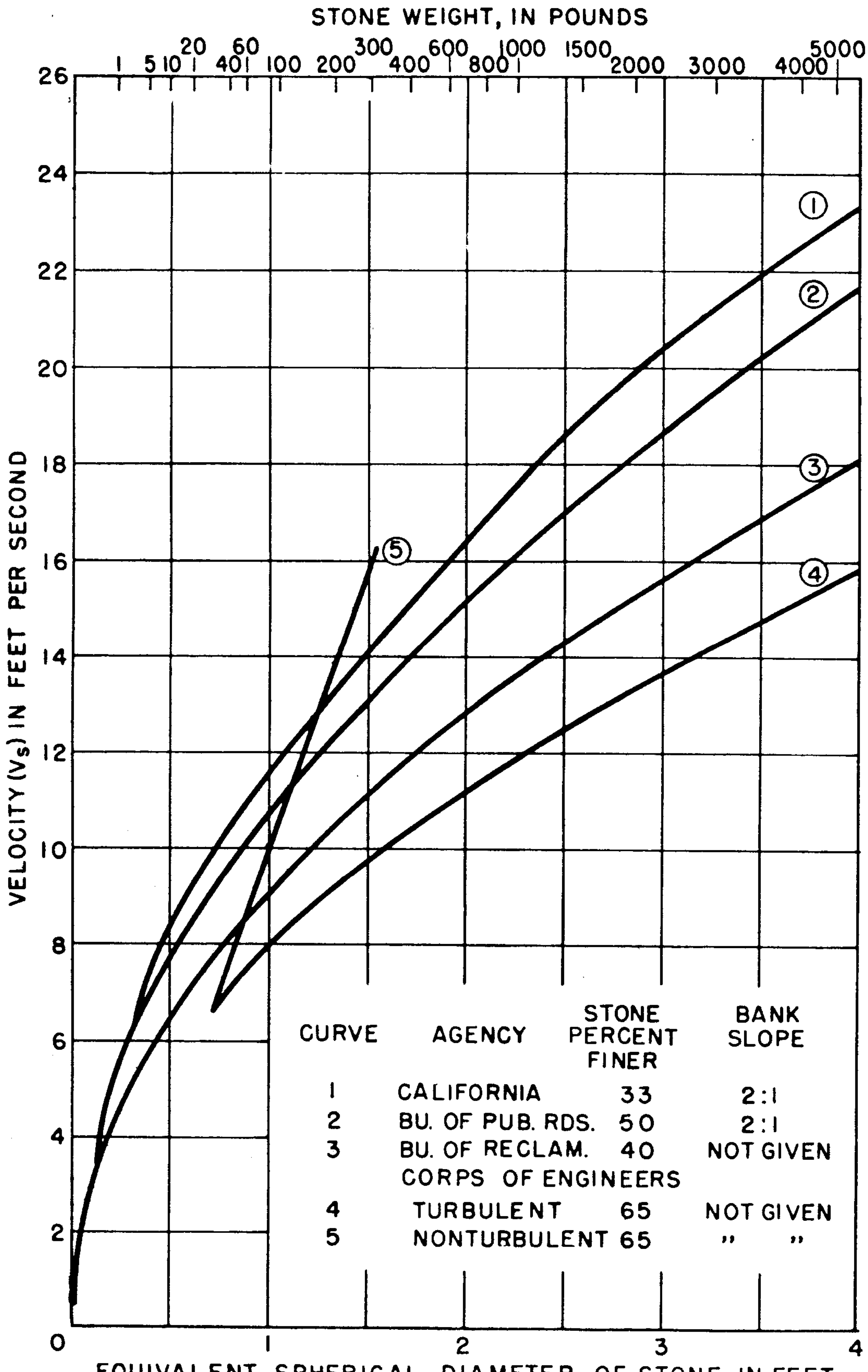
solving the equation for "d" yields:

$$d = \left(\frac{V}{9.4\ Yo^{1/6}}\right)^3$$

d = diameter in meters

Yo = depth of flow

V = velocity



-COMPARISON OF SIZE BY VARIOUS METHODS

Figure 20
from Bureau of
Public Roads
H.E.C. #11

7.85 The American Society of Civil Engineers Manual of Sedimentation Engineering suggests the use of the 1936 Isbash formula for determining the size (in weight) of riprap on sloped banks.[92]

$$W = \frac{4.1 \times 10^{-5}\ G_s\ V^6}{(G_s - 1)^3\ \cos^3\ \emptyset}$$

which can be revised to solve the diameter of a spherical stone as:

$$D = \left[\frac{(24.6 \times 10^{-5})\ (V^6)\ G_s}{\pi (G_s - 1)^3\ (\cos\ \emptyset)\ \gamma s}\right]^{1/3}$$

Where: G_s = specific gravity of stone
V = flow velocity, feet per second
$\emptyset$ = slope angle
W = weight of stone, pounds

This method is also the basis of the recommended riprap design curve used by the Delaware Water Resources Commission, and the California Division of Highways uses a modified version of it. A simplified form of the equation for stone with a specific gravity of 2.65 on a mild slope is:

$$W = 2.44 \times 10^{-5}\ V^6$$

W=weight of rock, pounds
V=velocity against stone, FPS
(see Fig. 21)

The study by Stevens, Simons, and Lewis found that this method by Isbash has a safety factor greater than 1.0, and some 20 percent higher than the Corps of Engineers and California Division of Highways' version of the Isbash equation.[78]

7.86 The U.S. Army Corps of Engineers uses an equation by Isbash for determining riprap sizes for use in flowing and turbulent water for application at stilling basins, or where the rock is deposited in

flowing water. Rearranged to solve the stone diameter, it is:[84]

$$D_{50} = \frac{V^2}{C^2 \quad 2G \left(\frac{\gamma s - \gamma w}{\gamma w}\right)}$$

Where: D = stone diameter, feet

V = mean flow velocity, feet per second

g = gravity acceleration

γs = density of stone, lb./ft.^{3}

γw = density of water, lb./ft.^{3}

C = 0.86 for turbulent flow

C = 1.20 for mild flow

The equivalent weight (w) of the stone can be found from the mean diameter as:

$$D = \left(\frac{6W}{\pi \ \gamma s}\right)^{1/3}$$

Stevens, Simons, and Lewis found in their theoretical analysis that for flat beds, the above Isbash formula gave a safety factor greater than 1.0 whenever the flow depth was over twice the riprap stone diameter.[78]

7.87 The Corps of Engineers has riprap size criteria specifically for energy dissipators at storm drain and culvert outlets, flowing full or part full.[84]

$$D_{50} = C \left(\frac{B^2}{TW}\right) \left(\frac{Q}{B^{2.5}}\right)^{1.33}$$

Where: D_{50} = mean stone diameter, feet

B = culvert width or pipe diameter, feet

Q = discharge rate, CFS

TW = tailwater depth above invert

C = 0.0200 horizontal riprap
0.0125 preformed scour hole with a depth of 1/2 B
0.0082 preformed scour hole with a depth of B

7.88 For open channels designed for flood control, the U.S. Army Corps of Engineers recommends a riprap design procedure based on the shear stress in the channel.[87]

The maximum theoretical shear on the bed of the channel is:

$$\tau_o = \gamma RS$$

This is rewritten as:

$$\tau_o = \frac{\gamma \, \overline{V}^2}{\left(32.6 \log_{10} \frac{12.2Y}{D_{50}}\right)^2}$$

γ = Unit weight of water

R = Hydraulic radius, feet

S = Energy gradient

D_{50} = Mean stone size, feet

Y = Depth of flow

$\overline{V}$ = Average velocity in a channel segment

τ_o = Shear in a channel segment

The shear stress on the channel bed at bends is found by multiplying the basic bed shear determined above by a bend coefficient. The shear at the bend is:

$$\tau_b = \frac{2.65 \; \tau_o}{\sqrt{\frac{r}{w}}}$$

r = radius of bend

w = water surface width

The allowable shear that can be withstood by the riprap on a flat bed is described as:

$$\tau = 0.04 \; (\gamma_s - \gamma) \; D_{50}$$

γ_s = Unit weight of stone

The riprap size is then:

$$D_{50} = \frac{\tau}{0.04 \; (\gamma_s - \gamma)}$$

The allowable shear for riprap on the channel slopes is:

$$\tau_s = \frac{(1 - \sin^2 \; \emptyset)^{1/2}}{\sin^2 \; \theta}$$

$\emptyset$ = Angle of side slope

θ = Angle of repose for riprap

In evaluating the above procedure, the author notes that the allowable shear appears to be based upon Shields' data for flow on a rough boundary (see section 6.33). The term in parenthesis for the allowable shear on slopes is as derived by Lane in 1953, and used by the Bureau of Reclamation. Using stone weight of 165 lbs./Ft.^{3}, the equation for allowable shear on flat slopes reduces to:

$$\tau = 4.1\ D_{50} \quad \text{(Ft. Units)}$$

This is similar to findings by Anderson and the Federal Highway Administration (F.H.A.) H.E.C. - 15. (Anderson had a coefficient of 4.0, F.H.A. uses 5.0)

In using any of the formulas based on the shear stress theory, it must be remembered that the actual shear stress is generally less than the theoretical value due to non-uniform flow and lateral turbulence. The shear stress on the bed decreases near the channel bank, and for narrow channels, never reaches the theoretical maximum even in the center.

The maximum shear on the sides never exceeds 75 percent of the theoretical maximum on the bed, and can be less on the narrow channels. However, in bends, the shear can be 2.5 times higher than the theoretical value.[76]

7.89 The required thickness of riprap linings on channels has been subject to considerable debate, but there is general agreement relative to the stone size, as noted below:

Source	Recommended Riprap Thickness
U.S. Department of Transportation	D_{100} (maximum stone diameter)
Bureau of Reclamation	1.5 times D_{100}
Corps of Engineers	Length of maximum stone size, or 2 times D_{50}
Soil Conservation Service	1.5 times D_{50} (with filter) 3.0 times D_{50} (without filter)

Stone riprap must be proportioned and graded so as to prevent the migration of soil particles from beneath the riprap. When this is not possible, the riprap must be underlain by a filter material that will hold the soil in place. The filters may be either the "graded soil filters" patterned after those of Karl Terzaghi, or be a permeable fabric material such as woven plastic.

8.0 MAN'S INFLUENCE ON RIVER MORPHOLOGY

Man's activities have greatly changed the morphology and ecology of many rivers throughout the country. The urbanization process in particular is sufficient to induce long-term changes in rivers by increasing peak flow rates, increasing sediment loads, and encroaching on floodplains.

8.1 URBANIZATION

Urbanization and land development is accompanied by increasing the area of the watershed covered with impervious surfaces. Larger impervious areas increase the amount of surface runoff, and decrease infiltration. In addition, urbanization and the presence of storm sewers increases the velocity of surface runoff and allows runoff to rapidly concentrate in rivers.

The combination of larger runoff amounts and faster concentration times can drastically change the flow rates in rivers. For instance, Leopold found that a typical area with twenty percent impervious surfaces and fifty percent served by storm sewers will have 2.5 times more runoff than natural watersheds.[38] Urbanization will also increase the frequency of bankfull flows in the river. With a watershed that is twenty percent impervious and twenty percent storm sewered, the frequency of bankfull flows will double from seven times in ten years to fourteen times in ten years.[38]

In addition to increasing peak flows, urbanization reduces the infiltration of water into the soil and the storage of surface water. This leads to a long-term reduction in the river's base flow and mean annual flow.

As a result of higher peak flow rates and the greater frequency at which they occur, rivers will enlarge their waterway area by vertical and/or horizontal degradation. However, the lower mean flow rate of urban streams can carry little sediment, and bottoms will thus accumulate unstable debris between storms. Unless that lateral erosion is contained, the river will generally increase in width faster

than in depth. This has been confirmed by studies of 32 rivers in Maryland before and after flooding. In those cases, rural rivers increased their width/depth ratio by 47 percent, while the ratio on urban rivers increased by 70 percent.[15] Even when detention basins are utilized to reduce peak runoff rates, the higher volumes of runoff flow over longer durations, thus increasing sediment transport and scour.

8.11 The removal of vegetation and disturbance of surface soils during construction allows greater soil erosion to occur in urbanizing watersheds. The sediment loads in rivers may increase up to as much as 10 to 100 times the natural sediment loads.

After construction has ceased and the ground stabilized, sediment levels may decrease below those found in nature. This is because the widespread use of thick lawns and paved surfaces will decrease sediment sources. The high sediment loads during urbanization degrade water quality, that obstruct low flow levels.

Rivers respond to urbanization by eroding during the frequent floods and subsequently filling with sediment in between floods. This cycle leads to unnatural variations in the shape and size of rivers and creates a great deal of stress on aquatic life. The rate of erosion during floods is not necessarily equal to the rate of sediment deposition between floods, and the river may degrade or aggrade.

8.12 Wolman has presented a typical sequence of river morphology changes due to human influence for the southeastern United States, and it has been confirmed by similar studies elsewhere.

He states that many rivers had relatively stable channels during the forest conditions that prevailed up to 1830. After 1830, the widespread clearing of the forest and agricultural activities increased sediment loads and the rivers responded by a period of general aggradation. The sediment yield peaked between 1880 and 1910, and began a slow decline as farms reverted to grazing or woodlands.

The decline of agriculture and sediment loads allowed the rivers to seek a new equilibrium, and consequently degradation of the riverbeds began in the 1930's. Degradation continued until the post-war construction boom of the 1950's, when stream aggradation once again took place due to excess sediment loads.

Beginning in the mid 1960's, the rate of construction slowed as many suburban areas were fully developed. The sediment loads were reduced by the establishment of ground cover and, as a result, degradation occured. This present period of degradation is being accelerated by the higher peak flow rates produced by impervious surfaces.

8.13 The author has observed several cases where encroachment of the floodplain has compounded the effect of urbanization on river morphology. The typical sequence begins when land owners abutting a river fill the rear of the properties to gain usable dry land. After several years of the river eroding the fill, landowners construct retaining walls along the river bank to protect the fill from continual erosion. In response to the banks being fixed with a rigid boundary and the floodplain filled, the river will attempt to enlarge its channel for the higher flow rates by degrading its bed. The process of degradation will erode the streambed below the retaining walls, which then collapse. Not only do the retaining walls collapse, but the degradation may also undermine bridges over the river and utility lines under the river bed. This exact sequence has occurred along portions of the Rooster River in Bridgeport, Connecticut during the past 40 years, with an average measured degradation of 2 feet in 20 years. This degradation led to the partial destruction of one bridge in 1976, and the complete destruction of a second bridge in 1980. In another recent case reported by Engineering News Record (3/27/80), a six lane Highway bridge in Memphis collapsed after the river bed degraded 10 feet in 23 years.

8.2 CHANNELIZATION

Channelization is the process of widening, straightening, deepening, and otherwise altering natural rivers. The intent is usually to provide a channel with higher flow capacity to reduce flooding, and often to reclaim land. Other reasons given for channelization include drainage of wetlands and navigation improvement.

Despite the influence of man, a channelized section of the river will try to revert back to its natural regime condition. It will often destroy the man-made system, and it is therefore important to understand how the river reacts. The rest of this chapter will describe how rivers respond to man's altering them, with the next chapter providing recommendations on sound river management techniques and design considerations.

8.21 Lane has presented a qualitative relationship that contains four variables and can be used to predict the river's reaction to a change.[18]

$$BL \times d_{50} \approx Q \times S$$

Where:

BL = bed load

d_{50} = bed load mean size

Q = mean discharge

S = slope

If any variable changes, the others must also change to maintain the proportions of the equations. For instance, consider a case where the discharge increases,

$$Q \approx \frac{BL \times d_{50}}{S}$$

If the value of "Q" increases, then either the bed load increases, the sediment size increases, or the slope decreases. Note that if the first two items increase without a change in upstream sediment supply,

the river will supply its own bed load by degrading and eventually decreasing the slope until stability is achieved.

In a similar manner, if the bed load "BL" is considered:

$$BL \approx \frac{Q \times S}{d_{50}}$$

An increase in bed load must be accompanied by an increase in discharge or slope, to convey the extra sediment at equilibrium, or a reduction in the sediment size must occur.

A similar qualitative equation has been presented by Schumm, using different variables.[71] Using the general equations for width, depth, meander length, discharge, and slope, he proposed that:

$$Q \approx \frac{b, d, \lambda}{S}$$

and $$BL \approx \frac{b, s, \lambda}{d, P}$$

Where the additional terms are:

b = width
d = depth
λ = meander length
P = sinuosity

Schumm provided the following typical responses to changes in discharge or bedload:

$$Q^+ \approx b^+, d^+, \lambda^+, S^-$$

$$Q^- \approx b^-, d^-, \lambda^-, S^+$$

$$BL^+ \approx b^+, d^-, \lambda^+, S^+, P^-$$

$$BL^- \approx b^-, d^+, \lambda^-, S^-, P^+$$

The plus and minus sign indicate the direction of change in the variable.

The equations above will provide an indication of how rivers respond to change. Several specific situations are outlined hereinafter concerning river realignment, deepening, and the effect of dams.

8.22 The realignment of rivers into straighter alignment is often done to improve capacity, reclaim land, or to increase use for transportation. Channel straightening can cause severe disruption of the river morphology and ecology.

The straightening of a channel between any two points reduces the length but not the vertical drop, hence the channel slope is increased. This leads to higher flow velocities, and greater sediment transport. The channel will erode and degrade unless artificially stabilized or an additional sediment load is supplied.[46]

Changing the channel slope can induce a change in channel pattern. For instance, if the bed is coarse and resists erosion, the bed will erode laterally to supply the additional sediment. This, in turn, can lead to a braided channel that is wide, shallow, and unstable. In other cases, the straightened channel may be unstable as it attempts to establish a meandering pattern.

Degradation is common on steepened channels, and can progress upstream and into tributaries. The material eroded may form sediment deposits downstream of the straightened reach.

Straightening channels can affect the ecology of a river by eliminating the natural sequence of pools and riffles that normally provide a varied habitat. The elimination of fish feeding areas in meander pools, oxbows, and low velocity flow areas reduces the level of aquatic life. The unstable banks or beds that are subject to eroding are also poor habitat.[64]

The channel realignment can increase passage of waters to downstream reaches and alter the peak rate of runoff there. The upstream areas will drain quicker, and recharge of water into the soil decreases. This is most pronounced on porous soils, or where floodplain inundation becomes less frequent.[64]

8.23 Increasing the depth of a channel by dredging or excavation increases its cross-sectional area and flow capacity. A negative aspect of increasing depth is the lowering of the normal water level in the channel, and a resulting change in the groundwater table. This can alter subsurface water supplies, and dry up adjacent streamside wetlands that have ecological and flood retention values.

Increasing the depth of the channel, without changing the width, will increase the slope of the channel banks. This has to be done with caution to avoid slumping of the banks. In addition, the slope of tributaries will have to increase by downcutting to meet the new main channel grade. They may then progressively scour upstream until their entire length has degraded to a new equilibrium.

The greater depth of the channel will reduce the frequency of floodplain overbank flows. The Council on Environmental Quality stated that this alone caused ecological problems at 6 to 42 projects they studied.[44]

The degradation that takes place upstream of the deeper channel supplies sediment to the river. This sediment will be deposited downstream, causing aggradation as the river increases slope to transport the extra sediment load. The aggradation may lead to changes in the river pattern in downstream reaches.

8.24 The construction of dams to impound water along free flowing rivers often changes the river morphology. Aggradation has been found upstream from dams, and degradation occurs downstream.

Upstream aggradation usually results from any increase in the river's base level, including the situation when dams are constructed. The pool of water impounded by the dam increases water depths, decreases velocity, and allows sediments to settle. A delta will form at the point where river sediments are deposited at the head of the pool. As the delta grows, it decreases the slope of the river upstream and aggradation takes place. This may change the river pattern, and cause higher flood stages.

The water discharged from the dam is clear and free of sediment. The natural channel downstream of the dam will no longer have a

supply of sediment to balance the sediment transport capacity. As a result, the downstream river will scour beginning at the dam, and a general degradation takes place until the river slope and sediment transport capacity are reduced. The degradation may lead to steep unstable banks, a change in river pattern, and even the undermining of the dam.[42]

For example, the Mattabesset River in Kensington, Connecticut has degraded over 6 feet since 1897 downstream of the Kenmere Dam, despite the presence of 6 foot deep concrete erosion control sills across the channel. The combination of a reduced upstream sediment load, (due to the dam) plus a uniform soil that does not form an armored layer, has led to rapid degradation at a rate of almost one foot per decade. The degradation has already eroded the channel bed, destroyed the concrete erosion control sills plus the stilling basin apron at the toe of the spillway, and is now threatening the spillway itself.

An excellent example of the effect of a dam on upstream areas is at the Leesville Dam on the Salmon River in East Haddam, Connecticut. Several different dams have been located on the 100-square mile watershed since the late eighteenth century. The present dam is 20 feet high and 140 feet wide. The Salmon River itself is a steep, fast-flowing river with a sediment load that is high in bed load (sands and gravel).

The bed load is trapped in the dam's impoundment and by 1938 reached to an elevation near the crest of the spillway. The hurricane of that year washed much of the sediment out of the impoundment when portions of the dam failed, completely filling the downstream channel, requiring that it be dredged. Studies in the 1970's revealed that sediment has once again filled the impoundment, and that aggradation has extended upstream for several miles. During periods of low flow in the summer, the sediments are exposed and the river cuts a meandering channel through the sediment to the dam. The channel downstream has degraded to an armored bed of gravel, which provides excellent fish habitat. However, the aquatic life is periodically disturbed by high sediment discharges over the spillway during storms.

8.26 River morphology may be inadvertently altered by man as a side effect of excavation in the riverbed. The author is familiar with two different types of cases.

The first case is when changes in the river morphology take place due to the removal of sand and gravel. For instance, portions of the Naugatuck River between Shelton and Beacon Falls, Connecticut have been extensively mined for sand and gravel used for construction projects. The removal of this sediment material from the floodplain has allowed the width of the river to expand laterally, and as a result the river tends to be adopting a braided pattern in some areas.

A second example was observed by the author in Summit County, Colorado. Numerous rivers in that area were hydraulically dredged in the early part of the century to recover gold from the sediments of the river bottom. The dredging completely turned the sediments over, placing the coarse material from the older deposits on the top, and upsetting the balance between flow conditions and sediment transport. It prevents the river from scouring during floods, and the flood stages can therefore be expected to be higher.

River bed excavation for sand and gravel is the reported cause for a 15 foot deep scour hole and near failure of the I-10 bridge over the Salt River in Phoenix, The excavation had lowered the river bed downstream of the bridge by 20 feet. Engineering News Record (3/27/80) reported the excavation caused degradation around the piers plus a change in flow direction.

9.0 ECOLOGICAL CONSIDERATIONS

The aquatic life found in watercourses depends entirely on fluvial conditions for survival and is a major factor to evaluate while studying existing or proposed watercourses. The application of fluvial geomorphology is an effective way of reducing and compensating for ecological damages related to river projects.

9.1 FEDERAL POLICIIES

Beginning in 1979, federally funded water projects required uniform "Habitat Evaluation Procedures" to be utilized in assessing ecological impacts and damages for ecological losses, and the new regulations for the Fish and Wildlife Coordination Act requires Federal Agencies to assess impacts, to quantify benefits and costs, and encourages mitigation of damages. Projects are to be planned and designed to minimize impacts, avoid adverse impacts, and to try to offset losses.[93] The act requires federal agencies to consult with wildlife agencies prior to modifying any rivers, and to justify the adequacy of conservation measures. The goals of the federal policies are:

a, Reduce ecological losses--avoid unnecessary impacts by using the best available alternates to minimize damage.
b. Compensation - to offset unavoidable losses by creating new habitats of value to offset destroyed habitats.
c. Restoration and Enhancement - try to improve environmental quality by protecting and improving existing habitats.

9.2 IMPACTS

The riverine environment is subject to constant change and stress. While most aquatic species are able to adopt to change, the extreme range of possible habitat conditions can lead to shifts in the ecological system. The ecology of the watercourse is altered when the soil, microbiology, vegetation or hydraulic characteristics are

seriously disturbed.[17]

Much of the ecological damage occurring in the riverine system is due to urbanization and channelization.[1]

As pointed out by Leopold, the process of urbanization increases both the sediment loads and peak flow rates. The sediment increases turbidity, and this reduces photosynthesis and dissolved oxygen levels and impacts aquatic life.[44] The excess sediment is particularly harmful along watercourses with gravel beds that are normally high in aquatic life.[17]

The higher peak flow rates scour the bottom and banks removing vegetation, food sources, and shelter for aquatic life. Many species cannot tolerate excessive flow rates for long durations. Following high flow rates, the abnormally low flows in urban areas may not be able to transport the sediment generated by urbanization, and sediment deposition can bury organisms and prevent a stable bed from forming. The low flow rates may not be adequate to support former stream life.

The detrimental effects of channelization include:[2]

a. loss of shelter along the streambank and bed due to removal of snags and use of uniform, smooth cross sections
b. disruption of the nature pool/riffle pattern and diverse flow conditions due to changes in the flow rate and use of uniform bed slopes
c. increased flow velocities due to smoother channel perimeters and steeper slopes (when streams are straightened)
d. increased erosion and turbidity due to higher flow velocities
e. loss of vegetation along the banks and subsequent increases in the water temperature
f. reduced length of streams due to straightening the alignment, providing smaller aquatic habitats
g. loss of aesthetic value due to the removal of vegetation and alteration of natural features
h. loss of recreational value
i. reduction in aquatic food sources due to removal of vegetation,

removal of organic material from bed, and excess sediment

j. reduction in diversity of aquatic life due to reductions in the habitat diversity.

9.3 ECOLOGICAL VALUES

The stream bottom and banks are the most important elements of the aquatic habitat. It is desirable to have a well-sorted bed material to provide variations in grain size.[28] Stable material, such as cobbles, coarse gravel, and cohesive silts have been found more productive than moveable sands. Streams that have continuous movement of the bed have been found to be biologically inactive as the growth of emergent vegetation is prevented.[46] The following rates of food production have been determined:[90]

Bottom Material	Food Production, Grams/Ft.^{2}
Silt	3.07
Cobbles	2.47
Coarse Gravel	1.51
Fine Gravel	0.93
Sand	0.10

Variations in channel pattern and form help to encourage desirable diversity in aquatic life.[28, 81] For instance, trout require cold water with shaded gravel beds in riffles, but use deep, slow moving pools for shelter during hot weather and during high discharge periods. Other species prefer warm water with sand bottoms.

The highest quality riffles for ecological purposes are those with bed material of fine and coarse gravel, while moderate quality ecological conditions can be obtained when the bed material is of cobbles or bedrock. Water depths in the riffles of small streams should be at least 6 to 12 inches to support sizeable fish, with flow velocities of 2 to 3 feet per second.

Grizzell feels that moderate levels of channel scouring may be beneficial to the stream habitat as it removes the less productive fine grained sediments from well-sorted soils.[17] However, he points

out that excessive scour from floods can wash out vegetation and sources of shade along the banks. High flow velocities can also remove organisms from the bed and reduce that natural food for fish and water fowl.

Pools should have a length greater than the stream width and a depth of over 3 feet. Where abundant shelter is present, high quality pools in small streams can be as shallow as 2 feet.[81]

It is desirable to have sheltered areas along 50 percent or more of the river length. The shelter can be in the form of logs, stumps, boulders, overhanging vegetation, snags, and man-made flow deflectors.[81]

The removal of snags and sheltered areas reduces the trapping of organic material that serves as a food source, as well as removing quiet resting places. Studies have found that the removal of snags can reduce total trout populations as well as the points where many fish species concentrate.[47]

The vegetation along the channel banks is important as a source of shade to protect rivers from direct sunlight and solar heat. Full size trees are needed to shade rivers over 30 feet wide, and should be present along at least 40 percent of the river's length.[90] Excessive shade from high trees should be avoided, as it would discourage the growth of streamside grasses that stabilize the banks and emergent vegetation that serves as shelter and food sources.

On small streams, it is important to preserve some overhanging and streamside vegetation to provide a nutrient base for aquatic life. Snags are able to catch and trap particulate matter, and leaves, and provide substrate for microinvertebrates that shred, collect, graze on organics, and, in turn, serve as food for predators.[47] Small streams depend on terrestrial organic input, while larger rivers depend on their upstream tributaries for organic material for aquatic life.

9.4 MITIGATION

The detrimental effects of channelization can be reduced by designing the river improvements to be compatible with the morphology of natural streams and floodplains.

In-stream structures and other procedures can also be used to help improve the environmental characteristics of both conventional channelization projects and the simulated channel/floodway type of projects. The results have been considered positive and effective in reducing the impact of channelization.[2, 57, 49, 43]

Specific methods of supplementing the basic channel design to mitigate ecological damage include:

a. Deflectors - flow deflectors are constructed on the stream bed adjacent to the banks. They are constructed out of rock, timber, gabions or piles. Their purpose is to concentrate low flows by obstructing up to one half of the stream width in order to create higher local flow velocities that dig pools, with shelter on their downstream side. They can be quite effective, but need to be fairly low to avoid obstructing flood flows. They may be alternated with a spacing of five to seven stream widths to force the flow to meander. The deflectors should meet the bank at a 30 degree angle, and have a sturdy, solid fill between the front face and the bank to prevent bank erosion when the deflector is overtopped by high flows. Detailed instructions are available.[94]

b. Check Dams - small check dams, frequently only 1 to 3 feet high, can be installed across the steeper streams to form small pools of low velocity flow. Downstream of the check dam, scour holes form and provide a varied flow condition. Aeration of the water occurs at the overflow point. They may be constructed of rock, gabions, timber, or concrete, and should have a depressed center section to concentrate low flows. Check dams should not be used on flat streams where they may create excessively large pools that drown out the riffles.

c. Boulders - rocks of various sizes can be placed in the stream to provide shelter and irregular low flow patterns. They should have a height similar to the normal flow depth and be keyed into the streambed. They can be particularly effective in providing shelter in deep streams where deflectors could be damaged by flood waters.[49] Local scour can occur around them, and therefore they should not be placed next to the banks or with their axis perpendicular to the flow.

d. Pools - The profile of the channel should have well defined pools, 3 feet or deeper, in areas where they will tend to be self cleaning. The pools may be at river bends, or they may be created at scour holes below check dams or downstream of flow deflectors.

e. Riffles - areas of shallow, fast flows should form between the pools or at the narrow channel formed by deflectors. The goal is to create areas of fast flows over gravel beds for spawning areas in normally low velocity streams.[43]

f. Stream Velocities - Variable stream velocities should be provided to encourage a diverse habitat. In cold water areas, velocities of 2 feet per second are desirable for trout and other game fish.

g. Wetlands - It is desirable to preserve or even create streamside swamps and marshes alongside of new channels. This would provide food sources for many species and help make up for any reduced channel lengths.

h. Bed Material - In rivers that are not subject to deposition, a coarse bed material of gravel and cobbles can be provided if not already present. This type of bed has been found to provide the best conditions for cold water fisheries, and is least susceptible to movement.

i. Vehicle Crossings - should be confined to bridges or improved fords with a riprap base to minimize erosion and disturbance of streambank vegetation.

j. Vegetation - every effort should be made to preserve existing vegetation and to provide new vegetation along the channel banks. This provides cover and food sources for wildlife, and shades the water as well as supplying organic material. Preservation of existing vegetation may be done by working from the stream bed or by working on only one side of the river. Species of new vegetation should be carefully chosen in order to provide sources of food during all seasons.

k. Buffer Zone - a vegetated buffer is required on both sides of the channel to shelter the river from intense land use areas, provide wildlife cover, and trap eroded soils before they enter the river.

l. Spoil - excavated soils should not be used to form large fill areas on the flood plain. Some states, including Indiana, encourage limited, local spoil piles in order to have diverse terrestrial areas, but do not allow spoil piles to form continuous levees blocking local drainage.

m. Disturbed Soils - should be protected from erosion as soon as possible to reduce sediment sources in the stream which could bury valuable food sources and spawning beds.

n. Migratory Barriers - minimize or eliminate barriers (culverts, bridges, dams) that interfere with the movement of fish.

The above measures can be effective in restoring the aquatic habitat to productive levels. Studies conducted on the Weber River in Utah before and after stream modifications were made, found that in-stream structures such as check dams and deflectors changed the streams' morphology and caused pools and riffles to form. Biological surveys reported a rapid recolonization of the site with species similar to the original condition. Fish were concentrated in the scour holes.[2]

The Connecticut Department of Transportation recently completed a project along Roaring Brook, adjacent to I-86 in Willington, where an attempt was made to reestablish natural conditions, and similar projects have been undertaken in Virginia.

Plans for the Roaring Brook project initially called for construction of a straight, stone-lined channel. They were later revised as a requirement for an Inland Wetlands Permit to have a meandering channel. The final plans were implemented, with the addition of man-made pools and riffles to provide a varied habitat. Selected boulders were placed to allow local scour to occur, and to provide shelter for fish. A normal flow channel was provided, within a broader flood-prone area. The width varies from 6 to 30 feet.

Many of the features were based on recommendations from the "Connecticut Fly Fisherman's Association" and "Trout Unlimited." The project has been monitored, and is already serving as a recreational fishing area for trout.

Other studies have confirmed the effectiveness of drop structures to create biologically productive scour pools, and the use of gabion diversions did create high local velocities, and increased the percentage of the bed that was exposed gravel.[43, 49]

Local stream restoration projects have been undertaken to repair eroded channels damaged by floods. In New York State, projects of this type have been planned by the Soil Conservation Service and funded by Federal Manpower Training programs.[77] The projects have been successful in improving the stream with use of simple materials and unskilled labor (see Figure 21).

The instream rehabilitation structures are successful in creating varied flow conditions with micro habitats, but do not account for long term stability or changes in stream length.[2] Thus it is still very important to have proper design of the overall channel and flood plain system. They may be used in both local restoration projects and channelization projects, and for both completed project sites (retrofit), or as part of new projects.

Various time spans have been reported for the natural ecosystem to reestablish a stable community. The Soil Conservation Service has said that channel banks in Georgia were stabilized after two growing seasons,[44] while Keller reports that a complete biological recovery takes 50 to 100 years.[28] The time span will obviously depend on the size of the system, degree of artificial restoration, and local climate.

9.5 CHANNEL RESTORATION

The restoration of existing streams is an activity to revive a channel's capacity, aesthetics, and environmental values without extensive new work. It generally applies to streams in developed areas that have already declined in stability or capacity, and is often accompanied by management of the floodplain so as to eliminate the need for complete channelization.

The procedure has been described by Keller as follows:[28, 29]

a. Delineate the floodplain.

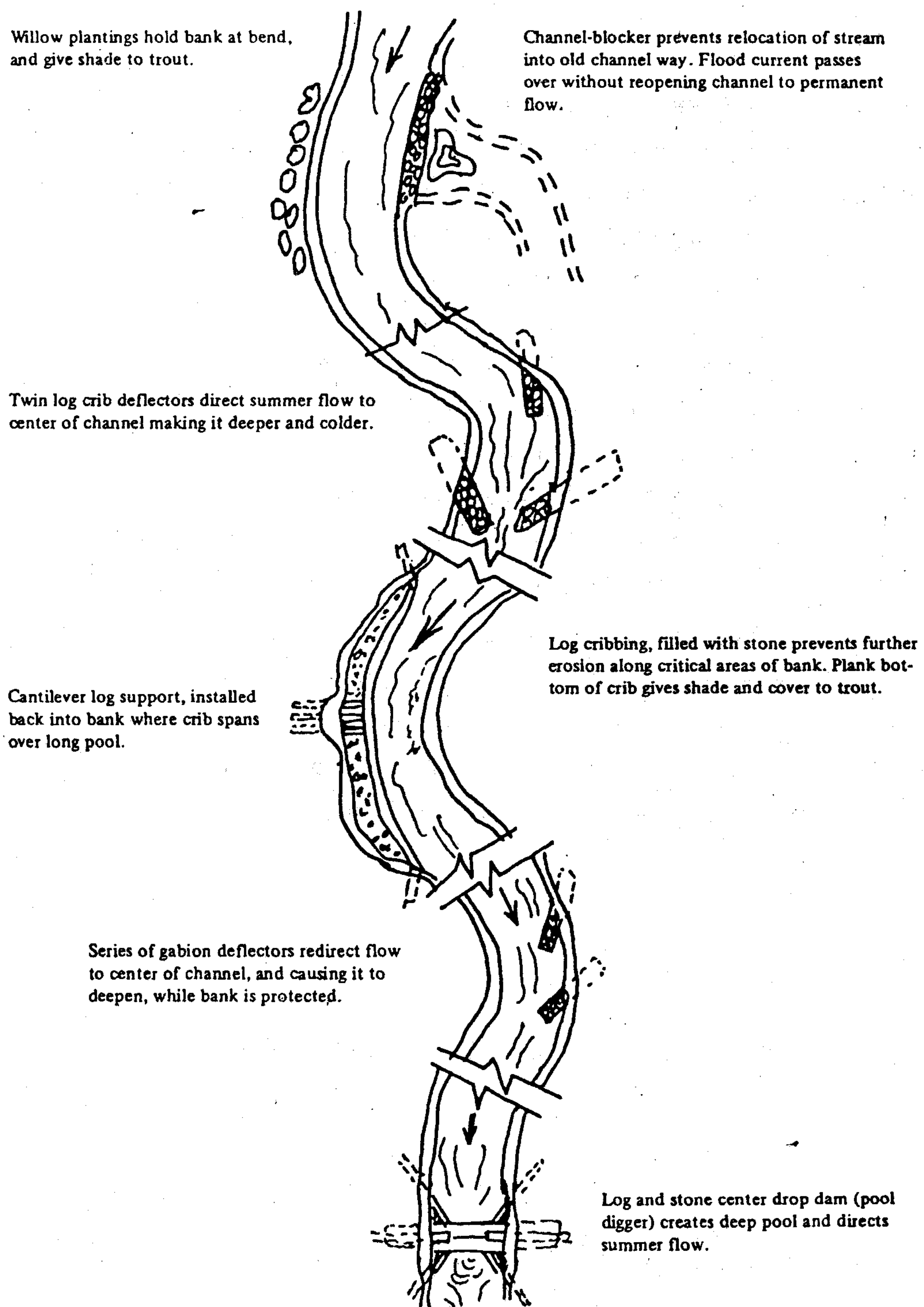


Figure 21
Typical Stream
Restoration
Ref. 77

b. Obtain right-of-ways for access.
c. Remove debris, underbrush, trash, flow obstructions.
d. Regrade unstable banks and stabilize them.
e. Revegetate eroding soils.
f. Improve ecological conditions.

Note that channel restoration is to increase the flow efficiency of an existing channel, and does not include extensive straightening, widening, or deepening of the channel.

In urban areas, restoration of channels may be aided by programs to control soil erosion and sediment movement, as well as discouraging any increase in the peak rates of runoff.

The use of fluvial geomorphology techniques, in-stream structures, and channel restoration will have little effect on improving the productivity of streams with poor water quality or a lack of basic nutrients. The control of both point and non-point sources of pollution is thus an important element of projects to revitalize watercourses.

10.0 RIVER MANAGEMENT

The extensive use of floodplains and stream side areas by man requires that the fluvial system be carefully managed to minimize conflict between man and nature. The management goals include drainage and flood control, water supply, recreation, aesthetics, open space, wildlife conservation, and others.

In rural areas, river management goals tend to focus on maintaining natural flow capacity, preventing encroachment, recreation, and wildlife conservation. Management techniques include regulation of wetlands, acquisition of river corridors, flood insurance programs, and providing recreation facilities. Occasional river restoration projects are implemented to improve the quality of degraded areas (removal of trash and debris, streambank erosion control, planting vegetation).

In suburban and urban areas the river banks and floodplains are frequently already developed and suffer from excessive flooding, fluctuating periods of erosion and sediment deposition, poor ecological conditions, lack of access to the water, and visual blight. Conventional management techniques for erosion and flood control in developed areas have been the construction of large rectangular or trapezoidal channels and underground conduits to improve flood flow capacity and prevent channel erosion. These channelization projects generally succeed in reducing flood damages, but fail to provide an environmentally sound river.

The renewed interest in our environment during the last 20 years has led to a great deal of opposition to the conventional channelization projects and spurred the search for alternates. Non-structural flood control projects and flood insurance requirements are reducing the further growth of flood plain areas and attempt to control flood damages, but fail to protect existing developed areas. In addition, non-structural techniques and flood insurance programs fail to restore and revive our rivers or to provide open space for recreation and wildlife.

There is still a need for flood control projects, particularly in developed areas, and the writer contends that properly designed

projects can enhance the environment by simulating natural conditions. Natural river corridors that are of poor quality may also be restored to a higher quality by the application of fluvial geomorphology principles.

10.1 PLANNING PROCESS

The alteration of river channels must be carefully planned in order to create a system that has adequate hydraulic capacity, is dynamic over short-term periods yet stable over the long-term periods, and is an attractive, environmentally sound community asset.

The first step is to study and understand the morphology, hydraulics, and sediment transport characteristics of natural rivers through use of basic data such as in the first part of this report. This data can then be applied in the planning and design of both minor river restoration projects and major flood control projects.

The planning process should be conducted by a multi-specialist team including hydraulic engineers, biologists, a soils engineer, a landscape architect, and a community planner. The project site should be inventoried to identify areas of critical concern:

a. flood damage areas
b. erosion and deposition problems
c. water quality and aquatic habitat
d. wetlands
e. vegetation
f. wildlife
g. soils and geology
h. aesthetics
i. recreation potential

After the inventory, the project team has to identify areas of high community value based on multiple criteria, and decide where improvements are desired to provide additional flow capacity and to enhance or protect environmental characteristics. Every river is unique in how it relates to natural and man made surroundings, and

therefore each requires unique solutions. For example, increasing flood flow capacity of rivers with high environmental value may require construction of a new floodway that is roughly parallel to, but not adjacent to the river. Rivers with lower environmental value may be directly modified to provide more capacity, but only if an effort is made to help restore the environmental characteristics and mitigate damage.

The remainder of this chapter discusses some specific concepts to be considered in planning river projects by applying fluvial geomorphology to establish the size, shape, and pattern of the overall channel and floodplain, with detailed in-stream and streambank management techniques utilized to create diverse habitats and to improve the overall appearance.

10.11 CHANNEL DESIGN CAPACITY AND FLOODWAYS

Large channels that are designed to carry the total flow of major floods within their banks will tend to be unstable because the normal flows will not be able to transport the sediments in excessively large, wide channels. Channels that are undersized and unlined will erode badly, and be unstable until enlarged. The flow capacity of new channels should be set so as to maintain them in equilibrium, unless special conditions warrant the frequent sediment removal from large channels or linings to prevent erosion of small channels.

With respect to the fluvial geomorphology, the data in Section 4.1 of this report indicates that channel equilibrium typically occurs in rivers with a bankfull capacity equal to a 1.5 to 2.3 year flood return frequency (mean annual flood analysis). Flood flows in excess of these rates are normally conveyed by floodplains. If the floodplain is not acceptable for conveying flow (due to development, etc.), a manmade "floodway" may be used. A prime consideration in the design of river projects is the use of a dual conveyance system, similar to the above natural combination of a river channel and its adjacent floodplain. In situations where natural floodplains were

undersized, absent, or have been developed and filled in, a synthetic floodplain (also called a floodway) is being excavated to carry excess flows.

This allows flood control projects to have their main channel proportioned to convey normal flows and floods with average return frequencies of up to about 2 years. The normal flow channel is thus relatively narrow compared to traditional flood channels, and can have sufficient concentrated flow to maintain adequate water depth for aquatic life and sediment transport. This helps improve both the environmental quality, physical stability, and the appearance of the channel.

After setting the size of the main channel to carry normal flows, the man-made floodway is dimensioned so as to convey the excess flood flows. The surface of the floodway is above the normal flow depth of the main channel, and therefore is usually dry. The large size (compared to the main channel) and dry surface of the floodway allow for the planting of ground covers, and occasional trees. The floodway serves as open space, can be used for recreation, and is a green belt corridor serving as a buffer between the main channel and more intense land uses. The floodway may be on either or both sides of the main channel, and can be designed to vary in width to offer some aesthetic variation. Where the natural rivers are adequate to convey normal flows, additional capacity for flood flows may be obtained by creating a floodway without disturbing the natural channel.

The concept of using the artificial floodway has been found to be an attractive alternative to conventional flood control projects. Applicable areas have been in suburban and low density urban areas where structural improvements are cost effective, and where land can be acquired. Note that in rural areas, structural type flood control projects are often not cost effective, while in highly urban areas there may be difficulties in acquiring sufficient land or in relocation of residents or business.

10.12 PATTERN AND SLOPE

Man-made channels should have a pattern compatible to the bed slope and discharge rates if long-term stability is desired with minimal erosion and deposition. The first step is to identify the pattern that the river would seek to develop for the available slope and discharge. In general, the vertical change in elevation of the proposed channel is controlled by existing geologic and topographic conditions. However, the length of channel may be adjusted by altering the sinuosity to establish the desired channel bed slope.

The optimum channel slope for a given pattern may be estimated using the equations of Section 3.4 by Lane, Ackers, and Charlton, Leopold and Wolman. Knowing the bankfull discharge, the range of stable slopes for each pattern can be estimated. The designer should then set the new channel slope to be compatible with the proposed channel pattern being planned.

The desirable channel patterns are sinuous or straight, in order to avoid the instability and large width of braided channels. The meandering pattern has advantages in that it is conducive to a wide range of habitats, and is attractive. The straight channel requires little land and is easy to construct. The disadvantage of meandering channels is that lateral and downstream movement of meanders on active rivers can consume valuable land and cause problems at bridge approaches.

Under many conditions, it may not be possible to create the desired pattern when excess slope is present. In these cases, the mean channel slope may be reduced by providing one or more drop structures to lower the bed at a fixed point protected from scour. An example of this type of situation would be where site conditions require a straight single stem channel, and the steep slope dictates a braided or meander channel. A stable, straight channel could be made, if the excess slope is absorbed at a drop structure, or by lining it to prevent erosion.

In summary, one should identify the relationship of pattern and slope, and then set the length, or channel grade changes at chutes or

drops to provide a slope corresponding to the desired pattern.

10.13 CHANNEL CROSS SECTION DESIGN

The mean channel depth, width, and side slope conditions are all set after determining the capacity and slope for the desired pattern. Engineers can readily determine the required cross section geometry based on rigid boundary hydraulics for a given discharge and slope. The tendency has been to use relatively compact cross sections that minimize the perimeter length, excavation, costs, and land.

In order to avoid the frequent instability of the man-made river channels, it is recommended that the width-depth ratio of alluvial rivers that are subject to both erosion and deposition be similar to those of regime rivers as described in Section 4.2 of this report. The author suggests that the relation between the width and depth of channels with moderate sediment loads be based on regime or tractive stress equations, with the actual dimensions set by the hydraulic criteria for the acceptable water surface profile. The initial cross section size and shape is then checked for stability and sediment transport as per section 10.14 of this report.

The values of width and depth should be considered as mean values, with both plus and minus dimensions being used at various points to provide for converging and diverging flow along the meandering thalweg. The width, depth, and side slopes may vary to provide a diverse habitat, and interesting topographical features for aesthetics.

The optimum channel width where floodplain flows are to be avoided is the maximum size that can exist without the formation of excessive sediment bars at normal flow rates. If the channel is too narrow, it will be subject to excessive scour and degradation. This is undesirable particularly in urban areas as it may undermine structures and is environmentally unsound. Channels that are too wide will have a tendency for sediments to settle, forming unstable bars and collecting debris. This optimum width should be based not only on the data of Section 4.2, but also on sediment transport computations and observations of stable segments of the river up and down stream.

Where the channel changes in width due to irregular flow patterns, erosion, deposition, and energy losses, special attention is required to the design details of transition sections. At points where the channel contracts, the decrease in width will be accompanied by an increase in velocity that may scour the bottom. Therefore, the depth must be increased as per Section 7.5 of this report, or the channel must be lined to resist erosion.

At channel expansions, the decrease in velocity will encourage the formation of sediment bars. If these are unacceptable due to flow obstruction, then the channel depth should decrease towards the banks with a well-defined thalweg to concentrate normal flows at a scouring velocity.

10.14 CHANNEL STABILITY

The ability of the channel's bed and bank material to resist movement in non-alluvial channels and the rate of sediment movement in alluvial channels should be carefully analyzed after the initial trial dimensions are established.

The stability of alluvial channels with low sediment loads and non-alluvial channels may be analyzed with the threshold velocity or tractive stress methods.

The method of threshold or permissible velocities may be used by either assuming the channel size or determining it with regime formulas, and then computing flow velocities with a flow equation. The flow velocities are compared with threshold velocities to indicate if the channel is stable (see sections 6.2, 7.52 and 7.61).

The tractive force method can be used by determining the critical stress on the channel bed at the center, and then solving the allowable depth of flow. The width of the channel is then easily determined by knowing the permissible discharge per unit width. The side slopes are then evaluated based on the shear and angle of repose. As an alternate procedure, an initial trial channel size can be determined using the U.S. Bureau of Reclamation data in Section 6.36, which is then checked for stability. General information on this method is discussed

in Section 6.3.

The "Regime Theory" equations may be used to set the size of alluvial channels with mobile boundaries of fine soils with steady flow. Their application should be on streams with high sediment loads, although studies by Bray on Canadian gravel bed rivers showed regime equations to be valid with little sediment movement (see Sections 4.2, 7.51, 7.62 and 7.71).

The design of channels with high sediment loads may be based primarily upon sediment transport requirements. Hydraulic design of these channels is not very meaningful if rigid boundaries are used, due to changing size and roughness. The sediment load is first estimated by measurement or prediction methods, and one or more sediment transport formulas of the type presented in Section 6.4 are used to determine the required channel size for the given sediment load in order to maintain a dynamic stability. The procedure cannot be expected to yield highly reliable results due to the difficulty in determining sediment loads, their variations with time, and the reliability of sediment transport formulas.

The above methods for estimating the size of alluvial channels are useful guides, and must be combined with a good deal of engineering judgement and experience while research continues.

As part of the channel stability analysis, it is important to note that it is not necessary to avoid all bed movement. The desired condition is for a dynamically stable river with the sediment transport rate equal to the sediment load.

In clear water channels with little sediment load, and in non-alluvial rivers, the channel should generally be designed for non-scouring flow conditions.

Even where the overall channel is found to be stable, it is still desirable to locate local areas subject to non-typical conditions (bends, steep slope areas, contractions, expansions). Sediment transport and scour formulas can be utilized to identify sediment sources or sinks where deposition or erosion are likely. Because of cost and data limitations on small projects, frequently only a limited investigation is possible. The most important concept is to find comparative sediment transport rates at several cross sections, and areas with above or

below average sediment transport capacities. Cross sections with relatively low sediment transport rates may be prone to deposition while areas with above average sediment transport rates may be subject to scour.

Where the sediment transport capacity is low, it may be necessary to have an increase in transport capacity by increasing slope or decreasing stream width. If this is not possible, access can be provided for the periodical removal of sediment.

In areas with excess sediment transport which are subject to scour, the channel can be lined, increased in width, or decreased in slope to reduce flow velocities.

The Corps of Engineers suggests that only the Colby formula be used for these rough estimates of sediment transport in man-made channels.[87]

10.15 DESIGN OF PROFILE DETAILS

The man-made channel should include a continuous series of pools and riffles with a meandering thalweg regardless of the channel pattern. This will recreate the irregular geometry of the natural channel, without having to wait through a period of instability as the channel tries to create its own micro-pattern.

The geometry of the thalweg meanders and the location of the pools and riffles can be based upon Sections 3.1 and 3.6 of the report. The spacing between pools should be 10 to 14 times the channel width, and be at the outside of the meander bends (or thalweg bends in straight channels).

In straight channels, the meandering thalweg may be created by having asymmetric banks, varying width to converge flow, or artificial sills or point bars to force flows to cross the center of the channel. Riprap may be used to control the bed elevations in areas designated to be riffles.

The U.S. Soil Conservation Service recommends that at least 35 percent of the channel consist of riffles, with at least another 35 percent in the form of pools.[90] The ponded areas at pools have no slope, and may allow use of artificial falls for aesthetics or to aerate the water.

10.16 CONTROL OF AGGRADATION

The aggrading channel fills with sediments due to a sediment load greater than the sediment transport capacity. Knowing this, the aggrading of existing channels can be controlled where necessary by reducing the sediment load or by increasing the sediment transport capacity.[92]

The sediment load may be reduced by land use policies to discourage erosion, use of sediment traps or reservoirs, stream bank stabilization, and debris basins. The measures can often be readily applied on small watersheds, but are difficult to implement on large watersheds.

The sediment transport capacity can be increased in several ways. The easiest way is to make the river narrower to force flows to converge in the center of the channel, where the higher velocities will carry more sediment. This may be accomplished by using training structures such as pile dikes, jacks, rock dikes, or groins. Other ways to increase sediment transport include flow augmentation, increasing slope, and reducing hydraulic friction.

10.17 CONTROL OF DEGRADATION

The degradation of channels occurs when the rate of sediment transport exceeds sediment supply. The degradation may be controlled by either decreasing the sediment transport of the river or increasing supply.[92]

The transport rate can be reduced by decreasing the flow rate, or decreasing the slope to reduce velocities. The slope of small rivers is often reduced by using a series of sills or weirs to establish a fixed bed profile at an equilibrium slope. Larger rivers have their peak rates of discharge reduced by impounding reservoirs that store large quantitites of water for gradual release over a period of time. Widening the channel can increase friction and in that way decrease the scouring velocities, but is only practical where sufficient land is available for use.

One of the more common ways to control degradation is to line the channel with an erosion resistant material. Techniques include the

use of heavy rock riprap, concrete slabs, retaining walls, and gabions of small rock contained in wire mesh baskets.

Increasing the sediment load is seldom intentionally done because of the concern for downstream areas. However, it occurs inadvertently when people attempt to fill depressions or scour holes with bed load size material.

10.18 HYDRAULIC CONSIDERATIONS

The hydraulic analysis of channels designed to resemble natural rivers is much more complex than the analysis of uniform channels. The analysis of rivers with irregular cross-sections, variable cross-sections, alluvial beds, or a channel with a shifting bed, needs to consider the headloss at bends, contractions, expansions, effect of sediment on the friction factors, irregular bed and floodplain geometry, secondary currents, flow separation at bars or islands, and non-uniform velocity distribution. For some in-bank flow conditions, meandering channels have been found to be more efficient than straight channels due to having less form resistance.[56] For high stages above bankfull, floodplain conveyance is reduced when meandering channels cross the floodplain at an angle to the floodplain flow.

The time consuming computations for water surface profiles in rivers with complex geometry is almost always done with high speed computers. The current programs in use are all limited to rivers with rigid boundary conditions without scour or deposition, and with constant friction factors that do not vary with flow conditions. Several assumptions thus have to be made even with the computer analysis.

There is research in progress on developing and testing hydraulic computer programs for use on rivers with moveable boundaries and lateral flow, but they are not yet acceptable for general use. Among the institutions with work in progress are Colorado State University, San Diego State University, and the U.S. Army Corps of Engineers. Physical models are also used to study alluvial rivers and sediment transport.

The level of effort and cost of using advance computer programs and physical models prohibits their use on small projects. However,

significant improvements in the stability and environmental characteristics of riverine projects can be achieved by applying known principles of fluvial geomorphology combined with conventional one dimensional computer models already available.

10.2 ECOLOGICAL CONSIDERATIONS

Restoring and improving terrestrial and aquatic habitats should be a major item in the planning and design of river projects. This applies to both the restoration of existing channels and the construction of new ones.

The first step in providing productive and diverse habitats in river projects is through application of fluvial geomorphology concepts. This helps to simulate the natural channel characteristics and provides the physical stability and diversity that is needed. In addition, the in-stream and streamside procedures described in Section 9.4 can be used in the channel and adjacent areas.

The combination of applying fluvial geomorphology for the overall project features combined with in-stream techniques will help mitigate ecological damage.

10.3 BRIDGE DESIGN

The depth of scour in riverbeds at bridge piers and abutments is an extremely important issue for civil engineers for both safety and economic reasons, and should be addressed from both geological and engineering aspects.

The question of safety arises because one of the more common modes of failure for small bridges is due to settlement of bridge footings. While large bridges are usually placed on bedrock or piles, the much more common bridges with simple, single spans in the magnitude of 50 to 75 feet often have shallow spread footings. The author has inspected many bridges of this size over rivers in Connecticut and estimates that at least one third of them had their footings exposed. In addition, the author is familiar with numerous bridges that failed as a result of flooding, and in each case the

failure is caused by excessive settlement and displacement of the footings due to scour.

The issue of economics is important because of the high cost of bridges and their replacement. Seventy-two bridges were damaged or destroyed by floods in Connecticut during 1955 incurring tremendous repair costs. Many of the bridge failures occur to structures that are in excellent condition other than the footings. Constructing the footings deeper than necessary is also expensive, as the footings and abutments thereon are a significant portion of the bridge costs.

With the above factors in mind, it is important for bridge designers to recognize hydraulic and geologic properties of the river.

The Highway Research Board of the U.S. reports that 46 out of 87 bridge designers and State Highway Departments responding to a survey indicated the possible depths of scour at bridges were based purely on "engineering judgement," and an additional group of 10 did not make any scour predictions.[22] With this in mind, it is important for engineers to develop the practice of fluvial hydraulics and apply it in bridge design.

A number of items are presented as recommended river morphology and hydraulic design considerations at bridges.

10.31 LOCATION

The preferred location for bridges over rivers is at straight reaches of the river with uniform flow, stable, noneroding banks and bed material, where the bridge alignment is perpendicular to the river flow, and where a short (inexpensive) span will be able to span as much of the floodplain as possible. The river bed at bridges built over riffles may aggrade and have sediment problems, while bridges located over pools may be subject to scour.[13]

10.32 HYDRAULICS

The bridge opening should be properly sized for the specified design flood, with consideration for possible changes in flow due to land use conditions. For the majority of bridges, it is not economically

feasible to design for the maximum possible flood. In those cases, both the superstructure and foundations must be designed so that they will not be destroyed even if the road or embankment is overtopped.

While the bridge itself should be designed to be above the projected water level, the bridge approaches may sometimes be at lower elevations to pass excess floodplain flows around the bridge, or act as a fuse plug to protect the bridge from destruction.

The U.S. Department of Agriculture, Soil Conservation Service, has developed a table of data to indicate when bridges become susceptible to failure due to high velocity and stage relationships for bridges, railroad bridge approaches, and road bridge approaches as noted below:[89]

VELOCITY 1/	FLOOD DEPTH 2/ Bridge	Railroad Approach	Road Approach
Greater than 15 feet per second	2' below low bridge cord	Over 1 foot	2-4' & over
10 to 15 feet per second	1' below low bridge cord	Over 1 foot	2.5-4 feet & over
5 to 10 feet per second	At bridge floor level	Over 1 foot	Over 3 feet
2 to 5 feet per second	2' over bridge floor	Over 1 foot	Over 4 feet

1/ Channel velocity at bridge for superstructure flooding and velocity around bridge for approach failure.

2/ Depth of flooding in low point of road at start of approach ramp.

The hydraulic analysis should consider the alignment of the river, floodplain width, headlosses, downstream water surface profile, clearance for ice and debris, variable friction factors, and the effect of the utility pipes suspended beneath bridges.

10.33 SCOUR

The depth of scour at the bridge opening should be estimated prior to design of the substructure. The scour depth estimate is composed of three elements:

a. The normal changes in depth and degradation of the river should be reviewed assuming that it may occur even if the bridge is not built. For alluvial rivers, the methods of Section 4.3 may be used to predict normal depth, while the information from Section 7.5 of this report may be used to make a rough estimate of degradation.

b. After estimating the minimum bed level of the river without the bridge, the effect of the constriction must be considered. The scour depth at the bridge constricted may be predicted based on the equations of Section 7.6. If only one equation is used, it should be carefully selected to match the site conditions as closely as possible. Simons suggests using all applicable methods, taking their mean, and increasing it by 25 percent for design purposes.[76]

c. The scour due to local irregular flow at bridge piers and embankments will extend below the elevation of the combined degradations and constriction scour. The equations of Sections 7.6 and 7.7 can be used where applicable, or in combinations as Simons suggests.

The total depth of scour at the bridge is the summation of degradation, constriction, and irregular flow scour. The computed value should be compared with other bridges on the same river, or on comparable rivers in the same area to see if a reasonable value has been obtained.

It is recommended that a conservative approach be used in design of hydraulic systems subject to scour. Of the many formulas available, the designer must select one or more of them for actual use. The

formula chosen should be carefully investigated to insure it is valid for the particular application, and intuition and field inspection will be needed to avoid unreasonable solutions. Simons suggests that scour depths be predicted by several methods, using their average for design.

When estimating the scour depths, the channel alignment must be considered to determine if a change in alignment would alter the streambed elevation. For instance, a bridge located on a straight section of a meandering river may in time be positioned over a meander bend. The presence of a meander bend at the bridge would generally be accompanied by a scour hole at that location due to the bend.

The bridge designer should try to anticipate how future, potential changes to the watershed or river will affect the bridge. A change in land use may cause major changes in flow rates and sediment loads that could upset the assumed design conditions. Upstream channelization could increase flow velocities and scour, or the construction of a dam could lower the sediment load and thus increase scour. Others have described how the excavation of sand and gravel from the river downstream of a bridge led to an unplanned degradation of the bed by headcutting that reached under the bridge.[33]

10.34 BRIDGE SUBSTRUCTURE

A preliminary design of the bridge abutments, piers, and embankments should be conducted, placing the lowest bearing surface below the projected scour depth. If the cost becomes excessive, the substructure may be placed in shallower water and protected from scour by any one of several techniques. Common protection measures include riprap, sheet piles, piles, rigid linings, and vegetative cover.

10.35 CLEARANCE

Most bridges are designed to have the bridge superstructure

above the projected water surface level, plus freeboard or clearance. The amount of clearance should be based upon expected wave heights, ice or debris conditions, tidal influences combined with riverine floods, and navigational requirements.

10.36 SCOUR AND FLOW CAPACITY

There will be cases where it will be possible to reduce bridge scour by increasing the bridge span, or changing its position with respect to the river and floodplain to gain greatest efficiency.

One advantage of not using a protective liner to prevent scour is the additional hydraulic capacity of bridges during scour. The material removed from the riverbed during floods provides an increase in the flow area beneath the bridge. Bauer has recommended that to provide additional flow capacity, the scour protection layer of riprap should be placed 0.4 times the normal opening height below the riverbed. He says that the sediment placed over the riprap will be eroded during floods if the flow velocity in the constriction is 2 to 3 times the floodplain velocity.[4] Laursen also recommends that the riprap at bridges be placed below the normal bed elevation, in order to reduce scouring forces on it.[36]

In addition, by increasing the depth of the bridge opening, the scouring action is accompanied by a change in the form of energy at the bridge.

The material that is scoured from the channel beneath the bridge will often be deposited a short distance downstream as flow velocities decrease over the floodplain. This material has been observed by the author in the form of downstream sediment bars. These may obstruct the flow of water leaving the bridge, and cause an undesirable tailwater condition at the bridge. A large part of the energy losses at bridges actually occurs downstream of the bridge where flows may be affected by sediment deposits that obstruct the expansion of the effective flow area.[20, 34]

Bridges that are undersized, or designed to have a high allowable headloss, create an increase in flow depth on their upstream side. The

low velocities in headwater pools encourage the deposition of sediment. In extreme cases, aggradation may result from the bridge altering the pattern of flow approaching the bridge as well as causing increasing levels of upstream flooding.

10.37 BRIDGE PIERS

The upstream face of bridge piers should be designed to minimize headlosses, avoid catching debris, and reduce scour. The shape of the pier's upstream face can be designed to have a low headloss coefficient by using the shapes mentioned in references 34 and 10. In a similar manner, the pier shapes most susceptible to scour can be avoided by referring to data in references 41 and 45. For bridges with critical or super critical flow, the U.S. Army Corps of Engineers extends the pier upstream of the bridge, with a sloped leading edge.[87] This reduces the energy loss and flow depth under the bridge by having the flow separation at the pier occur before the bridge construction. The pier extension also helps to catch debris before it can go into the bridge opening.

10.4 BRIDGE SPUR DIKES

One of the methods that may be used to reduce scour and improve hydraulic efficiency at bridges is the construction of spur dikes (also known as guide banks). They are earth or rock embankments projecting upstream from the bridge abutments with an increasing width between them. They can serve the following functions:[53, 8, 22]

a. Confine flow from braided rivers and from broad floodplains into a single channel under the bridge.
b. Improve the distribution of flow in the channel cross-section.
c. Control the skew angle of flow approaching the abutments and piers.
d. Break up meander patterns to prevent bends and bend scour from occurring at bridges.
e. Prevent floodplain flows parallel to and along the toe of the road embankment.

f. Reduce the rapid convergence of flow that normally occurs at bridge abutments.

Most of the research reports agree that the spur dike at the bridge be parallel to the channel centerline, and gradually curve away from the center as one goes upstream. The recommended shape of the curve is that of an ellipse, with a 2.5 to 1 ratio of the major axis to the minor axis.

The literature indicates there is a difference of opinion on the required length of the spur dike. A "rule of thumb" used in Canada on alluvial rivers is to have the upstream length equal to three quarters of the bridge span, and a downstream length of one quarter of the bridge span.

A Russian scientist named Andreev published recommendations on the length of spur dike in relation to the flow distribution. He provided data from which the desired ratio of the spur dike length to the bridge span can be determined from the ratio of the total flow to the main channel flow.[53]

QT/Qc	1.0-1.2	1.25	1.5	1.75	2.0	2.5
L_{SD}/W	0	0.15	0.3	0.45	0.6	0.75

Where: QT = total flow

Qc = channel flow

L_{SD} = length of spur dike

W = bridge span

The value of the term "L_{SD}" is the total length of spur dike for both sides of the bridge. This total length would be distributed between the two sides in proportion to the floodplain flow distribution on the two sides.[53]

Research work at Colorado State University and field surveys

by the U.S. Geological Survey have been adopted by the U.S. Department of Transportation for estimating spur dike lengths. They felt that the width of the bridge span was only important up to 100 feet, and used the ratio of the floodplain flow and channel flow in the 100-foot wide section as their key parameter. The U.S. DOT provided a figure which is used, knowing the flow ratio and velocity, to determine the spur dike length. They also recommend a minimum length of 100 feet, with a 50 percent increase for bridges at a 45 degree skew angle to the channel.[8]

In addition to reducing scour at bridge abutments, spur dikes reduce the headloss at bridges. This is because the spur dikes reduce the rate of flow contraction as water approaches the bridge, and help prevent flow separation occurring at bridge abutments. The above factors have been found to encourage a more uniform velocity distribution through bridge openings, and help increase the "effective" bridge opening.

Unfortunately, there is little quantitative information available on the amount of energy (or headloss) that is saved when using spur dikes. Laboratory reports from Lehigh University indicate that a simple straight line spur dike can increase the effective width of bridge openings by 10 to 15 percent.[21] U.S. Geological Survey reports allow the bridge discharge coefficient to be increased by up to 40 percent for elliptical spur dikes combined with certain bridge types.[48]

10.5 APPLICATIONS

The application of fluvial geomorphology to flood control and river restoration projects has been very limited. Several documented cases have been on large rivers where the cost of the project justified the use of physical models at a level not permissible by the planning and design budgets for urban projects. Other projects have mostly been small stream restoration projects intended to improve fish habitat, but did not need extensive hydraulic improvements.

In Connecticut, there are several moderate size projects in the

design and construction phases which include increasing the hydraulic capacity of the river system while trying to simulate the shape and form of natural rivers and floodplains and to minimize environmental damage.

The Farm River Flood Control Project in East Haven, Connecticut involves channel, floodway and floodplain improvements and four new bridges along three miles of the Farm River. The original channel was meandering, alluvial, and underfit on a mature floodplain and terrace formed by glacial meltwaters. Portions of the floodplain had been developed with several suburban residential neighborhoods during the 1950's and 1960's. Approximately 300 houses, 15 commercial businesses, and a public school and indoor hockey rink were subject to flooding by flows with an average return frequency of 100 years, many of which are flooded as often as once in 10 years.

The $4.5 million project is being jointly funded by the Department of Environmental Protection and the Town of East Haven, with the U.S. Soil Conservation Service planning to fund an upstream detention dam.

Two sections of the project (from Willow Road to Hellstrom Road) included limited widening of the existing channel on its west side, while preserving the bank and vegetation on the east side of the channel for wildlife cover and to shade the river. A floodway has been constructed adjacent to the west side of the channel in order to provide a man-made floodplain at a lower elevation to convey peak flows. The floodway is being used as public open space for passive recreation, and a three mile long bicycle path is being provided along the river bank.

The size of the main channel was proportioned so excess flows would be conveyed on the floodway approximately twice a year. Although not yet completed, the project has already successfully conveyed a major flood in excess of the main channel capacity without damage. The main channel is conveying summer low flows without exposed bottom areas or stagnant water, and is maintaining its planned nonuniform cross section of variable width and depth without excessive erosion or deposition.

In the vicinity of Gloria Place, a second floodway was constructed to enable flood flows to by-pass a large meander loop which had high ecological value but low flow capacity. Normal flows remain in the original natural channel, and a fuse plug weir was constructed at the outside of a bend so that peak flows go into the floodway. This weir and floodway were operational twice in the first year, and performed very well. Limited erosion occurred at the weir during the first flood which was before the construction and restoration were finished.

The design of the $14 million Trout Brook Flood Control Project in West Hartford, Connecticut also includes the use of a relatively narrow main flow channel designed for the mean annual flood, supplemented by a floodway to carry excess flows in major floods. The main channel has a meandering alignment featuring a variable width and a profile designed to create artificial pools and riffles. The flow velocities required a channel lining of riprap, and a partial floodway lining of celleur concrete mats with topsoil and grass over them. As in the Farm River Project, the floodway will form a linear park along the river and will have walking and bicycle paths.

This project is of special interest because it included a complete redesign to change the plans from the original 1960's concept of using a conventional trapezoidal channel. There was strong public interest in having the flood control project in combination with a green belt corridor for open space.

The concept of designing flood control projects to simulate a natural river system has been well received by the public. Advantages are the ability to create an acceptable aquatic habitat, providing a green open space corridor along the river, improved aesthetics, and optimum sediment transport. The disadvantages that have been experienced include the need for more land than is used in traditional trapezoidal or rectangular channels, maintenance requirements for the grass floodways, and the increased complexity of the design and construction.

The cost of the river corridor/floodway type of project can be

either above or below that of traditional trapezoidal channels, depending on the individual situation. The floodway type of project generally requires more excavation for a given flow capacity than a compact trapezoidal channel. However, the author has found that the floodway type of projects have lower flow velocities and often do not need artificial linings. The cost of channel linings is generally more than the cost of the extra excavation.

There has been a reluctance to implement the river corridor/ floodway type of project in towns with little interest in maintaining an attractive environment. Some public officials, including engineers, are reluctant to implement new concepts and have little concern about ecological conditions.

The river corridor/floodway concept has been particularly favored in suburban areas where:

a. A structural type of flood control project is required due to high flood damages that preclude non-structural solutions
b. There is a desire for open space
c. Residents have strong interest in the environment
d. Land is available.

The performance of the above projects will be closely monitored to establish long term performance trends.

10.6 FUTURE TRENDS

Several Federal Agencies and policies are placing greater emphasis on environmental considerations during river modifications and thus encourage greater use of fluvial geomorphology concepts.

Section 404 of Public Law 91-500 authorizes the Corps of Engineers to monitor and regulate river and channel projects to protect water quality. Fish and wildlife habitats are a major concern, as well as sediment and erosion control. Consequently, channel designs need to be more responsive towards environmental considerations.

The U.S. Department of Agriculture Soil Conservation Service is also encouraging the design of environmentally sound channels, as per their recently revised manual on the "Design of Open Channels".

The American Association of State Highway and Transportation Officials recently published "Highway Drainage Guidelines" that are widely used on federal and state funded highways. The guides recommend use of fluvial geomorphology concepts for relocation of channels and mitigation of ecological losses caused by highway construction.

In Great Britain, the Agricultural Ministry is now requiring that river projects be planned to mitigate environmental and aesthetic losses, and is publishing policy guidelines to present the general concepts to be utilized.

As a result of the continuing environmental concerns related to wetlands and rivers, we can expect to see an increased interest in reducing environmental losses and compensating for unavoidable losses. The application of fluvial geomorphology principles is expected to be a key factor in achieving these goals and is already becoming a matter of federal policy.

The fact that mitigation measures are available should not be counted on to make channelization projects permissable. The use of fluvial geomorphology design techniques should be considered only when channel improvement or restoration is absolutely necessary. Channel improvements should never be conducted without first evaluating non-structural alternatives which generally have little or no environmental damages.

GLOSSARY

Aggradation: A general raising of the river bed by deposition of sediments over a long period of time and over a long distance.

Alluvial Rivers: Alluvial rivers have banks and beds of unconsolidated sedimentary material subject to being eroded, transported, and deposited by the river. The channel geometry and flow conditions are interrelated.

Armor: A natural layer of particles, usually gravel and cobble sizes, that may cover the surface of the bed as a coarse residue following erosion of the finer bed materials for a particular flow range. The layer is often only one to three particles thick and reduces erosion of underlying particles.

Bar: A sand wave of approximately triangular cross section in a vertical plane in the direction of flow with gentle upstream slopes and steep downstream slopes. It travels downstream as a result of the movement of the sediment up the upstream slope and the deposition of part of this material on the downstream slope.

Bed forms: Generic terms used to describe small irregularities on the bed. Includes ripples, dunes, antidunes.

Bed Load: Sediment that moves by saltation (jumping), rolling, or sliding along and on the river bed as it is transported downstream.

Bed-Load Discharge: The quantity of bed load passing a cross section of a stream in a unit of time.

Bed Material: The material of which the streambed is composed.

Channelization: The widening, deepening, and realignment of natural channels in order to increase their flow capacity.

Channel Restoration: The process of improving the flow conditions and ecological value of rivers by use of minor operations to reduce erosion, remove trash, vegetate areas, etc. Does not include major channelization work.

Degradation: A general lowering of the river bed by erosion over a long period of time and over a long distance.

Floodplain: That portion of a valley that is periodically inundated by water in excess of the channel capacity. It is often formed of sediment deposits.

Geomorphology: The study of the shape and form of the earth's surface.

Pool: A section of the river with relatively deep water and low flow velocities.

Regime Rivers (also graded rivers): A situation where the rate of erosion, deposition, flow rates, and sediment conditions in an alluvial channel are in dynamic equilibrium.

Regime Theory: An empirical group of equations analytical concepts for analyzing alluvial rivers, which was originally developed in India and Pakistan.

Riffles: A section of the river with relatively shallow flow depths and higher velocity turbulent flow.

Riprap: A manmade lining of loose rock on the bed or banks of a channel used to prevent erosion.

Sediment: Fragmented and decomposed rock and soils that are transported and deposited by waters.

Sediment Discharge (or sediment load): The quantity of sediment that is carried past any cross section of a stream in a unit of time. The discharge may be limited to certain kinds of sediment or to discharge through a specific part of the cross section.

Sediment Yield: The total sediment outflow from a watershed or a drainage area at a point of reference and in a specified period of time. This is equal to the sediment discharge from the drainage area.

Suspended Load: That portion of the total sediment load that is transported by water with little contact with the bed. It includes suspended bed material and the wash load.

Terrace: A former floodplain no longer subject to frequent flooding, generally caused when a river channel erodes a deeper channel below the floodplain elevation.

Thalweg: An imaginary line along the length of a river connecting the deepest points.

Turbulent Flow: A state of flow wherein the water is agitated by cross currents and eddies, and flow velocities vary in magnitude and direction in an irregular manner.

Uniform Flow: A flow that is constant in both time and direction along the stream lines.

Wash Load: That portion of the total sediment load that is smaller than the bed material, and is being tansported from upstream areas. The quantity transported depends on the rate of supply, and usually consists of clay and silt that remain in suspension. It may include sand if the bed material is gravel.

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APPENDIX

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